Final Papers
Edited by Pierre Quenneville

Poster Papers

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Dear WCTE 2012 delegates,

On behalf of the steering committee of the World Conference on Timber Engineering 2012, we are very pleased to introduce the Book of Abstracts. This reference document summarises the full papers that cover the state-of-the-art across the breadth of timber engineering, materials, connections, architecture and manufactured products.

This conference is hosted by the New Zealand Timber Design Society and our Australian colleagues. It is the first time that WCTE has returned to its roots which started with the Pacific Timber Engineering Conference in 1984.

To those who are present at WCTE 2012 in Auckland, the beautiful “City of Sails”, we welcome you and invite you to peruse the wide range of abstracts for the 240 oral presentations that you will have opportunity to hear, or over 100 posters to see and discuss.

Following the Christchurch earthquakes of last year and the Tohoku earthquake in Japan, the resilience and post disaster performance offered by timber structures has become increasingly of interest and is emphasised in the sessions devoted to seismic engineering and case studies. Typical of last conferences, there are numerous presentations on connections, materials, lateral load resisting systems and CLT. Architecture is specially emphasised in the Wednesday sessions. There is one book of abstracts for each day and the abstracts for the posters are included in the last book.

Finally, we wish to thank the sponsors and the various contributions from the support organisations. We especially wish to thank those who gave their time reviewing the very large number of abstracts.

Hugh Morris  Pierre Quenneville
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INFLUENCE OF STRESS REDUCTION METHODS ON THE CAPACITY OF ADHESIVELY BONDED TIMBER JOINTS

Till Vallée¹, Thomas Tannert², Quentin Gadais³

ABSTRACT: Adhesively bonded joints are characterized by stress peaks towards the ends of the overlaps, which trigger failure. A repeatedly reported method to increase the joint capacity is to reduce the stress peaks by either acting on the geometry or on the adhesive. In the presented research, the effectiveness of three stress reduction methods: i) adhesive grading, ii) chamfering, and iii) reverse chamfering, was experimentally and numerically investigated. The experiments on full-scale double lap joints showed that none of the applied methods significantly affected joint capacity. A probabilistic method considering a statistical formulation of size-effects was applied to predict the joint capacity and delivered accurate values for the experimental series. The presented work allows for a better insight into the relation between stress-state and capacity of adhesively bonded timber joints.

KEYWORDS: Adhesive bonding, timber joints, joint capacity, stress reduction methods, finite element analysis

1 INTRODUCTION
Adhesively bonded joints, due to their complexity, are almost inaccessible to simple analytical analyses. To quantify the influence of parameters that deviate from the usual idealizations made on the stress-strain state, e.g. [1-3], only numerical methods such as Finite Elements Analysis (FEA) can be used. The range of parameters that justifies the use of FEA is wide: among them adherend or adhesive roundings; adherend shaping, adhesive gaps and adhesive grading.

End of the overlap to a more ductile adhesive, while the less stressed inner part is associated to a stiffer adhesive. The goal is to achieve a more even stress distribution and higher joint strengths. The concept can be traced back to the 1970’s [4-5] and is still pursued today, e.g. [6-7].

Chamfering the adherend ends to reduce the stresses in adhesive joints has also been investigated. Adams et al. [8] have shown that tapering the metallic outer adherends greatly reduced the stresses; the resulting experimentally gathered joint strengths increased more than twofold. Similar numerical results were obtained by Hildebrand [9] on single lap joints composed of composite and metallic adherends. The effect of chamfering on the reduction of stresses at the end of the overlaps of bonded joints was subsequently confirmed, often only numerically, e.g. [10-11].

When reviewing these works, two caveats have to be made: 1) numerical investigations usually point out the stress reduction effects and conclude that there ought to be a benefit for strength; (2) experimental validation has been, in most cases, performed on adherends which do not exhibit brittleness. The situation for bonded joints involving brittle adherends has not yet received much attention. This is, for a great part, due to the fact that capacity prediction methods for joints involving brittle adherends were just recently formulated [12-13].

The research presented herein studied the influence of adhesive roundings and adherend chamfering on the strength of adhesively bonded timber joints.

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2 EXPERIMENTAL INVESTIGATION

2.1 SPECIMEN DESCRIPTION

In order to investigate the effect of stress reduction methods, symmetrical double-lap joints with rectangular sections were fabricated. The joints consisted of two outer (t_o = 38 mm) and two inner adherends (t_i = 19 mm) connected by a layer of adhesive. Specimens were b = 50 mm wide, with an overlap of L = 100 mm and an adhesive layer thicknesses of t_a = 1 mm. Figure 1 details the nomenclature. The timber surface was mechanically planed and subsequently residual dust was removed. The adhesive layer thickness was enforced using PVC washers put at a distance of 25 mm from the overlaps. In all cases, the adhesive cured at laboratory temperature (22±2°C) for at least a week.

![Figure 1: Nomenclature used for the bonded joints](image1)

2.2 STRESS REDUCTION METHODS

Three stress reduction methods were investigated: (i) adhesive grading; (ii) chamfering, and (iii) inverse chamfering. Grading was investigated on Series S1 by using two different adhesive types for the bonded splice: a stiff adhesive in the centre part, and a softer adhesive towards the ends of the overlap. The adhesive grading level was defined as the ratio of the stiffer adhesive related to the full overlap.

In the frame of the investigations of chamfers, in series S2 (Figure 2), the chamfer level was varied in four steps: 0%, 33%, 66% and 100%. In series S3 (Figure 3) the inverse chamfer level was varied in four steps: 0%, 33%, 66% and 100%.

![Figure 2: Specimens series S2 chamfering levels](image2)

![Figure 3: Specimens series S3 inverse chamfering levels](image3)

2.3 MATERIALS

Two different cold-curing 2C adhesives were used: (i) a stiff and brittle epoxy, SikaDur330, not exhibiting any plastic behaviour; and a very soft acrylic adhesive exhibiting major plasticity, SikaFast5221. The adhesives were experimentally characterized in tension according to EN ISO 527-2 [14]; the results are listed in table 1.

The timber species used was Spruce (Picea abies) cut from high quality defect-free boards and conditioned to 12% moisture content prior to specimen manufacturing and then stored in constant climate until testing. The longitudinal modulus of elasticity E_1, and the transverse modulus of elasticity E_2 were determined on small clear specimens from the same boards.

The strength of the timber was characterized based upon the Norris failure criterion [15], which has, in a 2D stress state, the form given by Eq. (1):

\[
\frac{\sigma_1^2}{f_1^2} + \frac{\sigma_2^2}{f_2^2} + \frac{\tau_{12}^2}{f_{12}^*} = 1
\]

To determine the strength parameters, f_1, f_2, and f_{12}, tests were performed on dog-bone specimens exhibiting different orientations, α, relatively to the grain. All samples were tested according to ISO 527 at room temperature using an Instron universal testing machine. Four series were performed: 0°; 10°; 45°; and 90°.

![Table 1: Material properties](image4)

<table>
<thead>
<tr>
<th>Material</th>
<th>E_1 [GPa]</th>
<th>E_2</th>
<th>ν_13</th>
<th>ν_23</th>
<th>f_1* [MPa]</th>
<th>f_2* [MPa]</th>
<th>f_{12}* [MPa]</th>
</tr>
</thead>
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<tr>
<td>Timber</td>
<td>18</td>
<td>1.1</td>
<td>0.4</td>
<td>0.04</td>
<td>98.2</td>
<td>4.5</td>
<td>16.5</td>
</tr>
<tr>
<td>SikaDur</td>
<td>4.56</td>
<td>0.37</td>
<td>0.04</td>
<td>0.04</td>
<td>39.0 (ε_l = 0.85%)</td>
<td>5.5 (ε_l = 0.50%)</td>
<td></td>
</tr>
<tr>
<td>SikaFast</td>
<td>0.14</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>11.8 (ε_l = 50%)</td>
<td>11.8 (ε_l = 50%)</td>
<td></td>
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</tbody>
</table>
2.4 PROCEDURE AND RESULTS
All tests were carried out on Zwick universal testing machines with a capacity of 250 kN. Quasi-static axial tensile tests were performed under a displacement-controlled rate of 5 mm/s, in all cases up to failure load. Because of their low through-thickness stiffness and strength, the timber specimen had to be cut in dog-bones shapes, to allow for the tensile force to be introduced. All individual tests were repeated three times. All investigated adhesively bonded joints failed in a brittle manner, independently on the fact if they involved brittle or ductile adhesives. The joints almost always failed by splitting just below the end of the overlap, as illustrated by Figure 4. The experimentally gathered strengths for each series are displayed in Figures 5-7.

3 NUMERICAL MODELLING
All configurations were numerically modelled using the FEA software package Ansys. Two-dimensional 8-node orthotropic elements were used to build up a model that exploited the two symmetry axis. The timber was modelled as linear-elastic orthotropic; SikaDur330 was modelled linear-elastic isotropic, while SikaFast5221 were idealized as exhibiting bi-linear von Mises plasticity. In all cases, the mesh size was refined at the loci of stress peaks; mesh sizes were approx. 0.1mm.

For the sake of brevity, only the stress profiles of Series S2, both shear and through-thickness, are presented in Figures 8-9. The stresses do concentrate at the ends of the overlaps, and as far as adhesive grading is concerned, at the limit of the two adhesives in the bond-line.
by Eq. (2):
the probability of survival of the joint can be calculated elements
material in needed, herein the Norris criterion was used.
survival depends on the simultaneous non-failure of all constituent element i corresponding to a load level
As a result, for a given applied load, F, the probability of survival of the joint can be calculated by Eq. (2):
\[ P_s(F) = \prod_{i=1}^{n} P_{i,s}(F) \] (2)
The function \( P_s \) stands for the probability of survival of the constituent element i corresponding to a load level F. Herein a two-parameter Weibull distribution has been considered to express \( P_s \), formulated using Eq. (3).
\[ P_s = \exp \left[ -\left( \frac{\sigma}{m} \right)^k \right] \] (3)
In Eq. (4), \( \sigma \) is the stress acting over a volume \( V \), \( m \) is the characteristic stress or scale parameter and \( k \) is the shape parameter that gives a measure of strength variability.

Although initially established for main stresses, Weibull theory has been extended for any stress operator that defines failure [12-13]. The failure criterion from Eq. (1) can be interpreted as being stress operators governing the failure of the respective materials. In the following, failure functions labelled \( \phi_i \) are defined by reformulation Eq. (1) which leads to Eq. (4):
\[ \phi^2 = \min \left\{ \frac{\sigma^2_i}{f_1^2}, \frac{\sigma^2_i}{f_2^2}, \frac{\sigma^2_i}{f_3^2}, \frac{\sigma^2_i}{f_4^2} \right\} \] (4)
Consequently, if each constituent element \( i \), with a volume \( V_i \) is subjected to a constant value of the failure function \( \phi_{i,m} \), the probability of survival of the whole member is given by Eq. (5):
\[ P_s = \prod_{i=1}^{n} \exp \left[ -\frac{V_i}{V_0} \left( \frac{\phi_{i,m}}{m} \right)^k \right] = \exp \sum_{i=1}^{n} \left[ -\frac{V_i}{V_0} \left( \frac{\phi_{i,m}}{m} \right)^k \right] \] (5)
Eq. (5) can be implemented in a post-processing routine for FEA results and the strength of joints can then be easily predicted. Failure load has been defined as being the point of equal probability of survival i.e. for \( P_s = 0.5 \), which in a first approach designates the value \( F_{\text{pred}} \) for which half of specimen would survive.
The above detailed procedure was applied for all configurations tested within the experimental series. FEA had to be performed iteratively until \( P_s = 0.5 \) was achieved. The computed joint strengths are displayed in Figures 5-7, where they are compared to the corresponding experimental results.

4 PROBABILISTIC PREDICTION
As stress-based approaches, due to the huge stress peaks generated at the ends of the overlaps, are deemed to fail regarding the strength prediction of adhesively bonded joints, a probabilistic method has been pursued herein. The principles of this method were published previously [12-13], so that it is only summarized herein. The prediction method takes into consideration the scale sensitivity of the material strength, considering not only the magnitude of the stress fields, but also the volume over which they act. For a general overview on size effects and its relations to strength, the reader is kindly redirected to Bažant [16]. The following is reminded: (i) probabilistic strength prediction methods assume that the investigated material exhibits brittle failure; (ii) the material strength is then usually statistically described as being Weibull-distributed. For the implementation of any strength prediction method, including the ones based on probabilistic concepts, a failure criterion for the material in needed, herein the Norris criterion was used. Idealizing the joints under consideration as being constituted by \( n \) elements that could potentially fail, its survival depends on the simultaneous non-failure of all elements \( i \leq n \). As a result, for a given applied load, \( F \), the probability of survival of the joint can be calculated by Eq. (2):

The function \( P_s \) stands for the probability of survival of the constituent element i corresponding to a load level F. Herein a two-parameter Weibull distribution has been considered to express \( P_s \), formulated using Eq. (3).

In Fig. 9, the tension perpendicular stress profile for S2 is displayed. The influence of adhesive grading on the stress distribution is displayed in Figures 8 and 9. The two extreme grading levels, i.e. 0\% and 100\%, which correspond to a splice completely made up of the stiff or ductile adhesive, respectively, are representative for the influence of the adhesive stiffness. It can be seen that the stiff adhesive leads to higher stress peaks, compared to the soft adhesive, with maximal stress magnitudes lower by around one third. Despite this lowering in the stress magnitude, which in a stress-based context would yield in a strength increase. The experimental evidence, however, shows that strength is almost independent from the grading level. The probabilistic method on the other hand delivers predictions that are reasonably consistent with the experimental data.

Figures 8 and 9 also indicate that the corresponding stress profiles exhibit their maximum at the boundary between the two adhesives, and that these peaks are almost identical in shape and magnitude. Thus, even without consideration of probabilistic concepts, it is to expect that the corresponding joints strengths should be comparable, which is confirmed by the experimental results shown in Figure 5.

Similarly to the adhesive grading, adherend chamfering does significantly reduce the magnitude of the stress peaks. The corresponding experimentally determined
joint strengths were not affected by this particular stress reduction method. Here again, taking into account the fact that the joint strength is not only driven by the magnitude of stresses, but also by their distribution, applying a probabilistic approach, sheds light onto this apparent inconsistency by predicting joint strengths that are in good agreement with the experimental values. The results on the inverse chamfering indicate that failure is no longer governed by the failure criterion; instead cohesive failure was the dominant mode. Results show that increasing the level of inverse or internal chamfering leads to a reduction of joint strength, instead of the expected increase.

6 CONCLUSIONS

Adhesively bonded joints are characterized by extreme stress peaks towards the ends of the overlaps, which trigger failure, and thus define capacity. A repeatedly reported, and at first sight intuitive approach to increase the strength these structural elements is to reduce the stress peaks by grading the adhesives or chamfering the adherends. The research presented herein allowed for the following observations:

1. All adhesively bonded joints failed in a brittle manner, so did the materials they were constituted of.
2. Adhesive grading and adherend chamfering, according to FEA, lead to stress reductions. These reductions are typically associated to changes in the stress field, i.e. the reduced stresses act on larger volumes of the adherends.
3. Within the frame of the study presented herein, there is no direct correlation between stress magnitude and failure, even considering a verified failure criterion.
4. Since direct stress based approaches to predict the strength of adhesively bonded joints composed of brittle adherends are inapt, a probabilistic capacity prediction method was computationally implemented.
5. The probabilistic method delivered accurate joint strengths; it also offered a mechanically coherent for the apparent contradictions between stress reduction and lack of corresponding joint strength increase. It should be pointed out that these conclusions were only experimentally validated on the static strength of the joints, the authors explicitly discourage drawing any similar conclusions regarding the fatigue behaviour of bonded joints, for which the aforementioned stress reduction method might well have a positive effect.

ACKNOWLEDGEMENT

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ANALYSIS OF THE STRESS STATE OF A HALVED AND TABLED TRADITIONAL TIMBER SCARF JOINT WITH THE FINITE ELEMENT METHOD

Jose R. Aira¹, Francisco Arriaga², Guillermo Íñiguez-González³, Manuel Guaita⁴, Miguel Esteban⁵

ABSTRACT: The purpose of this study is to determine the stress distribution in the carpentry joint of halved and tabled scarf joint with the finite element method (FEM) and its comparison with the values obtained using the theory of Strength of Materials. The stress concentration areas where analyzed and the influence of mesh refinement was studied on the results in order to determine the mesh size that provides the stress values more consistent with the theory. In areas where stress concentration is lower, different mesh sizes show similar stress values. In areas where stress concentration occurs, the same values increase considerably with the refinement of the mesh. The results show a central symmetry of the isobar lines distribution where the centre of symmetry corresponds to the geometric centre of the joint. Comparison of normal stress levels obtained by the FEM and the classical theory shows small differences, except at points of stress concentration.

KEYWORDS: carpentry joints, halved and tabled scarf joint, FEM, stress concentration

1 INTRODUCTION

Joints are the points of transmission of forces between the members of a timber structure. In traditional joints stresses are transmitted from one piece to the other one by means of carpentry works that balance the axial and shear stresses through local compressions, tangential stresses and friction between the contact faces. The metallic elements are usually incorporated with the unique mission of keeping continuity of the contact faces [1]. The halved and tabled scarf joint consists of an end joint transmitting the tension axial force N through compression parallel to the grain located in the cross-section area of the notch b1, and this compression is transmitted to the entire cross-section through shear stress at the surface b1 (Figure 1).

![Figure 1: Halved and tabled scarf joint](image)

Moreover, the narrowing of the section in each of the pieces, resists the tensile force combined with a bending moment M caused by the eccentricity of the axial force N, producing a rotation that tends to disassemble the joint [2]. This effect can be avoided by making notches in the ends of the tabled joint or by placing metal straps. For easier installation procedure, a wedge is placed on each side to ensure tightly contact between the members (Figure 2).

![Figure 2: Different arrangements of the halved and tabled scarf joint](image)

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2 MATERIAL AND METHODS

2.1 DESCRIPTION OF THE MODEL

The geometric parameters of the studied joint are: h = 60 mm, t = 30 mm, b = 150 mm, b = 50 mm, l = 90 mm (Figure 1). To model the joint by FEM, the left part is considered coerced on sliding supports in all nodes preventing displacement in the X axis, and a fixed support in the upper left node to prevent movement in the Y axis. The right part receives the external load of 10 kN which is uniformly distributed in the entire cross-section. A friction coefficient between contact faces of 0.467 is considered.

The three possible failure modes have been studied in the joint: a) Bending-tension failure corresponding to the reduced section of the piece subjected to tensile and bending stresses, $\sigma_s$, b) Local compression failure corresponding to the section of the notch subjected to compression stress, $\sigma_c$, and c) Shear failure corresponding to the section of the horizontal plane subjected to shear stress, $\tau_{xy}$ (Figure 3). The critical sections are studied by comparing the stress values obtained by the application of FEM to the values obtained through the formulation of the classical theory of strength of materials in order to determine the influence of mesh size on results and the coincidence of the stress distributions obtained with theoretical values.

![Figure 3: Critical cross-sections](image)

2.2 SOFTWARE

The finite element analysis is made by a plane stress linear static study that allows the consideration of the thickness of the pieces. Wood is considered as an orthotropic material and the values of the elastic properties perpendicular to the grain are achieved by the arithmetic average in the radial and tangential directions. In order to perform the numerical simulation of the joint, each piece is modelled in the ANSYS finite element software taking the element of its internal library called PLANE42. This element is used for two dimensions modelling of solid structures and can be used either as a plane element (plane stress or plane strain) or as an axisymmetric element. The element is defined by four nodes having two degrees of freedom at each node (translations in the nodal x and y directions) and it has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities (Figure 4) [4].

![Figure 4: PLANE 42 element](image)

The model includes the simulation of contact between surfaces. Groups of two different lines are defined in the contact zone established. Each of the lines belong to a different solid but having the same coordinates and geometric position in order to obtain coincidence at the nodes of each line. These lines of friction are meshed with one dimension contact elements in the direction of the lines in order to define the surface to surface contact [3]. Thus, the contact pair is set using the internal library elements called TARGET and CONTACT.

3 RESULTS

3.1 STRESSES DISTRIBUTION

The areas with stress concentration and those with lower stress are studied. To identify graphically these regions, distribution of normal stresses $\sigma_s$ and distribution of tangential stresses $\tau_{xy}$ are showed from the ANSYS graphical output. Uniform mesh of size 2 mm is used (Figure 5).

![Figure 5: Distribution of normal stress $\sigma_s$ (on top) and shear stress $\tau_{xy}$ (at the bottom) in N/mm²](image)

The graphical output shows a central symmetry of the isobar lines distribution where the centre of symmetry corresponds to the geometric centre of the joint.
In the reduced section, subject to normal stress (bending combined with tension), there is not a point of high stress concentration except in the bottom where there is a concentration of tensile stress due to the abrupt decrease of the effective cross-section.

In the section of the notch, subject to normal stress (local compression), there are two areas of high compression stress concentration, one located at the top of the section and one in the bottom of it.

In the section of the horizontal plane, subject to shear stress, stress values are close to zero at the end of the heel, increasing progressively to high levels of stress near the central notch.

3.2 STUDY OF THE MESH SIZE

In the finite elements model four different uniform mesh sizes are used: 10 mm, 5 mm, 2 mm and 1 mm. The computer consuming time for size 2 mm and 1 mm is too long to be functional so a type of progressive mesh is required. The progressive mesh should have the same small size in stress concentration areas and increase in size progressively when the stress concentration decreases (Figure 6). Using a progressive mesh instead of a uniform mesh, the number of nodes and finite element model is lower and consequently the number of degrees of freedom and equations to be solved by the software is also lower.

![Figure 6: Uniform mesh and progressive mesh](image)

To compare the accuracy of both types of mesh, the stress distribution at the critical sections is analyzed using a uniform mesh and a progressive mesh with sizes 2 mm and 1 mm.

The stress distribution for both types of mesh is identical. The stress values of the uniform mesh of size 2 mm are the same to the stress values of the progressive mesh of minimum size 2 mm. The same applies to uniform mesh of size 1 mm and the progressive mesh of minimum size 1 mm. Therefore it is possible to use a progressive mesh to provide the same precision in the results that a uniform mesh, but using a much lower computer time.

After checking the validity of the progressive mesh, to achieve greater accuracy in the results, a progressive mesh of minimum sizes 2 mm, 1 mm, 0.5 mm and 0.2 mm are used in the analysis of the critical sections.

3.3 FAILURE MODE A (BENDING COMBINED WITH TENSION)

According to the theory of Strength of Materials, the normal stress in the reduced section is obtained by the algebraic addition of the normal stress produced by the axial force N and bending moment M (Figure 7). Therefore, the normal stress \( \sigma_x \) is given by the expression:

\[
\sigma_x = \frac{N}{b \cdot h} + \frac{M \cdot y}{I} = \frac{10000}{50 \cdot 60} + \frac{450000 \cdot 30}{900000} = \begin{bmatrix} -1167 \text{N/mm}^2 (C) \\ 1833 \text{N/mm}^2 (T) \end{bmatrix}
\]

![Figure 7: Reduced section subjected to tensile force combined with a bending moment](image)

To compare the results obtained by FEM with theoretical values, a graph with Cartesian axes is made (Figure 8). The graph shows the stress distribution obtained by FEM along the reduced section and the theoretical stress distribution. The vertical axis represents the reduced section height in mm and the horizontal axis represents the normal stress in direction parallel to the grain \( \sigma_x \) in N/mm².

![Figure 8: Normal stress distribution in reduced section](image)

The stress distribution for different mesh sizes is similar between the heights of 10 mm and 60 mm in the reduced section. That is, the values of compression stress in the top of the section are very similar for different mesh sizes. The tensile stress values are also coincident except for the point where there is a stress concentration. At this point, smaller mesh size indicate higher stress values by FEM.
When the volume of stress is calculated for different mesh sizes, it is observed that the values obtained are lower than the theoretical value. When the mesh is refined, the accuracy increases and the values obtained are approaching to the theoretical value. However, with the smaller mesh size (0.2 mm), the volume of stress obtained is even greater than the theoretical value.

### 3.4 FAILURE MODE B (LOCAL COMPRESSION)

The normal stress $\sigma_n$ in the section of the notch (Figure 9) can be obtained using the following expression:

$$\sigma_n = \frac{N}{b \cdot t} = \frac{10000}{50 \cdot 30} = 6.67 \text{ N/mm}^2$$

*Figure 9: Section of the notch subjected to compression stress*

In the same way, a graph with Cartesian axes is made (Figure 10) where the vertical axis represents the height of the section of the notch in mm and the horizontal axis represents the contact pressure $\sigma_n$ in N/mm².

The stress distribution is perfectly symmetrical. There are two areas of stress concentration, one located at the top of the section (height = 30 mm) and other located in the bottom of it (height = 0 mm). In these areas, stress values by FEM increases with decreasing mesh size. In the rest of the section where the stress concentration is low, the stress distribution for different mesh sizes is similar and close to the theoretical value. When the volume of stress is calculated for smaller mesh sizes, it is observed that the values obtained are lower than the theoretical value. When the mesh is refined, the accuracy increases and the values obtained are approaching to the theoretical value.

### 3.5 FAILURE MODE C (SHEAR)

Assuming a uniform distribution in the entire section of the horizontal plane (Figure 11), the shear stress can be obtained by the following expression:

$$\tau_{yx} = \frac{N}{b \cdot l} = \frac{10000}{50 \cdot 90} = 2.22 \text{ N/mm}^2$$

*Figure 11: Section of the horizontal plane subjected to shear stress*

As in other critical sections, a graph with Cartesian axes is made (Figure 12) where the vertical axis represents the shear stress $\tau_{yx}$ in N/mm² and the horizontal axis represents the length of the horizontal plane in mm.

*Figure 12: Shear stress distribution in section of horizontal plane*

The stress distribution shows values that significantly increase when approaching the beginning of the heel (length = 90 mm), where stress concentration occurs. In this area, the shear stress value by FEM increases with decreasing mesh size.
4 CONCLUSIONS

Through the analysis of the stress distribution in the different sections of study for each mesh size and comparing the values obtained from the FEM and the classical theory of Strength of Materials, can be concluded:
- The results show a central symmetry of the isobar lines distribution where the centre of symmetry corresponds to the geometric centre of the joint.
- In areas where stress concentration is lower, different mesh sizes show similar stress values. In areas where stress concentration occurs, the same values increase considerably with the refinement of the mesh being necessary to refine it enough to collect the maximum stress.
- When the volume of stress is calculated for smaller mesh sizes, it is observed that the values obtained are lower than the theoretical value. When the mesh is refined, increases the accuracy and the values obtained are approaching to the theoretical value. However, an excessive reduction of the mesh size can result in volume of stress slightly higher than the theoretical value due to very high stress concentration in specific areas (Table 1).

<table>
<thead>
<tr>
<th>Mesh size</th>
<th>Failure mode A</th>
<th>Failure mode B</th>
<th>Failure mode C</th>
</tr>
</thead>
<tbody>
<tr>
<td>theoretical value</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
</tr>
<tr>
<td>progressive mesh 2 mm</td>
<td>8.717</td>
<td>7.394</td>
<td>7.427</td>
</tr>
<tr>
<td>progressive mesh 1 mm</td>
<td>8.972</td>
<td>7.812</td>
<td>8.057</td>
</tr>
<tr>
<td>progressive mesh 0,5 mm</td>
<td>9.552</td>
<td>8.541</td>
<td>8.516</td>
</tr>
<tr>
<td>progressive mesh 0,2 mm</td>
<td>10.273</td>
<td>9.183</td>
<td>8.931</td>
</tr>
</tbody>
</table>

- Taking into account the computational resources and accuracy of the results, the correct mesh size is a progressive mesh which combines a large mesh size in areas where there is not stress concentration, with a mesh size refined enough in the stress concentration areas.
- Comparison of normal stress levels obtained by the FEM and the classical theory shows small differences except at points of stress concentration.

ACKNOWLEDGEMENT

Ministry of Science and Innovation, BIA2010-18858: “Non-destructive techniques for structural classification of wood in new construction and rehabilitation”.

REFERENCES

THE RESEARCH OF PLAN MIXED HYBRID TIMBER STRUCTURE AND PARAMETRIC EQUIVALENT BRACE STUDY OF FLOOR SLAB

Mengting Tsai¹, Mikio Koshihara²

ABSTRACT: In this research, a new type of mid-rise hybrid timber structure mixed with steel core is studied, and the primary objective of this study is to understand the structural behaviour and the force distribution under seismic loading of the five storey plan-mixed hybrid timber structure based on the building code in Japan. The proposed simulation model examined how the seismic loading suffered in timber part transfer to the core part, which is constructed by steel, through the floor slab and steel connector. This paper gives an overall view of the plan-mixed hybrid timber structure, and with the confirming required knowledge of this kind of structural systems, the enhanced performance and improvement of the mid- and even high-rise hybrid timber structural system will also be expected.

KEYWORDS: Hybrid timber structure, Ai distribution, Equivalent brace, Floor stiffness

1 INTRODUCTION

Wood is a natural, recyclable and environmentally friendly material. With the improvement of the new technology nowadays, it is possible to construct a mid- or even high-rise timber building with fire and earthquake safety solutions of the constructions. In this study, a new type of mid-rise hybrid timber structure mixed with steel core is studied in order to seek the potential possibility of this kind of building and promote the construction of the high-rise timber buildings.

The primary objective of this study is to understand the structural behaviour and the force distribution under seismic loading of the five storey plan mixed hybrid timber structure, which has been constructed in a 1/3 scale model and was tested in 2006 shown in figure 1 to 3, in order to find out a proper simulation model based on the building code in Japan.

1.1 OVERALL VIEW OF THE STUDY

First of all, the simulation models are proposed. The proposed simulation model examines how the seismic loading suffered in timber part transmits to the core part, which is constructed by steel, through the timber floor slab and the steel connectors. And the comparisons and the examinations between the results obtained from experiments and that from simulation models assure the corrections of these simulation models. Furthermore, the deformation relationships between timber part and steel part at each floor while suffering a seismic loading will also be discussed and realized.

In addition, the parameter study of the different types of equivalent brace converted from different types of the timber floor slab will also be discussed in order to find out the most efficient force transferring case.

Figure 1: Five storey experimental specimen

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² Mikio Koshihara, ICUS, Institute of Industrial Science, The University of Tokyo, Ce 406, 4-6-1 Komba Meguro-ku Tokyo 153-8505, Japan. Email: tsai@iis.u-tokyo.ac.jp
2.1.1 The Ai distribution

Based on the seismic design code in Japan, the Ai distribution is one of the factors used to generate the seismic force instead of inverted triangular loading distribution that is used in North American. The equation (1) and (2) show how the Ai can be calculated,

\[ Ai = 1 + \left( \frac{1}{\sqrt{ai}} - ai \right) \times 2 \times \frac{T}{1 + 3f} \]  

\[ ai = \frac{\sum W_i}{\sum W_n} \]  

where \( i \) = storey number, \( W_n \) = weight of storey \( n \), \( ai \) = weight ratio at storey \( i \), \( T \) = primary natural period according to \( T = 0.03h \).

2.1.2 The required seismic shear force \( Q_{ei} \)

Once the storey shear coefficient \( Ai \) is determined, the seismic shear force \( Q_{ei} \) at each floor can also be calculated by the following equation (3),

\[ Q_{ei} = Ci \times Z \times Ri \times Ai \times Co \times Wi \]  

where \( Z \) = area modulus, \( Ri \) = vibration characteristic coefficient, \( Co \) = standard shear coefficient, \( Q_{ei} \) = seismic shear force at storey \( i \). In the equation (3) the standard shear coefficient \( Co \) represents the earthquake magnitude, for example \( Co = 0.2 \) can be described as the design of the building which can resist at most 200 gal earthquake. Usually, the value of \( Co \) varies from 0.2 to 1.0 depending on the demands or types of the buildings. In this study, \( Co = 0.2 \) and 0.5 is considered.

And the area modulus \( Z = 1.0 \), vibration characteristic coefficient \( Ri = 1.0 \), which means the building is located on the ground type 2. The results of the shear force \( Q_{ei} \) at each floor alone the \( Y \) axis are shown in the table 1.

<table>
<thead>
<tr>
<th>H</th>
<th>ai</th>
<th>Ai</th>
<th>( \Sigma Wi )</th>
<th>Ci (Co)</th>
<th>( Q_{ei} ) (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>0.2</td>
<td>1.38</td>
<td>4.75</td>
<td>0.28</td>
<td>0.69</td>
</tr>
<tr>
<td>5F</td>
<td>0.4</td>
<td>1.20</td>
<td>9.79</td>
<td>0.24</td>
<td>0.60</td>
</tr>
<tr>
<td>4F</td>
<td>0.6</td>
<td>1.11</td>
<td>14.77</td>
<td>0.22</td>
<td>0.56</td>
</tr>
<tr>
<td>3F</td>
<td>0.8</td>
<td>1.05</td>
<td>19.78</td>
<td>0.21</td>
<td>0.53</td>
</tr>
<tr>
<td>2F</td>
<td>1</td>
<td>1</td>
<td>24.78</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2(b): The calculation results of the \( Q_{ei} \) in Axis Y5

<table>
<thead>
<tr>
<th>H</th>
<th>ai</th>
<th>Ai</th>
<th>( \Sigma Wi )</th>
<th>Ci (Co)</th>
<th>( Q_{ei} ) (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>0.2</td>
<td>1.38</td>
<td>9.15</td>
<td>0.28</td>
<td>0.69</td>
</tr>
<tr>
<td>5F</td>
<td>0.4</td>
<td>1.20</td>
<td>18.73</td>
<td>0.24</td>
<td>0.60</td>
</tr>
<tr>
<td>4F</td>
<td>0.6</td>
<td>1.11</td>
<td>28.30</td>
<td>0.22</td>
<td>0.56</td>
</tr>
<tr>
<td>3F</td>
<td>0.8</td>
<td>1.05</td>
<td>37.88</td>
<td>0.21</td>
<td>0.53</td>
</tr>
<tr>
<td>2F</td>
<td>1</td>
<td>1</td>
<td>47.45</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Table 3(c): The calculation results of the Qei in Axis Y9

<table>
<thead>
<tr>
<th>H</th>
<th>ai</th>
<th>Ai</th>
<th>(\Sigma Wi) (KN)</th>
<th>Ci (Co)</th>
<th>Qei (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>0.2</td>
<td>1.38</td>
<td>13.48</td>
<td>0.28</td>
<td>0.69</td>
</tr>
<tr>
<td>5F</td>
<td>0.4</td>
<td>1.20</td>
<td>27.57</td>
<td>0.24</td>
<td>0.60</td>
</tr>
<tr>
<td>4F</td>
<td>0.6</td>
<td>1.11</td>
<td>41.66</td>
<td>0.22</td>
<td>0.56</td>
</tr>
<tr>
<td>3F</td>
<td>0.8</td>
<td>1.05</td>
<td>55.75</td>
<td>0.21</td>
<td>0.53</td>
</tr>
<tr>
<td>2F</td>
<td>1</td>
<td>1</td>
<td>69.84</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 4(d): The calculation results of the Qei in Axis Y13

<table>
<thead>
<tr>
<th>H</th>
<th>ai</th>
<th>Ai</th>
<th>(\Sigma Wi) (KN)</th>
<th>Ci (Co)</th>
<th>Qei (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>0.2</td>
<td>1.38</td>
<td>8.78</td>
<td>0.28</td>
<td>0.69</td>
</tr>
<tr>
<td>5F</td>
<td>0.4</td>
<td>1.20</td>
<td>18.38</td>
<td>0.24</td>
<td>0.60</td>
</tr>
<tr>
<td>4F</td>
<td>0.6</td>
<td>1.11</td>
<td>27.97</td>
<td>0.22</td>
<td>0.56</td>
</tr>
<tr>
<td>3F</td>
<td>0.8</td>
<td>1.05</td>
<td>37.56</td>
<td>0.21</td>
<td>0.53</td>
</tr>
<tr>
<td>2F</td>
<td>1</td>
<td>1</td>
<td>47.16</td>
<td>0.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

2.1.3 The transmission of the force

After the seismic shear forces Qei at each floor are determined, meanwhile the floor slabs which resist the seismic shear forces have to be converted into the equivalent braces based on the stiffness of the floor which was generated from the results of the experiment, in order to simplify the simulation model. In figure 5, the general assumption of the force transmission system is shown, indicating the seismic force in the timber part transmits in 2 paths, one is to the steel part through the steel connector and the other is to the shear wall.

Figure 6 illustrates the equivalent brace model of the floor and the transmission of the seismic shear force Qei in the plan. In this model, the transmission of the force is calculated span by span, and in the end concentrated on the steel connector which connects the timber part and steel part in the axis Y09, and the connecting joints of each span can be simulated as a pin-pin connection.

2.2 DYNAMIC SIMULATION MODEL

Structural elements and joints are modelled considering some idealizations. The idealizations and simplification make the results of the analysis different from the experimental results. In this chapter it is discussed that the basic specifications of the simulation model that have been built up in the computer analysis program MIDAS. With the confirmation of the basic specifications, the model can be used for the further analysis.

2.2.1 The simulation model

To build up a dynamic simulation model, the first step is to convert the shear wall panel and the floor slab, which are the major elements that resist seismic force, into the equivalent brace. The ways of the constructions in each axis vary, and the types of the constructions and the converted EA of the equivalent brace are calculated and shown in the table 2. The equation (4) explains how the equivalent brace can be converted,

\[
P = K_w \cdot \delta_w = EA \cdot \frac{b^2}{\sqrt{h^2 + b^2}} \cdot \delta_b
\]

where P is the seismic loading applied, Kw is the stiffness of the shear wall obtained from the experiment of the shear wall, \(\delta_w\) is the deformation of the shear wall under seismic force P, and \(\delta_b\) is the deformation of the equivalent brace converted from shear wall. Based on the equation (4), it is expected to obtain the value EA of the equivalent brace.

The other specifications, including the cross section of the timber columns and beams, the type of the steel column and beam and so on, which are also required for the build-ups of simulation model are listed in table 3. The final simulation model has been made in the computer program MIDAS, and the input of the seismic loading is alone the Y axis. The height of each floor is 1 meter and each span is 2 meters, exactly the same with the experimental specimen.
Table 2: The types of the seismic loading resist elements and the converted value of EA

<table>
<thead>
<tr>
<th>Type</th>
<th>EA</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 panels (axis X1 to X9)</td>
<td>7686(KN)</td>
</tr>
<tr>
<td>2 panels (axis Y1)</td>
<td>7686(KN)</td>
</tr>
<tr>
<td>4 panels (axis Y5)</td>
<td>15372(KN)</td>
</tr>
<tr>
<td>Floor slab</td>
<td>25538(KN)</td>
</tr>
</tbody>
</table>

Table 3: The specifications of the model

<table>
<thead>
<tr>
<th>Material</th>
<th>Size (mm)</th>
<th>Type</th>
<th>E (KN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber part</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frames</td>
<td>100x100</td>
<td>E105-F300</td>
<td>10.5</td>
</tr>
<tr>
<td>Wall (4 panels)</td>
<td>667x1000</td>
<td>Type 1</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>t=9</td>
<td>Grade 2</td>
<td></td>
</tr>
<tr>
<td>Floor</td>
<td>2000x2000</td>
<td>Type 1</td>
<td>2.55</td>
</tr>
<tr>
<td></td>
<td>t=9</td>
<td>Grade 2</td>
<td></td>
</tr>
<tr>
<td>Steel part</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frames</td>
<td>H-100*100</td>
<td>SN400</td>
<td>199.9</td>
</tr>
<tr>
<td>Floor</td>
<td>t=4.5</td>
<td>—</td>
<td>355.5</td>
</tr>
</tbody>
</table>

2.2.2 Input Waves

The input earthquake waves that have been selected are Kobe 200 and Kobe 500 in order to make it reasonable to compare the simulation results between statics one (Co=0.2 and 0.5) and dynamic one. In addition, because the simulation model is the 1/3 of the actual buildings (same as experimental specimen), the time scale of the waves has to be justified in the 1/2.5 of the original time scale for the √K/M affection, where K is the stiffness of the building and M is the mass of the building.

2.3 THE RESULTS OF THE SIMULATIONS

According to the results of the simulation model, it is understood that the maximum shear force occurs in the two side connectors (X01 and X09), which fits one of the force distribution pattern observed in the experiment. The values of the calculations show the shear force calculated from the model is approximately 0.75 times the maximum value of the experimental result, which provides designers to estimate and design the steel connector between timber part and steel part.

It is also observed that the maximum deformation occurs at the second floor in the axis Y01 according to the comparisons of the results of the simulation model (statics and dynamics) and the experiment. With the higher the floor is, the smaller the floor deformation becomes, based on the results of the comparisons show in the figure 8 (Co=0.2 and Kobe200) and figure 9 (Co=0.5 and Kobe 500). Generally speaking, the simulation results are believed to be able to predict the deformation of each floor as shown in the figure, the deformation at each floor is not exactly the same though it is considered similar and can be trusted comparing with the experimental results.
3 THE PARAMETER STUDY OF THE FLOOR STIFFNESS

From the previous study, it is understood that although the shear wall is the main element to resist the seismic force, however, the floor is the key resisting element to transmit the seismic force from the timber part to steel part through the connector as well. In this chapter, to realize how the stiffness of the floor increased influences the general behaviour and then help to reduce the deformation of the building, the parameter study of the stiffness of the floors are carried out.

3.1 THE TYPES OF THE FLOOR STIFFNESS

The types of the stiffness of the floors are shown in table 4. In this table, the stiffness of the simulation model Kf-4 is the one which the specifications are obtained from the experiment specimen. Based on the model Kf-4, the other simulation models are made with the difference of the floor stiffness. The input of seismic loading in this study is up to 1000 gal (Co=1.0) in order to find out the ultimate behaviour of the model.

Table 4: The parameter of the floor stiffness and the converted EA

<table>
<thead>
<tr>
<th>Model</th>
<th>Stiffness of the floor (KN/mm)</th>
<th>Converted EA (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kf-1</td>
<td>27</td>
<td>6 times Kf-4</td>
</tr>
<tr>
<td>Kf-2</td>
<td>18</td>
<td>4 times Kf-4</td>
</tr>
<tr>
<td>Kf-3</td>
<td>9</td>
<td>2 times Kf-4</td>
</tr>
<tr>
<td>Kf-4</td>
<td>4.5</td>
<td>Test value</td>
</tr>
<tr>
<td>Kf-5</td>
<td>2.25</td>
<td>0.5 times Kf-4</td>
</tr>
<tr>
<td>Kf-6</td>
<td>0.45</td>
<td>0.1 times Kf-4</td>
</tr>
</tbody>
</table>

Table 5: The types of the floor construction

<table>
<thead>
<tr>
<th>Model</th>
<th>Floor Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kf-5</td>
<td>Timber slab LVL with shear panels</td>
</tr>
<tr>
<td>Kf-4</td>
<td>Timber slab LVL with RC slab</td>
</tr>
<tr>
<td>Kf-3</td>
<td>Timber slab with RC slab</td>
</tr>
<tr>
<td>Kf-2</td>
<td>LVL with RC slab</td>
</tr>
</tbody>
</table>

*Reference [1]

3.2 THE EVALUATION METHOD

The evaluation method of this study is based on the damage occurrence of the shear wall in the timber frame. Figure 10 indicates the P-d relationship of 3 different frames constructed by different materials, the P-d line of the timber frame and the steel frame are obtained from the experimental study. In this figure, the P-d line of the timber frame is focused. The P-d line of the timber frame shows that the initial stiffness (elastic) is about 5.5 KN/mm, and after the deformation is larger than 1/100 rad, the damage of the shear wall occurs. It is clear that the deformation which is smaller than 1/100 rad at the shear wall can be defined as safe. Any deformation of the shear wall is larger than 1/100 rad, it is considered as damaged, which is not safe anymore.

3.3 THE EVALUATION OF THE STUDY

3.3.1 The general behaviour

The results of the analysis show that the improvement of the stiffness of the floor makes the stiffness of the building been improved. Furthermore, the deformation at each floor is also been reduced, shown in figure 11. In the other hand, it is observed that the stiffness of the floor been improved makes the deformation at the steel part increased. The reason can be explained that the timber floor is an important element to resist the seismic force by transmitting the seismic force to the steel part. Therefore when the building suffers the seismic loading, the seismic force occurs in the timber part can be transmitted through the floor slab span by span to the steel connector first and transmit to the steel part through steel connectors.

According to the analysis results, it is believed that the stiffer the timber floor is, the much efficient the seismic force transmits from timber part to steel part, and in the end the larger deformation happens in the steel part. In the figure 11, it is also noticed that the influence of the floor stiffness is much obvious in the weaker floor slab than the stronger floor slab.
3.3.2 The evaluation of the storey displacement

In order to determine how the floor slab influence the storey displacement and its ultimate limitation, the input seismic loading increases up to 1000gal (Co=1.0). The results of the storey displacement of each model with different stiffness of the floor slab are shown in the figure 12. The oblique straight line limits the storey displacement in 1/100rad at each floor, the storey displacement lies above this line means the damage occurs at the shear wall, which is not safe anymore. According to figure 12, it is observed that Kf-1 and Kf-2 are the cases that the storey displacements at each floor lies below the safety line, which means to construct the floor by LVL with RC slab or some other stiffer construction system ensure the safety of this kind of building. However, it is also discovered that the storey displacement in the second floor of the model Kf-3 is the only floor in the model lies out of safety line, caused by the larger storey shear force. And the rest of the models lie out of the safety line, including Kf-4 (build up based on the experimental data), which fits the experimental results - the damage occurrence in the shear wall.

![Figure 11: The displacement at the RF](image1)

**Figure 11: The displacement at the RF**

**Figure 12: The displacement of each floor and the evaluation of the safety (Co=1.0, 1000gal)**

4 CONCLUSIONS

In this study, the structural behaviour of the mid-rise storey plan-mixed hybrid timber structure is discussed. From the structure point of view, this kind of structure system faces 3 major issues, which are (1) the shear force occurs in the steel connector connecting timber part and steel part, (2) the larger deformation that causes the damage occurrence in the shear wall and (3) the efficiency of the seismic force transmission from timber floor to the steel part. Based on these 3 major issues, the following results are concluded.

Firstly, the results of the calculations show the shear force occurs in the steel connector calculated from the simulation model is approximately 0.75 times the maximum value of the experimental result.

Secondly, with the stiffer the timber floor slab is, the much efficient the seismic force transmits from timber part to steel part, and in the end the larger deformation happens in the steel part.

In addition, to construct the floor slab by LVL with RC slab (shown in table 5) or some other stiffer construction system secure the storey displacement of each floor less than 1/100rad (the damage occurrence point of the shear wall), making it is possible to ensure the safety of the plan-mixed hybrid timber structure.

REFERENCES


FINITE ELEMENT SIMULATION OF STEEL-TO-TIMBER JOINT IN TENSION WITH ANGLE TO GRAIN

Bo-Han Xu¹, Abdelhamid Bouchair², Mustapha Taazount²

ABSTRACT: In this study, the behaviour of dowel-type steel-to-timber joints loaded in tension, with an angle to the grain, is analyzed experimentally and numerically. Two main types of failure are observed in the experiments such as wood splitting and embedding. The experimental results are used to validate a three-dimensional (3D) non-linear finite element model. The non-linear model uses the Hill criterion to manage the plastic yielding of the wood materials. A failure criterion is used to manage the damage evolution in wood, which is based on the interaction between shear stress parallel to grain and tension stress perpendicular to grain. Besides, the models incorporate the contact and the geometric non-linearity. Also, the crack initiation and growth are modelled, using the element removal controlled by the stress checking. The comparison with experimental results shows that the numerical model is in good agreement with them. The model can predict the ultimate load, identify the location of the initial cracking and simulate crack growth.

KEYWORDS: Steel-to-timber joint, Finite element model, Failure criterion, Angle to the grain

1 INTRODUCTION

The dowel-type steel-to-timber joint is commonly used in timber structures. Generally, timber is loaded in tension or compression parallel or perpendicular to grain combined with shear [1]. However, in some truss nodes, the connections between the members have various angles between the tension or compression load and the wood grain [2] (Figure 1). Although, some experimental studies on steel-to-timber joints, with slotted-in steel plates, are available in the literature [3-6], they are focused on the joints in tension parallel to grain or perpendicular to grain. Due to the anisotropic behaviour of timber, the direction of the load for each dowel does not fully coincide with its displacement direction. Thus, it is necessary to investigate the behaviour of dowel-type steel-to-timber joints loaded in tension, with an angle to the grain. The experimental way is very time consuming and expensive. A three-dimensional (3D) finite element model is needed.

In this paper, a 3D non-linear finite element model has been developed. The model takes into account different sources of non-linearity such as the elasto-plastic behaviour of materials, the contact evolution between steel, dowels and timber and geometric analysis of large deformation. The numerical model is validated by the experimental results.

2 EXPERIMENTAL RESULTS

Three specimens of steel-to-timber joints were tested with the load applied at a 45-degree angle to grain. The configuration and geometry of the specimens are shown in Figure 2 and Table 1.
3.2 MATERIAL PROPERTIES

Steel is considered isotropic ($E = 210000$ MPa, $\nu = 0.3$). The main mechanical characteristics, such as the yield strength ($f_y$) and the ultimate strength ($f_u$), were determined from tensile tests (Table 2).

### Table 2: Steel properties used in the model (MPa)

<table>
<thead>
<tr>
<th>Steel plate</th>
<th>Steel dowel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y$</td>
<td>$309$</td>
</tr>
<tr>
<td>$f_u$</td>
<td>$434$</td>
</tr>
<tr>
<td>$f_y$</td>
<td>$366$</td>
</tr>
<tr>
<td>$f_u$</td>
<td>$543$</td>
</tr>
</tbody>
</table>

The glued-laminated timber used in the tests corresponds to the resistance class GL24h in accordance with standard “EN 1194” [8]. The material characteristics that have been used in the numerical models are given in Table 3. The directions L, R and T are the usual orthotropic directions of wood. In this study, $E_0$ and $E_90$ are considered with $E_L = E_0$ and $E_R = E_90 = E_T$.

### Table 3: Timber properties used in the model

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0$ (MPa)</td>
<td>11600</td>
</tr>
<tr>
<td>$E_{90}$ (MPa)</td>
<td>390</td>
</tr>
<tr>
<td>$G_{mean}$ (MPa)</td>
<td>750</td>
</tr>
<tr>
<td>$v_{LR}$</td>
<td>0.41</td>
</tr>
<tr>
<td>$v_{LT}$</td>
<td>0.02</td>
</tr>
</tbody>
</table>

The non-linear material model uses the Hill criterion (Equation 1) to manage the plastic yielding of the timber. A failure criterion is used to manage the damage evolution in timber, which is based on the interaction between shear stress and tension (Equation 2). Numerically, in this criterion, the progressive failure is simulated through a reduction of the elastic modulus in both parallel and perpendicular to grain directions to represent the damage evolution in timber. Thus, a kind of damage evolution in timber is considered.

$$
\sigma = + a_3 (\sigma_y - \sigma_x)^2 + 3a_4 \tau_{xy}^2 + 3a_5 \tau_{yx}^2
$$

$$
\left[ \begin{array}{c}
\frac{a_1}{f_{c,80}} - \frac{1}{f_{c,0}} \\
\frac{a_2}{f_{c,0}} \\
\frac{a_4}{f_{c,0}} = \frac{2}{3f_v} \\
\end{array} \right] / \sqrt{2}
$$

$$
\left[ \begin{array}{c}
\sigma_x \\
\sigma_y \\
\sigma_z \\
\end{array} \right] = \left[ \begin{array}{c}
\frac{a_1}{f_{c,80}} - \frac{1}{f_{c,0}} \\
\frac{a_2}{f_{c,0}} \\
\frac{a_4}{f_{c,0}} = \frac{2}{3f_v} \\
\end{array} \right] / \sqrt{2}
$$
\[
\frac{\sigma_x^2}{f_{t,0}^2} + \frac{\sigma_y^2}{f_{t,90}^2} + \frac{\sigma_z^2}{f_{t,0}^2} - \frac{\sigma_x\sigma_y}{f_{t,0}f_{t,90}} - \frac{\sigma_y\sigma_z}{f_{t,0}f_{t,90}} - \frac{\tau_{xy}^2}{f_{t,90}^2} - \frac{\tau_{yz}^2}{f_{t,0}^2} - \frac{\tau_{xz}^2}{f_{t,90}^2} = 1 \tag{2}
\]

where \(\sigma_i\) and \(\tau_{ij}\) are the stresses in timber, 
\(f_{t,0}\) and \(f_{t,90}\) are the tensile strengths parallel and perpendicular to grain, 
\(f_{c,0}\) and \(f_{c,90}\) are the compressive strengths parallel and perpendicular to grain, 
\(f_v\) is the shear strength of timber. 

In this study, timber tensile and shear strength values are considered: 
\(f_{t,0} = 19.8\) MPa, \(f_{t,90} = 0.48\) MPa and \(f_v = 3.24\) MPa. These mean values are obtained from the characteristic values using the coefficient 1.2. Considering a normal distribution, this equivalence corresponds approximately to a coefficient of variation equal to 10%. Mean values of the compressive strengths are determined by tests: 
\(f_{c,0} = 43.57\) MPa and \(f_{c,90} = 3.7\) MPa.

3.3 CONTACT DESCRIPTION

The interaction between the different members was always modeled with deformation contact elements. Contact occurs in the interface between: the glulam and the steel plate, the dowel and the glulam, the dowel and the steel plate and finally the stiff steel plates at the supports and the glulam. Contact was modeled using the direct constraint method in MSC.Marc. The method requires the definition of the “contact body” that potentially may come in contact with the other. Contact may be developed with friction based on the Coulomb criterion. The method allows no movement until the friction force is reached. After that, the movement initiates and the friction forces remain constant. The friction coefficient between fasteners and timber is set equal to 0.3. The friction coefficient between fasteners and steel is set equal to 0.001 because of the small contact zone between the two materials. The stiff steel plates used on the supports in the tests are modeled with a management of the contact conditions between the steel plates and the timber. The friction coefficient between timber and steel plate is set equal to 0.2.

4 VALIDATION AND APPLICATION OF THE NUMERICAL MODEL

The 3D finite element model is validated by comparison of the global numerical and experimental load-slip curves. After that, the numerical models are applied to justify the failure mode and failure position.
5 CONCLUSIONS

A 3D numerical model based on finite element method has been developed to understand the mechanical behaviour of the joint. The model adopts the plastic flow law based on the Hill’s criterion associated to the Hoffman’s criterion representing the evolution of damage in timber by a reduction of the elastic modulus. The comparison with experimental results shows that the numerical model is in good agreement with them. The model can predict the ultimate load, identify the location of the initial cracking and simulate crack growth.

ACKNOWLEDGEMENT

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REFERENCES

BENDING RESISTANCE OF REPAIRED COLUMNS AND STUDS OF WOODEN WALL

Kei Sawata¹, Masahiko Toda², Satoru Kanetaka³, Takashi Harada⁴, Yoshihisa Sasaki⁵, Takuro Hirai⁶

ABSTRACT: In repairing many wooden houses, one finds that the lower parts of columns often need to be replaced with new wood material. This study investigated the bending resistance of columns for conventional Japanese wooden houses and that of studs for light frame wooden houses. The column and stud specimens were repaired by four and five methods, respectively. Column specimens were prepared from sugi solid lumber with the cross-sections of 105 × 105, 120 × 120, and 150 × 150 mm². Stud specimens were composed of two sets of 204 pieces of lumber connected with CN75 nails. Initial bending stiffness and maximum bending moment of both column and stud specimens differed noticeably, depending on the repair methods.

KEYWORDS: Initial bending stiffness, maximum bending moment, End-joint

1 INTRODUCTION

Wood is affected by environmental degradation during its service life. It is well known that biological factors, particularly wood fungi and termites, pose significant risk to the integrity of timber structures. To recover the safety and serviceability of damaged structures, degraded wood members should be removed and replaced with new wood materials. Several studies on repair methods for degraded members have been reported to date, in which mortise and tenon joints [1], mechanical joints [2,3], and adhesive joints [4-6] were examined. Very often, the repair of Japanese wooden houses requires the replacement of the lower parts of columns with new wood material, because of the high probability of biological degradation. The present study focused on the bending resistance of repaired members, investigating its effects on the shear resistance of opening frames that contained the repaired members.

2 MATERIALS AND METHODS

There were two types of bending specimens: the column specimen, corresponding to column in conventional Japanese wooden houses, and the stud specimen, corresponding to stud in light frame wooden houses. The two types were repaired by four and five methods, respectively. The frame specimens were of two types: door opening frames of conventional Japanese wooden houses and light frame wooden houses.

2.1 BENDING TESTS OF COLUMN SPECIMENS

Column specimens were prepared from sugi (Cryptomeria japonica) with the respective cross-sections of 105 × 105 mm², 120 × 120 mm², and 150 × 150 mm². Average wood density was 411 kg/m³ (standard deviation, 34.9 kg/m³), and average moisture content was 16.5% (standard deviation, 4.55%). Four-point bending tests were conducted on the following specimens. (Configuration of types CB, CC, CD, and CE are shown in Fig. 1.)

Type CA: Specimens without an end joint.
Type CB: Specimens with glued-in hardwood dowels. Test members were end-jointed using four keyaki (Zelkova serrata) dowels with epoxy resin adhesive. Each dowel was 300 mm in length and 21 mm in diameter for specimens with a cross-section of 105 × 105 mm²; or 24 mm in diameter for specimens with a cross-section of 120 × 120 mm² and 150 × 150 mm².
Type CC: Specimens end-jointed with 2 bolts and 20 nails. The bolt diameter was 12 mm. Column members and 12 mm thick structural softwood plywood were...
connected with CN90. Type CC was separated into types CCa and CCb, according to the loading direction.

Type CD: Specimens having Japanese traditional tenon-mortise joint. The cotter pin of this joint was keyaki. Type CD was separated into types CDa and CDb, according to the loading direction.

Type CE: Specimens end-jointed with 2 steel clamps (C120). Clamp was 6 mm in diameter, 45 mm in driven length, and 120 mm in length. Type CE was separated into types CEa and CEb, according to the loading direction.

Bending tests were conducted on six specimens for each joint type combination. Specimens were tested by four-point bending with a bending span 18 times the specimen height ($B$), in which the distance between the loading points was $6B$ (types CA, CB, CD, and CE) or $8B$ (type CC).

![Figure 1: Longitudinal joints of repaired member for column specimens. a and b, loading direction; B, breadth of column specimen.](image)

2.2 BENDING TESTS OF STUD SPECIMENS

Stud specimens were composed of two sets of 204 pieces of SPF lumber connected with CN75 nails. The average wood density was 469 kg/m$^3$ (standard deviation, 40.3 kg/m$^3$), and the average moisture content was 7.13% (standard deviation, 0.471%). Four-point bending tests were conducted on the following specimens. (Configuration of types SB, SC, SD, SE, and SF are shown in Fig. 2.)

Type SA: Specimens without an end-joint.

Type SB: Specimens end-jointed with polyurethane resin adhesive.

Type SC: Specimens end-jointed with 24 nails. SPF lumber and 12-mm thick karamatsu ($Larix kaempferi$) plywood were connected with CN75.

Type SD: Specimens end-jointed with 24 nails. SPF lumber and 2.2 mm thick steel plates were connected with CN75.

Type SE: Specimens with both nailed joints and adhesive joints, in which polyurethane resin adhesive was used. SPF lumber and 12 mm thick karamatsu plywood were connected with CN75.

Type SF: Specimens with both nailed joints and adhesive joints, in which polyurethane resin adhesive was used. SPF lumber and 2.2 mm thick steel plates were connected with CN75.

Bending tests were conducted on six specimens for each joint type combination. Specimens were tested by four-point bending with a bending span of 1500 mm, in which the distance between the loading points was 500 mm.

![Figure 2: Longitudinal joints of repaired member for stud specimens. Arrow shows loading direction.](image)
2.3 RACKING TESTS OF CONVENTIONAL JAPANESE WOODEN WALLS

Frame specimens were 1820 mm in length and 2700 mm in height, with a door opening of 1800 mm height, as shown in Figs. 3(a) and (b). The frame specimens consisted of 105 × 105 mm² columns, a sill, and a beam of sugi solid lumber. The members were connected with T-type steel plates (CP-T).

Racking tests were conducted on conventional door opening frames or upper partial walls sheathed with 12 mm thick larch plywood. Conventional frames had a 240 × 105 mm² sugi door head. The door head and columns were connected with a 15 × 15 mm² mizunara (Quercus crispula) cotter pin. The plywood-sheathed specimens had a 45 × 105 mm² sugi door head that was connected to the columns with four CN65 nails. The plywood was connected to the frame members with CN50 nails at 150 mm spacing.

Frame specimens had columns with and without end joint (types CA, CB, CCa, CDa, and CEa). The repaired part was positioned at the center of the height of the door opening. The racking tests were conducted on three specimens for each frame type and joint type combination.

A sill of the frame specimen was connected to a steel foundation with four bolts having a 16 mm diameter. Two hold-down connections were installed at the ends of the frame. The cyclic loading test was conducted in the same manner as the racking tests of conventional Japanese wooden walls.

2.4 RACKING TESTS OF LIGHT FRAME WOODEN WALLS

Frame specimens were 1820 mm in length and 2440 mm in height with an 1800 mm height opening, as shown in Fig. 3(c). Frame specimens consisted of 204 pieces of SPF lumber.

Racking tests were conducted on upper partial walls sheathed with 9.5 mm thick structural softwood plywood. The plywood was connected to the frame members with CN50 nails at 100 mm spacing.

Frame specimens had studs with and without an end joint (types SB and SA, respectively). The repaired part was 1000 mm from the sill plate. Racking tests were conducted on two specimens for each joint type combination.

A frame specimen was connected to a steel foundation with four bolts of 16 mm diameter. Two hold-down connections were installed at the ends of the frame. The cyclic loading test was conducted in the same manner as the racking tests of conventional Japanese wooden walls.

3 RESULTS AND DISCUSSION

3.1 BENDING RESISTANCE OF COLUMN SPECIMENS

Initial bending stiffness and maximum bending moment were obtained from the moment-bending angle curves. Initial bending stiffness was defined as the line that passed through points on the curves corresponding to 10% and 40% of the maximum bending moment.

Figure 4 shows the initial bending stiffness and the maximum bending moment of column specimens. The initial bending stiffness and the maximum bending moment differed depending on the joint type and loading direction. The initial bending stiffness had a large standard deviation on some joint types. Regardless of the specimen cross section, type CB showed the highest initial bending stiffness of the repair methods and type CEb showed the lowest values. The initial bending stiffness of repaired members with a cross section of 105 × 105 mm², 120 × 120 mm², and 150 × 150 mm² on each joint type were 12%–77%, 7%–85%, and 9%–76% of...
the control specimen (type CA) on an average, respectively. The maximum bending moment had a small standard deviation for each joint type. The maximum bending moment of the repaired member with a cross section of 105 × 105 mm², 120 × 120 mm², and 150 × 150 mm² were 5%–40%, 4%–36%, and 2%–26% of the control specimen on an average, respectively. The ratio of the maximum bending moment of the repaired member to that of the control specimen decreased as the cross section of specimens increased.

3.2 BENDING RESISTANCE OF STUD SPECIMENS

Figure 5 shows the initial bending stiffness and the maximum bending moment of stud specimens. The initial bending stiffness of the repaired member on each joint type was 65%–94% of the control specimen (type SA) on an average. Regardless of the type of side member, the specimens without an adhesive joint (types SC and SD) showed the lowest values. The maximum bending moment of the repaired member was 14%–75% of the control specimen on an average. Type SB showed the lowest maximum bending moment of the repair methods. Types SE and SF were 3.7 times larger or more, than type SB, and were 1.2 and 1.4 times larger than types SC and SD, respectively.

Figure 4: Bending resistance of column specimens. A is cross-section of column specimen.

3.3 SHEAR RESISTANCE OF CONVENTIONAL JAPANESE WOODEN WALLS

Conventional frame specimens broke at the tenon of the door head member or the cotter pin of door head-column joints; however, bending deformation of the repaired column was not obvious until the racking test was completed. The columns of plywood-sheathed specimens were clearly bent after nails at plywood-to-wood joints started being pulled off.

Figure 6(a) shows the envelope load-deformation curves obtained from the racking tests of a conventional wooden wall with a 240 × 105 mm² door head. Specimens broken by a tension fracture or plug shear at the tenon of the door head member showed a low load (some of types CCa, CDa, and CEa). The load-deformation curves of specimens broken by partial compression at the cotter pin or the split at the tenon of door head were affected insignificantly by the repair methods, and the load gradually increased up to almost 1/10 rad.
TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

In the case of plywood-sheathed specimens (Fig. 6(b)), types CA and CB showed similar load-deformation curves, whose loads increased up to nearly 1/10 rad, even though one load decreased when nails were pulled from the frame member. The load of types CCa and CDa also increased after nails were pulled from frame member, but these loads showed a smaller increase than in the case of types CA and CB, because the repaired part suffered noticeable damage. Type CEa showed characteristic load-deformation curves, with the load almost constant from 1/17 rad and above. The load-deformation curves of plywood-sheathed specimens were affected by the repair methods.

3.4 SHEAR RESISTANCE OF LIGHT FRAME WOODEN WALLS

The light frame specimens were pulled off the nails, which were fastened from the upper part of the wall to the studs. One control specimen (type SA) exhibited no damage except for pulled-off nails, and the stud of the other specimen failed to bend at the part contacting the

Figure 6: Load–deformation curves of conventional Japanese wooden frames with opening.

Figure 7: Load–deformation curves of light frame wooden frames with opening.
lintel. Type SB showed that the nailed studs separated or that the adhesive part had wood failure.

Figure 7 shows the envelope load-deformation curves obtained from the racking tests of light frame wooden walls. The load of one, i.e., type SA, increased up to nearly 1/10 rad, whereas that of the other decreased from 1/22 rad and above because of the failure of the stud. In the case of type SB, load increased from 1/100 or 1/30 rad and above; however, these loads showed a smaller increase than was observed in the case of type SA. The load-deformation curves of light frame specimens were also affected by the repair methods.

4 CONCLUSIONS

Bending tests and racking tests were conducted on repaired members and door opening frames with repaired members, respectively. The obtained results can be summarized as follows:

1. The initial bending stiffness and the maximum bending moment of repaired column and stud members were noticeably affected by the repair methods.
2. The shear resistance of conventional frames, which had a 240 × 105 mm² door head, with a repaired column was affected insignificantly by the repair methods.
3. The shear resistance of conventional frames, which had upper partial walls sheathed with plywood, with a repaired column was affected insignificantly by the repair methods in small shear deformation, but was affected by the repair methods as shear deformation increased.
4. The shear resistance of light frame specimens with repaired studs was affected by the repair methods as shear deformation increased.

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REFERENCES

VIBRATION SERVICEABILITY OF BONGOSSI WOOD PEDESTRIAN BRIDGE

Hideyuki Honda

ABSTRACT: The analytical study on vibration serviceability for timber pedestrian bridges is not carried out worldwide almost. The purpose of this study is to evaluate vibration serviceability of timber pedestrian bridge based on the field dynamic test and the dynamic response analysis due to walking of pedestrian for the objected bridge. The objected bridge is Tokiwa Bridge of large scale three spans continuous girder made by bongossi wood material constructed in Kitakyushu City of Japan. The design geometry is as, bridge length: 84.4 m, maximum span length: 31.0 m and clear width: 6.0 m. Vibration serviceability of the objected bridge was finally verified from both sides of the experiment and the analysis. As the result, this study is obtained the knowledge that it is possible to evaluate vibration serviceability of the timber pedestrian bridge analytically. The study which vibration serviceability by three dimensional dynamic response analysis is evaluated will be a worldwide little example.

KEYWORDS: Bongossi wood, pedestrian bridge, field test, dynamic response analysis, vibration serviceability

1 INTRODUCTION

The investigation of vibration serviceability for pedestrian bridges is an important problem in design method or maintenance. The vibration serviceability of pedestrian bridges has been studied for many pedestrian bridges made by concrete and steel, for example Kajikawa [1], Kobri [2], Tanaka [3], Obata [4], and Yoneda [5] in Japan. However, the study on vibration serviceability for timber pedestrian bridges is not almost carried out except for Yamada [6] and Kusaka [7]. The purpose of this study is to evaluate vibration serviceability of timber pedestrian bridge based on experiment and analysis. The objected bridge is Tokiwa Bridge of large scale three spans continuous girder made by bongossi wood material constructed at 1995 in Kitakyushu City of Japan as shown in Fig.1. The design geometry is as, bridge length: 84.4 m, maximum span length: 31.0 m and clear width: 6.0 m. The dynamic field test of Tokiwa Bridge was carried out in 2007. The vibration characteristics and dynamic behaviour were investigated, and then vibration serviceability of the bridge was verified by the response velocity measured. Next, the three dimensional dynamic response analysis due to walking of pedestrian on the bridge was also carried out, and then the experimental and analytical results were compared. Furthermore, the investigation of the dynamic response characteristics by the dynamic response analysis, and then evaluation of vibration serviceability were carried out, and vibration serviceability was finally verified from both sides of the experiment and the analysis. As the result, this study is obtained the knowledge that it is possible to evaluate vibration serviceability of the timber pedestrian bridge analytically.

Figure 1: Tokiwa Bridge objected in this study

2 FIELD DYNAMIC TEST

2.1 OBJECTED BRIDGE

The general drawing of Tokiwa Bridge objected in this study is shown in Fig. 2 to Fig.4. The bridge length is 84.4 m, and the maximum span length is 31.0 m. Each main girder with the cross section of 1200 x 265 mm is 3 spans continuous girder laminated by bongossi material of 5 layers. For this lamination method, the drift pins driven in 25 cm interval are used. Fig. 5 and Fig. 6 are shown the outside and inside of joint part. The reinforced steel pale shown in Fig. 6 was newly installed in
reinforcing work at 2005. Table 1 shows the design condition of the bridge.

<table>
<thead>
<tr>
<th>Table 1: Design condition of the objected bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constructed place</td>
</tr>
<tr>
<td>Bridge class</td>
</tr>
<tr>
<td>Bridge type</td>
</tr>
<tr>
<td>Bridge length</td>
</tr>
<tr>
<td>Span length</td>
</tr>
<tr>
<td>Width</td>
</tr>
<tr>
<td>Main timber kind</td>
</tr>
</tbody>
</table>

2.2 DYNAMIC TESTING METHOD

The field dynamic test was conducted in order to investigate dynamic characteristics of Tokiwa Bridge in 2007. The dynamic test was done by the tests such as (1) ambient vibration test, (2) impact loading test by dropping of sand bag with weight 0.3 kN and (3) resonant waking and tests by pedestrians. The measured data is checked and stored in digital recorder and personal computer, and then the dynamic behaviour and characteristics of the bridge are analyzed in monitoring system of the field test.

In resonant waking and running tests, the pedestrians of 1 or 2 were walked and run in the pace equal to vertical flexural first natural frequency of 2.25 Hz of the bridge obtained by the ambient vibration and impact loading tests, and then the response velocity and acceleration were measured. This is a critical experimental method in which the bridge becomes resonant state by the external force of pedestrian with 2.25 Hz.

Fig. 7 shows the measurement points of resonant waking and running tests. The symbol of V in this figure indicates the vertical direction of the bridge. The signal of In and Out on transfer of pedestrian is measured by the load switch shown in this figure. The transfer cases of pedestrian are walking and running at edge part of upstream.

2.3 RESULTS OF DYNAMIC TEST

Fig. 8 shows an example of response velocity by one person walking test with resonant pace of 2.25Hz at edge part of upstream. The structural characteristics of three spans continuous girder has appeared clearly from this vibration response wave. In the walking test by one pedestrian, the maximum value of response velocity was 1.23 cm/s at measured point of A4. That value was 1.63 cm/s at A4 in the walking test by two pedestrians. In the running test by one pedestrian, that value was 4.25 cm/s at A4. In the walking test by one pedestrian, the r.m.s.
Figure 10: An example of response velocity in one person walking test with resonant pace of 2.25Hz at edge part of upstream.

Figure 9: Evaluation of vibration serviceability

Table 2: Dynamic characteristics of the objected bridge

<table>
<thead>
<tr>
<th>No</th>
<th>Mode types</th>
<th>Natural frequencies f (Hz)</th>
<th>Damping coefficients h</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Horizontal 1st</td>
<td>1.37</td>
<td>1.40</td>
</tr>
<tr>
<td>2</td>
<td>Vertical 1st</td>
<td>2.25</td>
<td>2.27</td>
</tr>
<tr>
<td>3</td>
<td>Vertical 2nd</td>
<td>3.10</td>
<td>2.85</td>
</tr>
<tr>
<td>4</td>
<td>Torsion 1st</td>
<td>3.71</td>
<td>3.69</td>
</tr>
</tbody>
</table>

3 DYNAMIC RESPONSE ANALYSIS

3.1 ANALYTICAL METHOD

In the three dimensional dynamic response analysis of the bridge due to walking and running of pedestrian, Fig. 11 shows the coordinate system of x, y and z axis on node displacement and node force of element member. The $u_i$ and $u_j$ in this figure are axis displacement in x direction at node i and j. The $v_i$ and $v_j$ are axis displacement in y direction. The $w_i$ and $w_j$ are axis displacement in z direction. The $\theta_i$ and $\theta_j$ are rotational angle in x axis. The $\varphi_i$ and $\varphi_j$ are rotational angle in y axis. The $\psi_i$ and $\psi_j$ are rotational angle in z axis, respectively. The $x_i$ and $x_j$ are axis force in x direction at node i and j. The $y_i$ and $y_j$ are axis force in y direction. The $z_i$ and $z_j$ are axis force in z direction. The $M_{ij}$ and $M_{ji}$ are bending moment with respect to x axis at node i and j. The $M_{ij}$ and $M_{ji}$ are bending moment with respect to y axis. The $M_{ij}$ and $M_{ji}$ are bending moment with respect to z axis, respectively.

In the dynamic response analysis method, there are modal analysis method and direct integral method. In this study, the analysis like the following was carried out using direct integral method. The equation of motion on vibration system of pedestrian and bridge is given by

$$[M]\ddot{Z} + [C]\dot{Z} + [K]Z = [F]$$  (1)

where $[M]$ = mass matrix, $[C]$ = damping matrix, $[K]$ = stiffness matrix, $[F]$ = external force vector by pedestrian. As numerical integration method, Newmark’s $\beta$ method was used to solve the equation (1) in this study. The integral time interval $\Delta t$ is 0.05 seconds. The arbitrary (root mean square) value of response velocity was 0.44 cm/s at $A_4$. That value was 0.51 cm/s at $A_4$ in the walking test by two pedestrians. In the running test of one pedestrian, that value was 1.69 cm/s at $A_4$. Fig. 9 shows an example for evaluation of vibration serviceability by maximum response velocity in the walking test by one pedestrian. Pedestrian may have vibration sense of lightly perceptible, when one person walks in the resonant state.

Fig. 10 shows an example for evaluation of vibration serviceability by maximum response velocity in the running test of one pedestrian. Pedestrian may have vibration sense of extremely hard to walk, when one person run in the resonant state.

Spectral analysis using FFT was conducted with data measured by the dynamic test. The experimental and analytical dynamic characteristics are shown in Table 2 until vibration of the 4th degree. The vibration modes and natural frequencies of the bridge were also computed by three dimensional eigenvalue analysis using MSC/NASTRAN. There is good agreement between experimental and analytical results.
constant $\gamma$ is 1/2. The parameter $\beta$ is 1/4. The convergence accuracy of acceleration is 1/1000 in $\Delta_\theta$.

The damping matrix in this study is assumed by Rayleigh damping. The parameters of $\alpha$ and $\eta$ on damping are obtained from natural frequency $f_i$ and damping coefficient $h_i$ in vertical vibrations based on experimental values as shown in Table 2. The Rayleigh damping is given by

$$[C] = \alpha [K] + \eta [M]$$

where $\alpha = (h_1f_1-h_2f_2)/(\pi(f_1^2-f_2^2))$ and $\eta = 4\pi f_2(h_2-\pi f_2\alpha)$.

The external force by pedestrian is given by

$$\{F\}_i = F(t) = \{mg + mg \xi \sin(2\pi f t)\} \delta(t)$$

Where $mg =$ body weight of pedestrian, $p_s =$ pace of pedestrian (Hz), $\xi =$ impact force ratio, $\delta(t) =$ coefficient vector done proportional distribution at both nodes of element in which pedestrian loads.

The field experiment at the objected bridge and laboratory experiment which changed body weight, pace and step size of pedestrian were conducted in order to calculate the impact force ratio $\xi$ with four load cells of plate type. There are many cases in which pedestrian generally does walking and running in 2.0 - 2.5 Hz pace. The external force of pedestrian in this study improved the method using sine wave which Kajikawa proposed [8]. As it is shown in Fig. 12, the method as half sine wave removed negative load part (load to top from under floor system) of input sine wave was used.

The partial structural models in case with joint and in case without joint were made, and the effect of joint part was investigated from static and eigenvalue analyses. As the result, there was no difference by both structural models from both sides of natural frequency and static deflection. From this fact, the structural analysis model in this study has been modelled as without joint part. Beginning of structural modelling, the structural analysis model of this bridge was made as faithful model of 3 spans continuous girder laminated in 5 layers shown in Fig.13, and then eigenvalue analysis using the subspace method was carried out in order to confirm the convergence accuracy of three dimensional structure analytical model. As the result, the vertical first vibration was 2.19 Hz, and it became a value which was approximate to the experimental value. However, it was proven that three dimensional dynamic response analysis was difficult on this structural model, because the node number is enormous. From this fact, the structural analysis model was improved again, and structural model of 3 spans continuous girder laminated in 2 layers which considered the effect of bracing member was made. This structural model is shown in Fig. 14. The eigenvalue analysis was carried out for this structural model. The results are shown in the analyzed natural frequencies of Tables 2. The vertical first vibration mode analyzed is shown in Fig. 15.

### 3.2 DYNAMIC RESPONSE ANALYSIS

The three dimensional dynamic response analysis is originally developed by FORTRAN program. This analysis was carried out as a condition equal to the actual bridge experiment as; the body weight of pedestrian = 0.7 kN, the pace of pedestrian $p_s =$ 2.25 Hz, the step size of pedestrian = 0.65 m, the impact force ratio $\xi =$ 1.5, $\alpha$ and $\eta$ which were the damping parameters on Rayleigh

![Figure 11: Coordinate system of element member](image1)

![Figure 12: Half sine wave of external force](image2)

![Figure 13: Structural girder model of 5 layers](image3)

![Figure 14: Structural girder model of 2 layers](image4)

![Figure 15: Analyzed vibration node](image5)

![Figure 16: Analyzed points and walking lane](image6)
damping were calculated from equation (3), and it was made to be \( \alpha = 4.94 \times 10^{-4} \) and \( \eta = 0.311 \). Fig.16 shows the analyzed points and the walking lane. Three dimensional dynamic response analysis of this bridge due to walking of one pedestrian was carried out using the girder model laminated by 2 layers shown in Fig. 14. The response velocity waves at each analyzed point in resonant walking are shown in Fig. 17. As well as measured wave in Fig. 8, the vibration behaviour of three spans continuous girder has appeared clearly from this analyzed wave. The maximum value of measured point and analyzed point which is close to walking lane increases from other values, when analyzed wave in Fig. 17 and measured wave in Fig. 12 are compared. It is shown that lateral load sharing is clearly small, though the simple bracing member between each main girder has been installed. The maximum value and wave sharps based on experiment and analysis agree almost.

Power spectrum density function (PSD) was also required from spectrum analysis by FFT in order to examine frequency of response wave obtained the experiment and the analysis. As the result, the experiment wave became 2.25Hz, and the analysis wave became 2.24Hz. The validity of this three dimensional dynamic response analysis can be confirmed, because both peak spectra agree.

4 VIBRATYION SERVICEABLITY

The evaluation of vibration serviceability may become less for r.m.s. value which has meaning of mean value for variation data. From this fact, vibration serviceability in this study was evaluated by maximum value of response velocity. Fig. 18 shows evaluation of vibration serviceability for the objected bridge by maximum response velocity obtained the experiment and the analysis due to resonant waking of one pedestrian at edge part of upstream. From this figure, it is proven that the measured and analyzed values are together reactions of "not perceptible for vibration" or "lightly perceptible" on vibration serviceability of this bridge. It is also proven that that analyzed and measured values agree well and that the shape of response velocities resemble. From this fact, they seem to be able to evaluate vibration serviceability of the object bridge analytically, even if dynamic testing of the existence bridge is not done, when the vibration characteristic of the object bridge has been proven.

As an example, vibration serviceability of the objected bridge was invesgated by maximum value analyzed and measured response acceleration using the vibration limit of Ontario in Canada and BS5400. The result is shown in Fig. 19. The measured and analyzed values are proven on becoming very close values, and both values also include in permission region of the Ontario code.

![Figure 17: An example of response velocity analyzed by waking of one person with resonant pace of 2.25Hz at edge part of upstream](image)

![Figure 18: vibration serviceability of this bridge](image)

![Figure 19: An example on vibration serviceability of the objected bridge by maximum acceleration values using Ontario code and BS5400.](image)
5 CONCLUSIONS

This study is objected the pedestrian bridge with large scale three spans continuous girder made by bongossi wood material, and investigated vibration serviceability of the timber pedestrian bridge based on experiment and analysis. The three dimensional dynamic response analysis of bridge due to walking and running of pedestrian was originally developed by FORTRAN program. The investigation of the dynamic response characteristics by the dynamic response analysis and evaluation of vibration serviceability were carried out, and vibration serviceability was finally verified from both sides of the experiment and the analysis for the bridge. There was the validity of this analysis, because measured and analyzed value agreed well. It seems that vibration serviceability of this bridge is the degree in which pedestrian senses of “lightly perceptible”, and is no problem especially. As the result, this study is obtained the knowledge that it is possible to evaluate vibration serviceability of the timber pedestrian bridge analytically.

REFERENCES

REDUCED EXPRESSION FOR TIMBER STRUCTURE WITH FLEXIBLE HORIZONTAL DIAPHRAGM AND SEISMIC RESPONSE EVALUATION METHOD

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ABSTRACT: This paper discusses various effects of horizontal diaphragm flexibility on dynamic properties and seismic responses of timber structure. Equations of motion of a building with multiple discretized diaphragm elements are derived by defining reduced degrees of freedom. A method to predict dynamic properties is proposed by using the reduced degrees of freedom, and example structures are used to demonstrate its advantages. Finally, Application of the method to multi-span structure more than 3-span is also demonstrated.

KEYWORDS: Rigid floor, Flexible floor, Horizontal diaphragm, Stiffness eccentricity, Timber house

1 INTRODUCTION

All the elements resisting horizontal forces should be connected with stiff and strong floor diaphragm in terms of seismic resistance. Additionally, rigid floor is necessary to analyze seismic behavior of structure using simplified model such as multi-mass model with shear spring. Therefore, seismic design codes in many countries are based on “rigid floor assumption”. However, floor diaphragm of Japanese timber structures may not be stiff enough to satisfy rigid floor assumption. The relation between floor stiffness and dynamic properties of structure has been studied through many experiments and analyses of 3-dimensional frame model [1-4]. As a result, qualitative tendency has been obtained. However, theoretical approach appears to be lacked. Therefore, a method to evaluate dynamic properties considering floor flexibility is required. In Japan, floor diaphragm usually consists of timber panels fastened to floor frame with nails. Since connections of timber structure are like pin joints, floor frame deforms keeping parallelogram. Considering above behavior of timber structure, the authors have already presented dynamics-based approach for 1-span model, which is able to consider exact distribution of inertial force and displacement mode subjected to earthquake [5]. The theory gives accurate dynamic properties using some familiar properties of structure. The objective of this research is to extend the theory to 2-span model and present reduced expression for timber structure with flexible floor diaphragm. Finally, proposed method is applied to multi-span structure more than 3-span, and its validity and advantage are demonstrated.

2 EQUATION OF MOTION

2.1 Considered Model

As shown in Figure 1, 2-span single story model with multiple discretized diaphragm elements subjected to x-directional input motion is considered. Three springs and dashpots in each direction represent frames or walls. Floor diaphragm is divided to four areas ; Area11, Area12, Are21 and Are22, respectively. If each area acts like shear panel, displacement mode of structure is defined as shown in Figure 2. Components of rigid body which does not include shear deformation of floor diaphragm are \(u_x\) and \(\theta\). These are displacement of center of mass (c.m.) in \(x\)-direction and rotation around c.m., respectively. Other components are attributed to shear deformation of floor diaphragm ; \(\gamma_{x1}\), \(\gamma_{x2}\), \(\gamma_{y1}\) and \(\gamma_{y2}\), respectively. These are selected in condition that \(\gamma_{x1} + \gamma_{x2} = \gamma_{y1} - \gamma_{y2}\) equals to zero, which means shear forces do not rotate floor diaphragm. If bilaterally symmetric structure is considered, \(\gamma_{x1} = \gamma_{x2} (= \gamma_y)\) is derived. Moreover, \(u_x\), \((\theta + \gamma_{x1})\), \((\theta + \gamma_{x2})\) and \((\theta - \gamma_y)\) are dominant parameters to express motion of structure. As a result, it is found that the structure can be
deal with 4-degree of freedom system. Besides, beams are likely to affect stiffness of floor diaphragm. For example, if \( \gamma \) does not equal to \( \gamma_2 \), beams must be bent around their weak axes. Aoki et al. (2002) have reported that bending stiffness of beams is likely to affect total stiffness of floor diaphragm especially when floor shear stiffness is low [2]. Owing to this, rotational springs are added in boundary of each area as shown in Figure 3, which represents bending of beams and outputs moment proportional to \( (\gamma_1 - \gamma_2) \).

### 2.2 Equilibrium of Force in An Area

Figure 4 shows an example of equilibrium of force in Area 11. 1) Total forces and moment of stiffness element in Area 11, 2) Inertial force, 3) Shear forces transmitted from adjacent panels and 4) Bending moment of beams are taken into account. Coordinate of stiffness element and mass element in Area \( ij \) is defined as shown in Figure 5, and the following parameters are calculated.

\[
K_{xy} = \sum_{j} k_{ij}^{(l)} , \quad K_{ij} = \sum_{j} k_{ij}^{(l)} \left[ \sum_{j} k_{ij}^{(l)} (y_j)^2 + k_{xy}^{(l)} (y_j)^2 \right],
\]

\[
e_{xy} = \frac{\sum_{j} k_{ij}^{(l)} y_j}{K_{ij}},
\]

\[
m_i = \sum_{j} m_{ij}^{(l)} , \quad I_0 = \sum_{j} m_{ij}^{(l)} \left[ k_{ij}^{(l)} (y_j)^2 + K_{ij} (y_j)^2 \right]
\]

Where, \( k_{ij}^{(l)}, k_{xy}^{(l)} \) is stiffness of \( l \)-th element in \( x \)- and \( y \)-direction, respectively. \( x_j, y_j \) = Coordinate of \( k_{ij}^{(l)} \) and \( m_{ij}^{(l)} \) = mass of \( l \)-th element. \( x_j, y_j \) = Coordinate of \( m_{ij}^{(l)} \).

Equation (1a-c) show total stiffness, rotational stiffness and stiffness eccentricity in Area \( ij \), respectively. When the model is subjected to ground acceleration \( a_x \), Equilibrium of force in Area \( ij \) are described as follows.

\[
\Delta u_1 = r_{m} \theta + \gamma_1 \]

\[
\Delta u_2 = r_{m} \theta + \gamma_2 \]

\[
\Delta u_3 = r_{m} \theta + \gamma_3 \]

\[
\Delta u_4 = r_{m} \theta + \gamma_4 \]

\[
r_{m} = \text{Radius of gyration}
\]
Where, the following parameters are added.

\[
K_{0i}^{(s)} = \sum m_i \{y_i (y_i)^T\}, \quad K_{0i}^{(c)} = \sum m_i \{y_i (y_i)^T\}^T \\
I_{0i}^{(s)} = \sum m_i \{x_i (x_i)^T\} = m_i I_{0i}^{(s)} / 12 \\
I_{0i}^{(c)} = \sum m_i \{x_i (x_i)^T\} = m_i I_{0i}^{(c)} / 12
\]

These satisfy \( K_{0i} = K_{0i}^{(s)} + K_{0i}^{(c)} \) and \( I_0 = I_{0i}^{(s)} + I_{0i}^{(c)} \). The right member of Equation (5) is satisfied only when mass is equally distributed. \( G, V_0 \) = shear modulus and volume when floor diaphragm of each area is assumed to be elastic plate. \( K_s \) = Rotational stiffness of spring located in connection between lower areas (i.e. Area 11 and Area 12) and upper areas (i.e. Area 21 and Area 22). \( Q_{si}, Q_{ci} \) = shear forces transmitted from adjacent panels, which are not shown in equilibrium of total area because these are internal forces of floor diaphragm.

2.3 Equilibrium of Force in Total Area

By considering equilibrium of force in all areas and setting origin in center of mass, equation of motion is obtained as follows.

\[
\begin{bmatrix}
  m & ml/4 & -ml/4 & 0 \\
  I_{0i}^{(s)} / 2 & 0 & 0 & 0 \\
  \text{sym.} & I_{0i}^{(s)} / 2 & 0 & 0 \\
  \text{sym.} & I_{0i}^{(c)} & 0 & 0
\end{bmatrix}
\begin{bmatrix}
  \dot{u}_s \\
  \ddot{y}_s \\
  \ddot{y}_s
\end{bmatrix}
\begin{bmatrix}
  -e_s K_{si} \\
  K_{si}^{(s)} + GV_s / 2 + K_s \\
  K_{si}^{(c)} + GV_s / 2 + K_s
\end{bmatrix}
\begin{bmatrix}
  \dot{u}_s \\
  \ddot{y}_s \\
  \ddot{y}_s
\end{bmatrix}
\begin{bmatrix}
  -e_s K_{si} \\
  K_{si}^{(s)} + GV_s / 2 + K_s \\
  K_{si}^{(c)} + GV_s / 2 + K_s
\end{bmatrix}
\begin{bmatrix}
  \dot{u}_s \\
  \ddot{y}_s \\
  \ddot{y}_s
\end{bmatrix}
\]

Where, \( m = 4m_0 \) and \( V = 4V_0 \) are used, and the following parameters of global coordinate system are also defined.

\[
\begin{bmatrix}
  u_1 \\
  l_s / 4r_m - l_s / 4r_m \\
  (1 + \alpha) / 4 \\
  \text{sym.} \ (1 - \alpha) / 2
\end{bmatrix}
\begin{bmatrix}
  \dot{u}_s \\
  \ddot{y}_s \\
  \ddot{y}_s
\end{bmatrix}
\]

\[
\begin{bmatrix}
  \dot{u}_s \\
  \ddot{y}_s \\
  \ddot{y}_s
\end{bmatrix}
\]

The right member of Equation (12d) is satisfied only when mass is equally distributed. Using above parameters, Equation (6) is converted as follows.

\[
m \ddot{u} + ku = -m \{1\} \dot{u}_g
\]

Where, \( m = 4m_0 \) and \( V = 4V_0 \) are used, and the following parameters of global coordinate system are also defined.
Building Standard Law. expressing nondimensional parameters; stiffness of floor diaphragm is evaluate beam around weak axis. However, we can not exactly considered. When deflection of the two models are the same, we can obtain \( K_{di} = 3EJ/(2l_y) \). Where, \( E = Young's \) modulus of beam, \( J = moment \) of inertia of beam around weak axis. However, we can not exactly evaluate \( J \) because timber panels are typically fastened to beams with nails. Therefore, \( E = 8 \) kN/mm and \( l_y = 3.64 \) m are assumed, and two cases of \( J = (240*120^3/12)*3 \), (240*120^3/12)*3*10 are considered. The former means that there are three usual beams \( (K_{di} = 342 \) kNm/rad), and the latter is ten times of \( K_{di} = 3420 \) kNm/rad). These are described as "low bending stiffness" and "high bending stiffness", respectively.

\[
\begin{bmatrix}
1 & -\frac{k}{K_{y}} & 0 & 0 \\
-\frac{k}{K_{y}} & 1 & 0 & 0 \\
0 & 0 & 1 & -\frac{k}{K_{y}} \\
0 & 0 & -\frac{k}{K_{y}} & 1
\end{bmatrix}
\]

Where,

\[
\bar{\tau}_{yi} = e_{yi}/R_{ei}, \quad \bar{\tau}_{yj} = e_{yj}/R_{ej}
\]

\[
\cos^{2} \psi_{x1} = K_{x1}/K_{y}, \quad \sin^{2} \psi_{x1} = K_{x1}/K_{y},
\]

\[
\cos^{2} \psi_{x2} = K_{x2}/K_{y}, \quad \sin^{2} \psi_{x2} = K_{x2}/K_{y},
\]

\[
\sin^{2} \psi_{x} = K_{x}/K_{y}, \quad \sin^{2} \psi_{x} = K_{x}/K_{y}
\]

\[
\omega_{x} = \sqrt{4GV/I}, \quad \omega_{x} = \sqrt{K_{y}/I},
\]

\[
\alpha = (I^{(x)} - I^{(y)})/I
\]

\( \alpha_{1} \) and \( \alpha_{2} \) means floor stiffness against shear and bending, respectively. Since their units are the same as circular frequency, "circular frequency" is used. In the stiffness matrix \( (14d) \), stiffness of floor diaphragm is expressed by nondimensional parameters; \( \omega_{x}/\omega_{x} \) for shear and \( \omega_{x}/\omega_{o} \) for bending, respectively.

### 3 EVALUATION OF DYNAMIC PROPERTIES

#### 3.1 Model

In this paper, we focus on the effects of not only floor flexibility but also stiffness eccentricity. As shown in Figure 6, two types of model with different stiffness balance are considered. These models have nearly the same eccentric ratio \( R_{ei} \) which is used in Japanese Building Standard Law. \( R_{ei} \) means vulnerability to torsion, and it is calculated as follows.

\[
R_{ei} = \frac{e_{i}}{\sqrt{(K_{s}/K_{o})-e_{i}}}
\]

As for design of timber houses in Japan, \( R_{ei} \) must be lower than 0.3. \( R_{ei} \) of model 1 and model 2 are about 0.3.

Another characteristic of this research is consideration of bending stiffness of floor diaphragm. As mentioned by Aoki et al. (2002), the effect should not be neglected especially in the case of quite flexible floor diaphragm [2]. In this study, bending flexibility is simulated considering ideal rotational spring, and how to determine its stiffness is explained as follows.

As shown in Figure 7, simple beam and rigid link system are considered. When deflection of the two models are the same, k, we can obtain \( K_{di} = 3EJ/(2l_y) \). Where, \( E = Young's \) modulus of beam, \( I = moment \) of inertia of beam around weak axis. However, we can not exactly evaluate \( J \) because timber panels are typically fastened to beams with nails. Therefore, \( E = 8 \) kN/mm and \( l_y = 3.64 \) m are assumed, and two cases of \( J = (240*120^3/12)*3 \), (240*120^3/12)*3*10 are considered. The former means that there are three usual beams \( (K_{di} = 342 \) kNm/rad), and the latter is ten times of \( K_{di} = 3420 \) kNm/rad). These are described as "low bending stiffness" and "high bending stiffness", respectively.

### 3.2 Comparison of Dynamic Properties

We can obtain dynamic properties such as natural circular frequencies and modal shapes by solving eigen problem of Equation (13). Examples of evaluation are shown in Figure 8. Three combinations of \( \omega_{i}/\omega_{o} \) (shear stiffness) and \( K_{o} \) (bending stiffness) are considered; \( \omega_{i}/\omega_{o} = 0.7 \) (low bending stiffness), \( \omega_{i}/\omega_{o} = 0.7 \) (high bending stiffness), and \( \omega_{i}/\omega_{o} = 0 \). \( \omega_{i} \) means natural circular frequency of \( i \)-th mode. In all cases, dominant motion of 1st. mode is translation, 2nd. is torsion, 3rd. and 4th. are floor deformation, respectively. In the case of \( \omega_{i}/\omega_{o} \) is low, modal shape of 1st. mode includes not only torsion but also shear deformation of floor and \( \omega_{i} \) is decreased. In the case of model 2, bending stiffness also affect dynamic properties of 1st. mode. In contrast, those of model 1 are not so affected by bending stiffness. While natural circular frequency of 4th. mode is quite affected by binding stiffness, it is not likely to be important in terms of seismic response.

### 4 APPLICATION TO MULTI-SPAN STRUCTURE

#### 4.1 Modification of Equation of Motion

If inner frame is eccentric, we can not use Equation (6)
and (13). Therefore, modification of model and equation of motion is necessary. As shown in Figure 9, let \( p_l \) and \( (2 - p) l \) be the length of lower/upper areas in y-direction, respectively. In the previous chapters, we assumed \( p = 1 \).

Origin is set in the position as shown in Figure 9 instead of c.m. As a result, \( u_y \) is defined as displacement in origin. Therefore, Equation (6) is modified as follows.

\[
\begin{align*}
\begin{bmatrix}
\mathbf{m} & p\mathbf{m}l_1/4 & -(2 - p)\mathbf{m}l_1/4 & 0 & 0 \\
p^2 l_3^2/2 & 0 & 0 & 0 & 0 \\
\mathbf{f}_{\text{sym}} & \mathbf{K}_{\text{sym}} & 0 & 0 & 0 \\
p^2 l_3^2/2 & 0 & 0 & 0 & 0 \\
\mathbf{f}_{\text{sym}} & \mathbf{K}_{\text{sym}} & 0 & 0 & 0
\end{bmatrix}
\begin{bmatrix}
\mathbf{u}_x \\
\mathbf{u}_y \\
\mathbf{u}_z \\
\mathbf{u}_w \\
\mathbf{u}_v
\end{bmatrix}
\end{align*}
\]

Normalization of equation of motion like Section 2.4 is omitted.

4.2 Example Structure

In this section, Equation (19) is applied to multi-span structure more than 3-span. As shown in Figure 10(a), the model of two spans in \( x \)-direction and four spans in \( y \)-direction are studied. This is modeled on the specimen of traditional timber house tested by Shimizu et al. (2010) [4]. Only mud walls are considered as stiffness element and floor stiffness is thought to be relatively low. The stiffness of these elements is identified from other tests. Natural period of the model is assumed to be 0.4 sec when the model has no stiffness eccentricity and floor diaphragm is rigid.

4.3 Verification of Adequacy

When the model is converted to reduced 2-span model, we cannot determine where to locate inner frame. Therefore, three cases \( p = 0.67, 1, 1.33 \) are tried as shown in upper part of Figure 10(b),(c),(d). By doing so, natural period and modal shape of 1st. mode are evaluated. These are compared with the result from eigen value analysis of detailed frame model (Figure 10(a)) in lower part of Figure. 10(b),(c),(d). It is found that the model of \( p = 0.67 \) shows good accuracy and its \( T_1 \) is longest in all models. As a result, we can conclude that dynamic properties of multi-span structure can be estimated by calculating those of a few reduced 2-span models. The model having longest \( T_1 \) is likely to give reasonable solution.
TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

5 CONCLUSIONS

In this paper, reduced expression for timber structure with flexible floor diaphragm is presented. The findings of this research are as follows.

1) Equations of motion of a building with multiple discretized diaphragm elements are derived by defining reduced degrees of freedom.
2) In this model, not only shear stiffness of floor diaphragm but also bending stiffness of beams are taken into account. If shear stiffness of floor diaphragm is quite low, bending stiffness may affect dynamic properties.
3) Proposed method is based on definition of 2-span structure. However, it can be applied to 3-span structure or over.

In this research, linear structure is considered. However, equivalent linearization method based on this approach is likely to be effective for non linear structure. We will show the applicability in the future.

REFERENCES


Figure 10: Estimation of modal shape and natural circular frequency of 1st. mode using eigen value analysis of reduced model

T₁ = 0.489 sec

T₁ = 0.488 sec

T₁ = 0.475 sec

T₁ = 0.431 sec
Moment Resisting Performance of Wedged and Halved Scarfed Joint, Okkake-tsugi in plastic region

Arch. KOBAYASHI Yoshihiro1, KAMACHI Ken2, INAYAMA Masahiro3

ABSTRACT: In this research, we try to estimate splitting strength and rotational stiffness of these joints in plastic region including proportion effects of moment resisting performance with our proposing rotational stiffness and bending strength formula. In our previous research, we had verified analytical model of ones on elastic region, and had proposed its design formula. 4-point bending tests had been examined. It has shown that the design formula has estimated stiffness well. Theoretical frictional splitting strength in splitting examinations accorded splitting strength on joints in 4-point bending test well. And it verified that theoretical values of frictional splitting strength was consider that estimated value of bending strength on wedged and halved scarfed joints.

KEYWORDS: Wedged and Halved Scarfed Joint, bending test, rotational stiffness, splitting strength, design formula, plastic region

1 INTRODUCTION

Structures of some western traditional architecture are truss structures. Truss structures are ideal considering the characteristics of wooden material because it reduces bending moments and concentrates to strength of tensile or compression on the beams. But the structure is easy to collapse by an unexpected earthquake.

On the other hand, structures of Japanese traditional architecture are structurally, the style incorporating a horizontal beam known as a nuki (penetrating tie beam) used in combination with pillars to reinforce the structure or a set-in wooden panel. These structures are chosen by the reason is hard to be collapsed by an unexpected earthquake. But bending moment occurs at joints.

At the previous conference we proposed the design formula on rotational stiffness of a wedged and halved scarfed joint at elastic region. This study makes proposal of design formulas of rotational stiffness and bending strength in plastic region.

Figure: 1 Wedged and Halved Scarfed Joint

Figure: 2 M-θ graph of 4 point bending test
2  THE PREVIOUS REPORT

2.1  PROGRESS OF DESTRUCTION

The lower figures show the progress of a destruction of wedged and halved scarf joint on 4 point bending test. The previous reports showed before outbreak of slitting at a butt end of joint. The present report will announce after outbreak of splitting at a butt end of joint.

① Outbreak of splitting at an end of joint  ② Splitting extension ③ Occurring collapse at an end of cogging in the tension side ④ Finally, cogging shear failure, flexural failure of joint or collapse of cogging on fibre direction.

3  THE PRESENT REPORT

3.1  ESTIMATION OF SPLITTING POSITION

As 4 point bending test, a cleavage occurs at a butt end of wedged and halved scarf joint. Consequently, an evaluation formula of rotational stiffness needs to be lead by a mechanical model considered by cleavage strength at a butt end. So following is the assumptions for making the mechanical model.

- Assumed that a distribution of compressive stress of a butt end in a direction of material height is proportionate to distance from a neutral axis until occurring a cleavage (Bernoulli-Euler hypothesis).
- Assumed that reaction forces at a butt end are only compressive force in a direction of grain and friction of a butt end.
- Assumed that a butt end above splitting line doesn’t collapse.

The following mechanical model Figure: 4 is drawn up from these assumptions.

Legend:

- $V_f$: Cleavage strength by friction force between butt ends (N)
- $H$: Material height (mm)
- $w$: Material width (mm)
- $h_c$: Distance from top of test body to cleavage (mm)
- $X$: Distance from neutral axis to cleavage (mm)
- $y_p$: Distance from bottom of test body to neutral axis (mm)
- $R$: Friction force (N)
- $\mu$: Coefficient of friction
- $N(x)$: Compression in grain direction (N)

Figure: 3 Fractures of wedged and halved scarf joint on 4 point bending test

Figure: 4 Distribution of stress near a splitting of butt end
According to Figure 4, as occurring split at X, the force which is lifting from X the upper part: \( V(X) \) by friction force at the butt end: \( \frac{1}{2} \mu N_j \) is expressed as follows.
\[
V(x) = \int_X^{H-y_p} \Delta \sigma_f \left( \frac{W}{2} - y \right) \mu dx
\]
\[
V(X) = \frac{1}{2} \Delta \sigma_f \mu \left( \frac{W}{2} - y_p \right) \left( (H - y_p)^2 - X^2 \right)
\]  
(1)

According to (2) which adopted a calculation formula of cleavage strength on a beam with notch, \([1]\) \( V_f \): cleavage strength occurring only one side with a split on between butt end can be calculated by friction force.
\[
w \sqrt{h_e / (1 - h_e / H)} \text{ is a parameter as } \beta = 0 \text{ of (2)}.
\]

\[
V_f = C_f w \sqrt{\frac{h_e}{1 - \frac{h_e}{H}}}
\]  
(2)

As occurring split at X, \( V_f \) is
\[
V_f(X) = C_f w \frac{H - y_p - X}{y_p - X} = C_f w \frac{H(H - y_p - X)}{y_p - X}
\]  
(3)

According to (1) = (3) and principle of least work, the splitting position becomes X when \( \Delta \sigma_f \) becomes smallest.
\[
\frac{1}{2} \Delta \sigma_f \left( \frac{W}{2} - y_p \right) \mu ((H - y_p)^2 - X^2)
\]
\[
= C_f w \frac{H(H - y_p - X)}{y_p - X}
\]
\[
\Delta \sigma_f (X) = \frac{2C_f w}{(W/2 - y_p) \mu} X \sqrt{\frac{H(H-y_p-X)}{(H-y_p-X)^2-x^2}} \sqrt{y_p - X}
\]  
(4)

As \( \frac{\Delta \sigma_f (X)}{dx} = 0 \), a solution of \( X \) is
\[
X = \frac{H - 4y_p \pm \sqrt{(H - 4y_p)^2 + 16H(H - y_p)}}{8}
\]

According to \( X > 0 \),
\[
X = \frac{H \left( 1 - 4y_p \frac{W}{H} \right) + \sqrt{(1 - 4y_p \frac{W}{H})^2 + 16 \left( \frac{y_p}{H} \right)}}{8}
\]  
(5)

According to Table 1, theoretical \( h_e / H \) was estimated 1/6~1/4.

### 3.2 PROPOSAL OF EVALUATION FORMULA OF ROTATIONAL STIFFNESS IN PLASTIC REGION

We made the following assumptions to lead an evaluation formula of rotational stiffness of joint by a coggging lower end reaching bearing strength after a butt end splitting.
- Ignoring that bending deformation at a joint.
- Assumed that reaction forces at a butt end are only compressive force in a direction of grain and friction of a butt end.
- Ignoring besides frictional force of a butt end, considering occurring frictional forces of sides of upper or under joint.
- Assumed that a distribution of compressive stress of a butt end in a direction of material height is proportionate (Bernoulli-Euler hypothesis).
- Assumed that a distribution of stress of a butt end is trapezoid distribution.

The following mechanical model Figure 5 is drawn up from these assumptions

![Figure 5: Assumption of stress distribution in splitting after splitting until bearing strength at the end of coggging](image-url)
3.3 LEADING EVALUATION FORMULA OF ROTATIONAL STIFFNESS OF JOINT IN PLASTIC REGION

According to Figure 5, compressive stress of centre of trapezoid can be lead assumed that distributions of stress of a butt end above splitting,

$$\Delta f_f = \frac{\Delta f_g (H - y_p + x + y_p - 2y_f)}{2}$$

According to (6), compressive resultant forces above cleavages: $N_U$ is

$$N_U = \Delta f_f \frac{(H + x + y_p - 2y_f)}{2} \times (W - 2g) \times (H - y_p - x)$$

$$= \Delta f_f \frac{(H + x + y_p - 2y_f)(W - 2g)(H - y_p - x)}{2}$$

Triangular compressive resultant forces below neutral axis at cogging: $N_D$ is

$$N_D = \frac{1}{2} \Delta f_f \times y_f \times e = \frac{\Delta f_f y_f^2 e}{2}$$

Since the resultant compression force of the lower and the upper from splitting are equal,

$$N_U = N_D$$

Depending on (9), distance from the end of tensile side to the neutral axis after splitting: $y_f$

$$y_f = \frac{(W - 2g)(H - y_p - x)}{e}$$

Then, we set up a formula of moment of resistance of a joint. At this time, occurring friction force at the joint is following formula,

Friction force of unilateral side: $V_f = \frac{1}{2} \mu N_U$

Therefore, resisting moment of joint; $M_f = "\text{Moment depending on compressive force of triangular distribution at cogging under the neutral axis}" + "\text{Moment depending on compressive force of trapezoid distribution at both butt ends above cleavage}" + "\text{Friction force of unilateral butt \times Joint length}"

$$M_f = \int y_f \Delta f_f x^2 dx + \int (W - 2g) \Delta f_f x^2 dx + \frac{\mu V_f L}{2}$$

$$= \frac{\Delta f_f e y_f^3}{3} + \frac{\Delta f_f (W - 2g) x^3 - (x + y_p - y_f)^3}{3} + \frac{\Delta f_f \mu L (W - 2g)(H + x + y_p - 2y_f)(H - y_p - x)}{4}$$

According to Figure 6:

$$\theta_f = \frac{\delta}{x} = \frac{2\Delta f_f}{x K_e} = \frac{2\Delta f_f}{K_e} = \frac{\Delta f_f}{2}$$

$$\Delta f_f = \frac{\theta_f K_e}{2}$$

3.4 LEADING EVALUATION FORMULA OF RESISTING MOMENT AT SPLITTING

According to (4) and (12), $\theta_f(X)$ relation is determined and the expression is substituted for (13). Accordingly,

$$M_f(X) = \frac{C_W L (H - y_p - X) [4e y_f^3 + 4(W - 2g)(H - y_p - X)^3 - (X + y_p - y_f)^3]}{6(W - 2g)(H - y_p - X)^3} + \frac{C_W L (H - y_p - X)[3e(W - 2g)(H + X + y_p - 2y_f)(H - y_p - X)]}{6(W - 2g)(H - y_p - X)^3}$$

3.5 LEADING EVALUATION FORMULA OF YIELD MOMENT AS REACHING BEARING STRENGTH AT COGGING BOTTOM

At first, destruction of wedged and halved scarfed joints occur small cleavages at butt ends. After growing the cleavages, the joints make compressive destruction at the end of tensile side on coggings. Finally, the joints are destroyed by shear failures of coggings, bending failures of joints, and collapse of the fibre direction at coggings.

Therefore, We were used bearing strength at the end of tensile side at cogging as occurring crushing: $F_c//_{\parallel}$ from Standard for Structural Design of Timber Structures (602.1.1)\(^3\).

$$F_c//_{\parallel} = 60.68 \cdot r_0$$

$r_0$: standard of specific gravity of timber
When Average specific gravity of Pseudotsuga menziesii in J1group is 0.5, \( F_e = 30.34 \text{(N/mm}^2) \)

When Average specific gravity of Cryptomeria japonica in J3group is 0.3, \( F_e = 18.204 \text{(N/mm}^2) \)

Therefore, as reaching yield strength at bottom end of cogging, standard bearing strength on fibre direction of timber is below,

\[
\Delta \sigma_f y_f = F_{el} \quad \Delta \sigma_f = \frac{F_{el}}{y_e} \tag{17}
\]

As cogging bottom reaching bearing strength; \( F_{el} \) the joints is intended to be reaching yield moment. Accordingly (15), Yield distortion angle: \( \theta_f \) is below,

\[
\theta_f = \frac{2F_{el}}{K_E y_f} \tag{18}
\]

(18) is substituted for (13), and leading yield moment: \( M_y \) as the bottom of cogging reaching bearing strength.

\[
M_y = \frac{F_{el} \left[ 4ey_x + 4(W-2g)(y-\frac{H}{2})^2 (y+\frac{H}{2})^2 \right]}{12y} + \frac{F_{el} \left[ 3\mu H(2g-2g)(2H+2g+y_p-2y_f)(H-y_p-\frac{H}{2}) \right]}{12y} \tag{19}
\]

3.6 LEADING EVALUATION FORMULA OF

ROTATIONAL STIFFNESS OF JOINT

AFTER REACHING BEARING STRENGTH AT COGGING BOTTOM

We made the following assumptions to lead an evaluation formula of rotational stiffness of joint after reaching bearing strength at cogging bottom.

- Ignoring that bending deformation at a joint.
- Assumed that reaction forces at a butt end are only compressive force in a direction of grain and friction of a butt end.
- Ignoring besides frictional force of a butt end, considering occurring frictional forces of sides of upper or under joint.
- Assumed that a distribution of compressive stress of a butt end in a direction of material height is proportionate (Bernoulli-Euler hypothesis).
- Assumed that a distribution of stress of a butt end is trapezoid distribution.
- Assumed that a distribution of stress of a cogging is trapezoid distribution.

The following mechanical model

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure7.png}
\caption{Figure: 7 Assumption of stress distribution after collapse at the end of cogging}
\end{figure}

According to \textbf{Figure: 7}, after reaching bearing strength at the lower end of cogging, assuming that displacement would only proceed in the plastic state of compressive stress that remains constant, according to (17), since the distance from the neutral axis of the cogging elastic range: \( y_e = F_{el}/\Delta \sigma_q \), the resultant force of the upper side of cleavage: \( N_U = \) the resultant force of the lower side of cleavage: \( N_D \), it can be obtained \( y_q \): the distance from the neutral axis tensile side edge.

\[
y_q = \frac{F_{el} e}{\Delta \sigma_q} \tag{20}
\]

The trapezoid resultant compressive force below neutral axis at the cogging: \( N_D \)

\[
N_D = \frac{\Delta \sigma_q y_q^2 e}{2} + \frac{\Delta \sigma_q y_p(y_q-y_p)e}{2} \tag{21}
\]

Assign to (21) to (20),

\[
N_D = \frac{F_{el} y_q^2 e}{2\Delta \sigma_q} + F_{el} y_p e - \frac{F_{el} y_q^2 e}{2\Delta \sigma_q} = F_{el} y_q e - \frac{F_{el} y_q^2 e}{2\Delta \sigma_q} \tag{22}
\]

According to (7) = (22), \( N_U = N_D \),

\[
2F_{el} \sigma_q + 2\Delta \sigma_q (W-2g)(y-y_p-X)q_y
= \Delta \sigma_q (H+X+y_p)(W-2g)(H-y_p-X) + \frac{F_{el} y_q}{\Delta \sigma_q} \tag{23}
\]

Then assign to (20) to \( \Delta \sigma_q = \frac{K_E \theta_q}{2} \), according to \( y_q = \frac{F_{el} e}{\Delta \sigma_q} \), the distance from the end of cogging tensile side to the neutral axis: \( y_q \) is decided after reaching bearing strength at the bottom end of the cogging when \( \theta_q \) was given.

\[
y_q = y_e + (H+X+y_p)(W-2g)(H-y_p-X) \tag{24}
\]

Therefore, resisting moment of joint;

\[
M_q = \text{"Moment depending on compressive force of trapezoid distribution at cogging under the neutral axis"} + \text{"Moment depending on compressive force of trapezoid distribution at both butt ends above cleavage" + "Friction force of unilateral butt \times Joint length"}
\]

\[
M_q = \int_0^\gamma_q \Delta \sigma_q x^2 dx + \int_\gamma_q^{\gamma_q + y_p} F_{el} e x dx + \int_{\gamma_q + y_p}^{H-y_p} (W-2g) \Delta \sigma_q x^2 dx
+ \mu N_l l \tag{25}
\]
\[ M_q = \frac{F_e y_q}{2 \Delta \sigma_e} + \frac{F_e y_q^2}{2 \Delta \sigma_e^2} + \frac{\sigma_e (W - 2g)}{6 \Delta \sigma_e} \left( H - y_q - \left( X + y_p - y_q \right)^3 \right) \]
\[ + \frac{\sigma_e \mu L (H + X + y_p - 2y_q) (W - 2g) (H - y_q - X)}{4} \]

Accordingly assign to (23), (24), to (20), after reaching bearing strength: \( F_{e/} \) at the bottom end of cogging, relational expressions of \( y_q \) and \( M_q \) were able to lead as raising plasticity by degrees.

Distance from tensile side of end at cogging as \( \theta_q \):

\[ y_q(\theta_q) = \frac{\tau_{e/}^2 x_e^2}{\tau_{e/} x_e^2 + (W - 2g)(H - y_q - X)} \]

Resistance moment as \( \theta_q \): \( M_q(\theta_q) \)

\[ M_q(\theta_q) = \frac{F_e y_q}{3 \Delta \sigma_e} + \frac{F_e y_q^2}{3 \Delta \sigma_e^2} + \frac{\sigma_e (W - 2g)}{3 \Delta \sigma_e} \left( H - y_q - \left( X + y_p - y_q \right)^3 \right) \]
\[ + \frac{\sigma_e \mu L (H + X + y_p - 2y_q) (W - 2g) (H - y_q - X)}{8} \]

Rotational stiffness of joint as \( \theta_q \): \( K_{R/}(\theta_q) \)

\[ K_{R/}(\theta_q) = \frac{M_q(\theta_q)}{\theta_q} \]

\[ = \frac{F_e y_q}{2 \theta_q} + \frac{F_e y_q^2}{2 \theta_q} + \frac{\sigma_e (W - 2g)}{3 \theta_q} \left( H - y_q - \left( X + y_p - y_q \right)^3 \right) \]
\[ + \frac{\sigma_e \mu L (H + X + y_p - 2y_q) (W - 2g) (H - y_q - X)}{8} \]

4 THE FOUR-POINT BENDING METHOD

4.1 SPECIMENS AND EXPERIMENTAL METHODOLOGY

4.1.1 Specimens
Specimens were Pseudotsuga menziesii and Cryptomeria Japonica, parameters including material height: \( H \), cogging width: \( e \) and joint length: \( L \). Specimens were a total 108 by 9 types by each 6 by 2 tree species. (Table: 2)

<table>
<thead>
<tr>
<th>Types</th>
<th>Material Width</th>
<th>Material Height</th>
<th>Cogging Width</th>
<th>Joint Length</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>DH120-e15-L300</td>
<td>120</td>
<td>15</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e15-L300</td>
<td>180</td>
<td>15</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH240-e15-L300</td>
<td>240</td>
<td>15</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH300-e15-L300</td>
<td>300</td>
<td>15</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e30-L300</td>
<td>180</td>
<td>30</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e45-L300</td>
<td>180</td>
<td>45</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e60-L300</td>
<td>180</td>
<td>60</td>
<td>300</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e15-L180</td>
<td>180</td>
<td>15</td>
<td>180</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e15-L450</td>
<td>180</td>
<td>15</td>
<td>450</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>DH180-e15-L450</td>
<td>180</td>
<td>15</td>
<td>450</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

4.1.2 Experimental methodology
The four-point bending method was performed with the span as \( \ell = 2478 \)mm and the distance from a force application point to a fulcrum point as \( a = 669 \)mm (Figure: 8).

Centre sag, compressive strain inclined to the grain and open of top and bottom at the cogging, and gap at the both side of scarf to perpendicular were measured (Figure: 9).
It was estimated that theoretical values corrected by as both of timbers’ cogging width 60mm.

And the material width 120 mm of Pseudotsuga menziesii became higher and strength became higher.

### Table: 3 Measurements’ results of moisture contents and Young’s modulus

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specimen Number</th>
<th>Moisture content (%)</th>
<th>Specific gravity</th>
<th>MOE (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material height</td>
<td>DH 180-e 15-L300-4</td>
<td>9.0</td>
<td>0.45</td>
<td>8.1</td>
</tr>
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<td>0.37</td>
<td>13.0</td>
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<tr>
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<td>DH 180-e 15-L300-5</td>
<td>19.4</td>
<td>0.49</td>
<td>13.6</td>
</tr>
<tr>
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<td>0.48</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td>DH 300-e 15-L300-6</td>
<td>19.5</td>
<td>0.44</td>
<td>12.2</td>
</tr>
<tr>
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<td>0.44</td>
<td>9.9</td>
</tr>
<tr>
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<td>DH 300-e 15-L300-4</td>
<td>19.5</td>
<td>0.45</td>
<td>10.0</td>
</tr>
<tr>
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<td>10.4</td>
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<tr>
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<td>15.5</td>
<td>0.48</td>
<td>13.2</td>
</tr>
<tr>
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<td>7.6</td>
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### 4.1.3 Specimens’ properties

After performance of the four-point bending method, the moisture content and Young’s modulus by longitudinal vibration were measured (Table: 3).

### 4.2 ANALYSIS OF EXPERIMENTAL RESULTS AND PREDICTION OF BENDING PERFORMANCE ON WEDGED AND HALVED SCARFED JOINT

#### 4.2.1 Prediction of Bending Strength and rotational stiffness

Means of 9 kinds of specimens were led by measurements of Young’s modulus and specific gravity of Table 3. According to (5)(10)(14)(15)(16)(19), we led $X, y_f, K_p, M_f, F_p, \beta$, and $M_\beta$. Table 4 shows them.

In the material width 120 mm of Pseudotsuga menziesii cases, both the highest values were as cogging width 30mm at theoretical value of rotational stiffness on elastic and plastic regions. And the material width 120 mm of Cryptomeria japonica cases, both the highest ones were as cogging width 45mm at theoretical value of rotational stiffness on elastic and plastic regions.

When theoretical values of moment of resistance of occurring splitting on joints and theoretical values of moment resistance of reaching of bearing strength at cogging bottom end were the highest values which were as both of timbers’ cogging width 60mm.

It was estimated that theoretical values corrected by MOE and specific gravity were larger material height,
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<th>Specific resistance (MPa)</th>
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4.2.2 Cleavage strength
Experimental values of cleavage strength of both species were in good agreement with theory, or were generally able to estimate the safe side. But the estimated values to the safe side were considered that the difference was safe between the cleavage strength to be upper part of load from splitting position considering friction force on butt end at 3.1 ESTIMATION OF SPLITTING POSITION and applying upward force at upper from neutral axis on butt end in case of wedged and halved scarfed joint.

The experimental values of rotational stiffness of elastico-plastic range, which varies widely within the same parameters. The cause seems to be a big impact, such as material properties and machining accuracy. However, according to these figures, seen by subtracting the effect of them, experimental and theoretical values of rotational stiffness of wedged and halved scarfed joint well corresponded.

And according to figures, Due to changes material height, cogging width and joint length in the parameters, increasing or decreasing trend of experimental and theoretical values in rotational stiffness are generally in good agreement in the elastic range.

Theoretical and experimental values of resistance moment as reaching bearing strength at bottom end of the cogging was also in good agreement with any parameters , or could be estimated on the safe side. Therefore, it is considered bending strength of wedged and halved scarfed joint can be estimated by the moment of resistance when the cogging reaches the bearing strength.

However, in another parameter by cogging width cogging width \( \geq 30 \)mm of both tree species due to shear fracture of the cogging before reaching bearing strength at bottom end of the cogging or leading to the ultimate by bending destruction the theoretical values were not able to estimate the experimental values. It seems that the experimental values were much lower than the estimated values by the stress was concentrated in the height of shear failure line from tensile side of end, accordingly assumption of Navier hypothesis didn't holding. Therefore, future evaluation the shear strength of the cogging of wedged and halved scarfed joint will be added one of the issues.

【By material height】
According to Figure: 11, theoretical values and experimental values well corresponded on relationship rotational stiffness between material height on both tree species. But the differences of experimental values of 300mm material height were big.

Theoretical and experimental mean values on power of approximations were between 2.1 ~ 2.7. This is reflected by affection of material height on second moment of area.

【By cogging width】
According to Figure: 12, Theoretical and experimental values of rotational stiffness of joint on all of cogging width were well corresponded. But experimental values of 15, 30, 45mm of cogging width on Cryptomeria Japonica were estimated safe side.

【By joint length】
According to Figure: 12, Theoretical and experimental values and experimental ones on Pseudotsuga menziesii were good corresponded. But experimental values on Cryptomeria japonica vary widely difference, especially theoretical value of 300mm and 450mm on rotational stiffness were lower than experimental ones. It was supposed that relation between shear length and bearing stiffness were not liner.
Comparing theoretical values with experimental ones on rotational stiffness and cleavage strength by joint length.

5 CONCLUSIONS
Experimental values of rotational stiffness on elasto-plastic region varied widely even in a same parameter. It seems that influence such as materials properties of material or the processing precision is big. However, if after deducting the impact of them, it became obvious that experimental and theoretical values of rotational stiffness of wedged and halved scarfed joint good correspond with.

6 ACKNOWLEDGMENT
My deepest appreciation goes to Associate prof. INAYAMA who provided helpful comments and suggestions.

REFERENCES
SEISIMIC PERFORMANCE OF TRADITIONAL WOODEN HOUSES DAMAGED IN THE 2011 PACIFIC COAST TOHOKU EARTHQUAKE

Hidemaru SHIMIZU¹, Masatoshi OGASAWARA², Seiyu HASHIMOTO³, Tatsuo OKUDA⁴, Yoshiyuki SUZUKI⁵

ABSTRACT: This paper reports findings made from damage reconnaissance and seismic performance evaluation performed by the authors after the 2011 off the Pacific Coast of Tohoku Earthquake. The reconnaissance was performed over the prefectures of Fukushima and Tochigi, which were subjected to strong shaking during the earthquake and focused on traditional wooden houses that showed limited signs of damage. The seismic performance of these well-performed wooden houses are evaluated using the “response limit strength” design method, which judges the performance based on the base shear coefficient at a story drift angle of 1/30 rad. The evaluation indicates that the wooden houses in the particular region tend to have a relatively high lateral strength and stiffness for traditional wooden houses in Japan.

KEYWORDS: The 2011 off the Pacific Coast of Tohoku Earthquake, Wooden house, Traditional structure, Seismic performance evaluation.

1 INTRODUCTION

The 2011 off the Pacific Coast of Tohoku Earthquake (Mw 9.0) [1] which struck on March 11, 2011, registered 7 in the Japanese Meteorological Agency (JMA) seismic intensity (SI) scale. The epicentre was 130 km off the Sanriku coast. The severe ground motion was followed by extremely violent and tall tsunami that damaged many townships, communities, and a very large number of buildings. The measured height of the tsunami was as tall as 40 m in some locations [2]. In such locations, the tsunami, combined with suspected liquefaction, caused severe damage to the foundations of RC buildings and steel buildings, and in some cases, overturned them. The tsunami caused extremely wide spread damage to wooden buildings. While public attention has focused on the devastating tsunami damage near the shore, the fact that many buildings were damaged by severe ground motion must not be overlooked.

This paper first describes key features of the ground motion through the response spectrum of representative recorded motions. Subsequently, findings from a field reconnaissance or a damage assessment are described and evaluated using the “response limit strength” design method, which judges the performance based on the base shear coefficient at a story drift angle of 1/30 rad. The evaluation indicates that the wooden houses in the particular region tend to have a relatively high lateral strength and stiffness for traditional wooden houses in Japan.
survey on damaged wooden houses and assessment of their seismic performance will be discussed. The focus of this paper is ground motion effects and not tsunami effects.

2 STRONG GROUND MOTION

Ground motions from the 2011 off the Pacific Coast of Tohoku Earthquake were recorded over the dense observation network of the National Research Institute for Earth Science and Disaster Prevention (NIED). At some recording stations, the larger of the two horizontal records registered 7 in the Japanese intensity scale. A value of 6-minus in the JMA-SI scale was recorded at 51 recording stations. The maximum acceleration recorded among all stations was 2.765 Gal. The Sa-Sd spectrum of this station, Tsukidate station in Miyagi Prefecture, is shown in Figure 1 [3, 4]. The most important feature of this Sa-Sd curve is the dominance of response with period below 0.5 second. The response is rather small in the 1 to 2 second range, which is generally understood as the range that causes serious damage to Japanese wooden buildings.

Fig. 1 also shows the seismic design spectrum of Japan, accounting for amplification in the sediment layer, and standard Soil type 2. Many NIED stations recorded motions similar to the Tsukidate record where the short period element was dominant. Therefore, the spectra indicated that the ground motion, though extremely severe, was not particularly damaging to buildings.

3 DAMAGE TO WOODEN HOUSES

The authors performed damage reconnaissance over the inland part of the Tohoku region (Fig. 2). An inventory survey of damaged wooden houses was conducted in the Tochigi and Fukushima Prefecture District from April to May 2011. Fig. 3 shows a typical residence examined during this reconnaissance. Some of these residences had their foundation infested by termite.

3.1 GENERAL OBSERVATIONS

Fig. 3a is a photograph of the OD residence in the Fukushima Prefecture. The main columns have a cross-sectional dimension of 110 by 120 mm, and are spaced in 910 mm spans (the basis floor-span dimension for traditional construction). The walls are mud walls 60 mm in thickness. There are many full walls upstairs but fewer downstairs. In some locations, there are no full walls or columns downstairs directly beneath the full walls upstairs. Judging from the proximity of this house to a recording station that measured JMA-SI scale of 6-minus, the house was subjected to a very severe motion. Damage due to termite was found as shown in Fig. 3b.
Fig. 4a shows an exterior view of the AR residence, which represents a typical wooden house that was investigated in Tochigi Prefecture. This house is registered as a cultural and heritage property and was constructed 300 years ago [5]. The walls of the AR residence are 80-mm thick mud walls. While all wood structures that were severely damaged by the earthquake were from the old era, the AR residence was an exception showing that some very old wood structures survived the earthquake with little damage. An NIED recording station about 5 km from this house measured JMA-SI scale of 5-plus.

### 3.2 EARTHQUAKE DAMAGE OF WOODEN HOUSES

The AR residence evaluated as lightly damaged. Interestingly, although the columns were not connected to the foundation as shown in Fig. 4c, no evidence of slipping was detected at this location.

Fig. 6b shows cracking in a mud wall in the OD residence. Because of the damage in framing elements and the mud wall, the OD residence was evaluated as moderately damaged. Fig. 6c shows an example of framing damage where the lower lintel is pulled out from the column.

### 3.3 METHOD OF ASSESSING SEISMIC PERFORMANCE

In Japan, the seismic performance of wooden houses is assessed by a lateral load-bearing capacity calculation method [5]. The base shear coefficient \( C_B \) was calculated by dividing the lateral load bearing capacity of the story at a specified drift angle of 1/30 rad by the weight of all stories above that story. The seismic performance of houses sampled during the reconnaissance was assessed by this method [5]. In this assessment, the bearing capacity of each structural plane was calculated by summing the load-deformation angle relationships in each of the two orthogonal loading directions on the assumption that the floors are rigid floor. Fig. 7 illustrates the concept of the evaluation procedure. Following the procedure in reference [4], the load versus deformation curve for each seismic load bearing element was summed within the floor. The calculation method was applied to the first story.

The summation rules are explained as follows: The bearing capacity of full walls, such as mud walls, is calculated by dividing the column span by the reference column span (1,820 mm) and multiplying the quotient by the reference bearing capacity. Further correction is done with respect to the difference of the thickness from the reference thickness (60 mm) but no correction is done for the height. The bearing capacities of gypsum board and structural plywood...
are assumed to be constant regardless of the material type, fastener type, and column spacing. The bearing capacities of upper and lower partial-height walls are calculated by multiplying a standard strength by the number of column spans irrespective of the wall material. The bearing capacities of penetrating tie beams and tendon joints in the timber framing are corrected by dividing the reference height (2,730 mm) by the column height. Bearing capacity of penetrating tie beams was taken as that of the horizontal tie beams, because they tend to provide a lower bound value. The seismic mass of each floor is determined as the sum of the dead and live loads. The dead load is calculated based on the value per unit area stipulated in Article 84 of the enforcement ordinance of the Building Standards Law. As the wall thickness of the residences is of a standard size ranging from 60 to 80 mm, the mass of the mud walls were taken equal to that for “lathed walls of traditional wooden houses,” 830 N/m. The live load was taken equal to a value of 600 N/m² specified in Article 85 of the ordinance.

### 3.4 RELATIONSHIP BETWEEN SEISMIC PERFORMANCE AND OBSERVED DAMAGE

Fig. 8 illustrates the result of seismic performance evaluation performed for the two residences. The primary lateral load resisting elements are categorized in 3 groups: full walls, upper/lower partial-height walls, and timber framing. The $C_B$ values are 0.52 and 0.36 in the X and Y directions, respectively, for the AR residence. The contribution of full walls to the $C_B$ is 30%. However, the partial-height walls also serve as important load bearing elements along with the full walls, as their contributions to the $C_B$ is 50%. When all contributions were summed, the base shear coefficient of the AR residence was not low. This result correlates with the observation that this residence was not severely damaged by the severe ground motion. For the OD House, the $C_B$ values are 0.43 and 0.33 in the X and Y directions, respectively. As in the AR residence, the partial-height walls serve as important load bearing elements along with full walls, as their contributions to the $C_B$ is about 40%. OD House, because the larger ground motion was entered, there was no higher seismic performance, is considered a lot of damage.

### 4 CONCLUSIONS

A field survey was conducted on the performance of wooden houses within the area subjected to strong ground motion during the 2011 off the Pacific Coast of Tohoku Earthquake. Traditional wooden houses with surprisingly little damage was discussed. The seismic performance of these traditional wooden houses are evaluated based on the response limit strength design method. The upper and lower partial walls were found to serve as important lateral load bearing elements along with full walls.

### ACKNOWLEDGEMENT

The authors express their gratitude to NIED for supplying the strong motion data.

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TIMBER-Glass COMPOSITE TRUSSES AND PLATES

Wolfgang Winter¹, Werner Hochhauser², Alireza Fadai³

ABSTRACT: This paper describes glued and embedded timber-glass composite (TGC) trusses as well as TGC plates and contributes to the calculation and sizing of these new building elements. The materials glass and timber are bonded by acrylate adhesive glue lines and in case of trusses also by block settings made of epoxide resin. Amongst others, following topics are dealt with: constructions, testing and results, calculation and sizing methods.

KEYWORDS: Timber, glass, adhesives, composites, trusses, plates

1 INTRODUCTION

Wood as a sustainable and an ecological material has an excellent chance to be an alternative construction material in urban areas. So from 1996-2009 the world production capacity of glue laminated products increased from 1 Mio. m³ to nearly 5 Mio. m³. More than 50 % of the world production comes from Germany and Austria. The building codes are in constant revision, actually nearly all European countries allow at least 4 storeys buildings in timber as regular solutions and pilot projects had been built up to 10 storeys. This development forces timber to deal with other construction materials and especially with glass. The unique combination “timber-glass” gives designers and constructors also new opportunities for novel load bearing systems. Timber-glass composite elements also offer excellent adhesive properties.

The European countries have to face profound changes in the demand for buildings. The technical possibilities have multiplied, new materials and new construction methods appeared and finally a set of completely new requirements concerning environmental aspects and criteria of sustainability has to be translated into the built reality. It is well known that the presence of natural sunlight improves the health of the persons living or working in buildings. Therefore, the possibility to increase the glass surface in buildings through load bearing composite elements made of timber and glass are of high interest.

Glass has a high compressive strength but it is a brittle material with “low” tensile strength. Compared to timber the tensile strength of float glass is approximately 45 N/mm² (spruce 14 N/mm²) and the theoretical compressive strength is 700 - 900 N/mm² (spruce 21 N/mm²). Therefore the glass panels must be connected with the building, so that stress peaks are avoided and a uniform transmission of load from the load bearing structure into the glass and vice versa is guaranteed.

Since 1999 the bonding of timber-glass-composites is a topic of research [1 - 5]. The idea was primarily the use of glass in structural timber constructions as a load-bearing and stiffening element. The insufficient knowledge of the requirements for the adhesives represented the major challenge in the evaluation of the adhesives for the bonding of glass with timber. Nowadays the requirements are well known thanks to several research projects and implementations (Fig. 1 and Fig. 2). Based on these experiences new modified adhesive systems have been developed in the last years.

However modern architecture demands even more: transparent building elements, not only in facades but also as interior elements, which can be used as beams or ceilings. Glued timber and glass offer another several advantages for hybrid building elements such as similar coefficients of thermal expansion. Therefore the Department for Structural Design and Timber Engineering (ITI), Institute of Architectural Sciences, Vienna University of Technology, studied the load bearing capacity of these construction components and developed simple calculation and sizing methods [6, 7].

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TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

As far as beams and trusses are concerned, the ITI pursued the basic principle of avoiding tensile loads in the glass panes. Previous research projects, e.g. [2], dealt with timber-glass composite I-beams using glass as web and therefore load the glass elements with tensile forces, which led to tensile cracks at the lower edge of the glass elements (Fig. 3). Thereby the lower wooden flange just served as reinforcement of the broken glass web.

In order to avoid high tensile loads in the glass and to use the lower chord not only as reinforcement, the ITI cut the continuous glass web into six glass pieces and connected these with the wooden elements via adhesives using the glass elements as shear areas (Fig. 4) and compression diagonals (Fig. 5).

Within the completed research project [6] also timber-glass composite plates have been developed and tested; calculation and sizing concepts have been derived. Thereby the ITI followed the principle of ribbed plates. In previous research projects, e.g. in [1], the main focus of research has been put on two-web tee beams (Fig. 6). These constructions lead to high bending stress in the glass elements and therefore to tensile failure on the underside of the glass plate.

In order to prevent high bending stress of the glass element, the ITI designed TGC plates with an additional wooden middle rib, which should transfer the main tensile force. The main focus within testing ribbed plates made of timber, glass and adhesives was put on the
determination of the effective width of the glass element, in order to be able to use the well-known calculation concepts “shear analogy method” and “γ-method” [14].

2 PRINCIPLES AND CONSTRUCTIONS

2.1 TGC TRUSSES

The cross section of the tested trusses is shown in Figure 7. The trusses have been named „Viennese box-type trusses” referring to box-type windows, due to their similar cross sections. A Vierendeel girder made of Kerto-Q [17] is used as substructure, which is planked with TGC panes ([6, 7]) on both sides. The panes consist of laminated veneer lumber and heat-strengthened glass connected via acrylates (shown blue in Fig. 7) and epoxide resin (shown red in Fig. 7). The acrylate adhesive glue lines mainly transfer shear forces between glass and timber, the block settings (made of epoxide resin) only transfer compressive forces.

For a reliable calculation of TGC trusses and therefore panes with superimposed load bearing mechanisms the question of the representation of the load distribution (shear area and compression diagonal) is relevant. The procedure for calculating the load distribution by means of analytical simplifications is described in [6] and [10].

The trusses combine two main load bearing mechanisms: lattice girder and shear panels.

Within the research project [6] three tests on trusses, with a span of 827.2 cm (shown in Figures 8 and 9), just glued circumferentially with acrylate adhesive glue lines (shear analogy; Fig. 4) and two further comparative tests on trusses, one with glue lines and block settings (Fig. 4 and 5) and one just using block settings (Fig. 5), have been carried out. However, the main focus was on studying the load bearing mechanism of shear panels.

The tests were carried out following the test standard DIN EN 595 [15], keeping the recommended load levels but reducing the duration of loading (Fig. 10).

2.2 TGC PLATES

The plan and front view as well as the bonding detail of TGC plates are shown in Figure 11.

The plates were made of float glass with a thickness of 8 mm and glued-laminated timber (4 x 10 cm). Both materials were bonded via acrylate adhesive glue lines (3 x 30 mm, cf. Fig. 11). Figures 12 and 13 show one of the plate specimens while testing.

![Figure 9: TGC truss specimen while testing [6, 7]](image_url)

![Figure 10: Test cycle for the testing of TGC trusses (Zeit = time) [6, 19]](image_url)

![Figure 11: Engineering drawing of the TGC plates [18]](image_url)
The tests were carried out following the test standard OENORM EN 408 [16].

Based on the measured deformation it has been possible to show the influence of both bending and shear deformation (Fig. 15).

These combined deflections caused a summarized calculation of deformations according to equation (1):

$$w_x = w_x,\text{BL} + \frac{1}{K_T} \sum_{i=1}^{n} Q_i$$

where $w_x = $ summarized deformation at any point, $w_x,\text{BL} = $ bending deformation corresponding to the bending line, $K_T = $ simplified spring stiffness of a shear panel according to equation (2), which is based on [11], and $Q_i = $ transverse force of the related shear panel.

$$K_T = \left[ \frac{2}{k \cdot h} \left( \frac{1}{1 + \frac{l}{3 \cdot h}} \right)^2 \right]$$

where $k = $ equivalent spring stiffness of a shear panel according to [6], $h = $ height of a glass pane and $l = $ length of a glass pane.

3.2 TGC PLATES

Five plate specimens have been tested in three point bending tests. They showed an average collapse load of 26.46 kN at a deflection of 33.4 mm (8.83 kN at a deflection of $L/300 = 8.33$ mm; cf. Fig. 16). For the calculation two different methods have been tested: the shear analogy method described in [12] as well as the $\gamma$-method [14]. By reason of the $\gamma$-method’s simplicity it finally has been chosen as basic calculation concept. The $\gamma$-method necessitates the identification of the effective
width of the glass plate. Within the testing of plates Figure 17 has been developed. It shows the back-calculated edge stresses of four glass panes as well as their mean value.

**Figure 16:** Theoretical and practical load deflection diagram of the five tested TGC plates in the middle of the plates [6, 18] (Kraft-Weg = Force-Deflection, Vergleich Platten – Bemessung = Comparison between plates (practically) and calculation respectively sizing, 3,0-fache Sicherheit = Safety factor 3.0, Bruch = Collapse, Mittelwert = Mean value, Gamma = Theoretical deflection according to the \( \gamma \)-method, Schubanalogie = Theoretical deflection according to the shear analogy method, ohne Verbund = Theoretical deflection without compound effects / without acrylate adhesive glue lines)

**Figure 17:** Calculated stress in the middle of the plate at a total load of 8.5 [kN]: (strains have been measured with 10 strain gauges, arranged on the upper side of the glass pane in an equal distance to each other), (Mittelwert = mean value) [6, 18]

Based on Figure 17 it has been able to derive an effective width according to equation (3):

\[
b_e = \frac{l_0}{10}
\]

where \( b_e \) = effective width and \( l_0 \) = span respectively the distance between two zero points of moments

4 CONCLUSIONS

Timber-glass hybrid elements meet modern architecture’s demands and can help increase the use of timber as building material as well as the statically use of glass panes and plates.

The project results also benefit several levels of the European industry. Amongst others, it provides a marketable component system for buildings that optimally uses timber and glass. By means of the use of organic renewable resources the innovative technology provides an alternative to the existing aluminium glass tradition. The results are especially beneficial to wood-rich countries in Europe having wood production chains as well as an aluminium glass tradition. Combining glass with the ductile material timber improves the structural performance after failure and expands the scope of applications beyond the initial limits. Furthermore, those composite elements enable a more efficient functionality of structural glass elements by allowing the use of approved timber joining techniques. The results provide a marketable component system for buildings that optimally uses timber and glass. Timber-glass components as an ecological and sustainable building material have a lot of potential to be discovered.

These products also open new markets to the wood industry with a potential part of 10 to 20 % of the European construction market. Wood-based alternatives to conventional steel construction also open opportunities to European countries to reduce their carbon emissions. Furthermore, the European timber construction industry will have the opportunity to export this new composite technology worldwide.

Within the practice oriented project the ITI and its industry partners HAAS-FERTIGBAU, KAPO, OTTO-CHEMIE and SIKA managed to develop simple calculation and sizing methods. Furthermore a guideline for ability testing of adhesives has been developed. The results of the research project have been published in a user-friendly final research report [6].

5 OUTLOOK

Based on the results of the project [6] the ITI at the Vienna University of Technology, as coordinator, managed to acquire the international follow-up research project “Load bearing timber-glass composites - LBTGC” with other 26 partners from Austria Sweden, Slovenia, Germany, Turkey, Brazil and Chile within the framework WoodWisdom-Net.

The consortium thereby set the goal of studying new application possibilities as well as the long term behaviour of these new hybrid building elements to derive adequate safety- and modification factors, which nowadays are chosen relatively conservative with a total
safety factor of \( \gamma_{\text{tot}} = 60 \) for adhesives [13] analogously according to formula (4).

\[
\gamma_{\text{tot}} = \frac{k_{\text{mod,long}}}{\gamma_M} = \frac{0.1}{6} = \frac{1}{60}
\]

where \( \gamma_{\text{tot}} = \) Total safety factor, \( k_{\text{mod,long}} = \) Modification factor for long-term behaviour of adhesives (mechanical fatigue and creep behaviour; [6] and [13] and \( \gamma_M = \) Material safety factor

The main challenges remaining and being addressed in the WoodWisdom project LBTGC, relate to the optimisation of timber-glass composite elements for load bearing structures not alone in terms of their response to long term loading but also in terms of seismic events. A ductile connection between the timber frame and a glass panel shall present a key feature in the seismic wall design. Apart from that, also the studies on long-term behaviour of TGC trusses and TGC plates will be deepened and the reliability of such structures in practical applications will be proved.

Nevertheless the unique combination of wood products with glass components will also request deeper investigations regarding the interfaces, the compatibility, and the collaboration between the different components and materials. The final issue consists in optimising a composite system by using the best characteristics of each material in order to develop and advance a high competitive composite building system.

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REFERENCES

ABSTRACT: The purpose of this paper is to present a brief listing of recommendations for design and construction with structural roundwood for wood coming from reforestation projects in Brazil. For theoretical and experimental studies were conducted with structural elements and their connecting joints using timber roundwood, which was classified and characterized both visually and mechanically. Examples are given of a range of projects and constructions, such as electrification poles, sheds, retaining walls, sound barriers, other barriers, bridge beams, timber mixed bridges, bridges with cable stays, residential housing with beam columns and porticos, residential log home, hyperbolic parabolic roofing, observation towers and others with structural systems and constructed from roundwood originating from reforestation projects (eucalyptus and pine trees) built in Brazil and in other countries. The study has the objective of publicizing the possible techniques and alternatives in the field of structures and construction using roundwood from reforested wood, providing savings and benefiting the environment in a sustainable manner, and promoting the regeneration cycle of forests. The results were compiled for the textbook in Portuguese entitled "Manual of design and construction of structures with roundwood from reforestation", published in Brazil in June 2010.

KEYWORDS: sustainable buildings; structural systems; roundwood treated; connections between structural elements; reforestation.

1 INTRODUCTION

The use of timber structural elements of reforestation in Brazil has grown over the past few years because of research carried out in order to make it more competitive with a material over other materials used in structural function. As a building material, wood reforestation is abundant, versatile and easily obtained, BRITO (2010)[3]. If technologically manipulated and protected from natural disasters caused by fire, erosion, insects and diseases, the native forest and reforested areas will ensure living conditions for future generations. At present there is the need to develop studies to find alternative materials economically viable and meet the requirements of sustainable construction. The possible structural systems using roundwood coming from reforested wood in constructions appear as options in this great challenge to reconcile aspects of sustainability social, economic and environmental. In this paper, we discuss some of the main topics of the textbook that was published in Portuguese Entitled "Manual of design and construction of structures with roundwood from reforestation", indicating recommendations for design and construction with this noble material. In the manual are presented the main characteristics of roundwood, the main types of classifications and characterizations of the physical properties of strength and elasticity of wood species for reforestation roundwood (eucalyptus and pine) and the classes of use for preservation treatment of wood to ensure increased lifetime of the structures. They also suggested several types of connections between structural elements and some examples of common structural systems for building structures with roundwood. It is also presented the criteria for design of structural elements with roundwood, according to ABNT NBR 7190:1997 [1]. In Chapter 8 of the original manual 124 data sheets were listed, with various types of design and construction of structural systems, constructive roundwood, built in Brazil and abroad, BRITO (2010) [2].

2 CONSIDERATIONS FOR DESIGN

As the design of structures with plump pieces of wood from reforestation in Brazil, is relatively new, there is still little research literature related to the subject. Thus, it becomes essential to create the tables manually with the characterization of various diameters and various species of wood reforestation for roundwood, effectively considering the section of the circular pieces, and that may be attached to a future revision of the NBR 7190.
2.1 CHAPTER 1: KEY FEATURES

The structures designed with parts roundwood from reforestation have great advantages compared with the pieces of lumber, correlating, economic and environmental sustainability.

In the process of industrialization of parts roundwood from reforestation, there is a large cost reduction, because it requires less investment in machinery and equipment, causing reduction in manpower, lower power consumption and less waste of natural resources and raw materials. During the process of cutting the pieces of sawn wood waste are generated the order of 60% to 70% of the original number to ensure the flatness of the parts and consequently the structural parts present transverse dimensions smaller, reducing the resistance of the part, as shown in Figure 1, CALIL & BRITO (2010)[4].

Another major advantage is the fact that these woods are generally from reforested trees, preserving native forests. Despite the eucalyptus plantation is a monoculture, this presents a great environmental benefit. An important positive factor is the great potential for sequestering carbon dioxide (CO₂). Young trees of high biomass production and short cycle require large amounts of CO₂ to promote photosynthesis. The potential for CO₂ sequestration is considered by most researchers a major if not the main criterion in assessing the eco-environmental benefits of a plant. The big eaters of CO₂ are the trees in the growth phase. The higher your turnover is more efficient the process. The high biomass production and high rotation transform eucalyptus in a large potential for sequestration of CO₂, the main cause of global warming.

Therefore, the main disadvantages of roundwood are related to geometric features, ensuring the acquisition of parts straight, with little dimensional variability due to the influence of the taper.

2.2 CHAPTER 2: STRUCTURAL CLASSIFICATION

In Chapter 2 of this is the manual sorting and structural characterization. The plump pieces of wood are classified by two main criteria: visual classification and mechanical sorting. CALIL & BRITO (2010) [4].

The visual rating is based on the premise that the mechanical properties of a piece of wood of different mechanical properties of wood free of defects due to the growth characteristics and these characteristics can be viewed and judged by the human eye. With the help of classification rules, the growth characteristics are used to select the timber in quality classes.

The main peculiarities for visual classification as described in the NBR 8456:1984 for accepting posts preserved must have standard features and be finished. As the item "Defects unacceptable" the NBR 8456:1984, the posts must be free from signs of decay, especially the heart, damage to the sapwood resulting from cutting or transportation; transverse fractures; marked depressions. In the item "Defects acceptable," believes that certain defects are acceptable, but with limited range, such as curvature, sinuosity by any stretch; cracks on top, body and base; slits in the top and bottom and maximum depth of 5 cm; us or holes existing nodes anywhere along the 30 cm bent shaft or helical.

Since the main classification mechanical tests for structural elements with cylindrical trunks, are: the static test figure 1 and figure 2, and testing the technique of transverse vibration figure 3 and figure 4.
2.3 CHAPTER 3: DURABILITY AND TREATMENT OF WOOD

In chapter 3 of the manual we comment on the durability and wood treatment, indicating the main causes of deterioration of wood, the criteria for determining the usage class depending on the type of biological risk that the timber will be submitted in order to indicate the choice of treatment method and condom product, wood species for reforestation.

Durability of the wood is the property of withstanding a greater or lesser extent to the attack of destructive agents under natural condition of use, as described in NBR 8456:1984.

A number of environmental agents have the potential to reduce the performance of the timber over time. The designer, however, can ensure the durability using a combination of three factors, CALIL & BRITO (2010)[4]:

- Better design detailing, for the purpose of greater efficiency, which are considered the criteria for protection against rain and sunlight, rapid drainage of water.
- Preservative treatment through chemical preservation of the product condom impregnation pressure in the autoclave and surface treatment.
- Inspection, maintenance and repairs.

The use system class offers a simplified tool for making decisions about the rational and intelligent use of wood in construction, providing a systematic approach to product and user to ensure durability of buildings. The system is the establishment of six classes of use based on the conditions of exposure or use of wood in anticipation of component performance and possible bio deteriorative agents present, Table 1. This system leads to a reflection on what measures should be taken during preparation of a construction project and assists in defining the preservative treatment of timber (product and process) depending on the condition of use that it will be exposed.

Table 1: Classes of Use, terms of wood in construction.

<table>
<thead>
<tr>
<th>Class of Use</th>
<th>Conditions of Use</th>
<th>Xilófago Organism</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Inside buildings, out of contact with soil, foundations or masonry, protected from the weather, internal sources of moisture. Local free-access underground or arboreal termites.</td>
<td>Of dry wood termites Insect borers of wood</td>
</tr>
<tr>
<td>2</td>
<td>Inside buildings, in contact with masonry, without contact with the ground or foundations, protected from the weather and internal sources of moisture.</td>
<td>Of dry wood termites Subterranean termites Arboral</td>
</tr>
<tr>
<td>3</td>
<td>Inside buildings, out of contact with the ground and continuously protected from the weather, which may occasionally be exposed to sources of moisture.</td>
<td>Of dry wood termites Subterranean termites Arboral Fungi mold/staining/decay</td>
</tr>
<tr>
<td>4</td>
<td>Outdoor use, out of contact with the ground and subject to weather.</td>
<td>Of dry wood termites Subterranean termites Arboral Fungi mold/staining/decay</td>
</tr>
<tr>
<td>5</td>
<td>Contact with soil, freshwater and other situations favorable to deterioration, such as cantilever in concrete and masonry.</td>
<td>Of dry wood termites Insect borers of wood Subterranean termites Arboral Fungi mold/staining/decay</td>
</tr>
<tr>
<td>6</td>
<td>Exposure to salt or brackish water.</td>
<td>Marine borers Fungi mold/staining/decay</td>
</tr>
</tbody>
</table>

CALIL & BRITO (2010)[4], adapted to the new proposal on the revision of NBR 7190.

In cases of structural systems for bridges, as CALIL et al (2006) [5], basically the wood components are classified in Classes of Use 4, 5 and 6. Systems already in structural and building of residential buildings, for example, wood components can be classified in classes Use as specified in figure 5. For the structural elements in contact with salt or brackish water shall be used for Class of Use 6.

2.4 CHAPTER 4: TYPES OF CONNECTIONS

The joints, also known as connections from the technical, fundamental points should be considered in the safety of wood structures. In some situations, the failure of a connection may be responsible for the collapse of the structure.

Connections in structures with roundwood pieces are more difficult to be made of the connections with lumber. In order to promote more efficient use of connections between the structural elements of roundwood pieces, Chapter 4 of the manual deals with the main types of connections usual for these structural elements, and presented some of these main types connections, presented in CALIL & BRITO (2010)[4].

2.5 CHAPTER 5: TYPES OF STRUCTURAL SYSTEMS

The structural elements object of study in structural systems are presented in Chapter 5 of the manual. They are generally circular cross-section bars varying wood species of eucalyptus and pine. Through the union of these bars can be formed compositions of simple elements or compounds. The different arrangements of these bars enable a range of structural system, can compose structures formed by the structural composition of the various systems being defined as a mixed structure.

The compatibility of the use of parts roundwood all other materials in construction enables a range of composite arrangements, even when these are made up of elements made of different materials, both in structure and in the closure. In Figure 6 we present some of the main types
of construction systems and structural parts using roundwood from reforestation, where some part of the attached of chips in Chapter 8 of the manual, presented in CALIL & BRITO (2010) [4].

2.6 CHAPTER 6: CRITERIA FOR CALCULATION FOR DESIGN
In chapter 6 of the manual are described the main criteria for the design of structural elements of roundwood. Considering the results of tests for classification and characterization of some species of roundwood from reforestation wood, tested in LaMEM was possible to have a general table, showing the average values of the properties of strength and stiffness, for consultation drafts, presented in CALIL & BRITO (2010) [4].

The criteria for calculation for design the scaling of roundwood section shall be in accordance with item 7.2.8 of NBR 7190:1987, whose pieces of circular section, under the action of any normal or tangential, can be considered as if they were of square section, area equivalent. The variable parts of circular section can be calculated as if they were uniform section equal to the section located at a distance from the thinner equal to one third of the total length is not given, however, a diameter greater than 1.5 x d₂ diameter of this end. In figure 6 represents the equivalent diameter (d_eq) calculation variable parts of circular section located at a distance from the thinner to one third of the total length.

![Figure 6: Equivalent diameter for pieces of circular cross-section variable. CALIL & BRITO (2010) [4].](image)

Where:
- d_eq corresponds to the equivalent diameter of calculation;
- d₁ is the bigger diameter (diameter of the base);
- d₂ is the smallest diameter (diameter of the top);
- L is the total length of the part.

At the end of Chapter 6 of the manual, were added two calculation models as an example of application according to NBR 7190.

The first deals with the design criteria of a column under axial load applied normal to the compression.

The second deals with criteria for verification of flexion in a third of a piece of roundwood coverage of variable section, both the ultimate limit state (ULS), the limit in the State Service (ELS), presented in CALIL & BRITO (2010) [4].

2.7 CHAPTER 7: GUIDELINES FOR PROJECT
In Chapter 7 of the manual are mentioned guidelines for the design and dimension of structures with roundwood, including tables of pre-practice design of bridges composed of roundwood beams, bridges and roundwood mixed concrete, and sheds gantry, presented in CALIL & BRITO (2010) [4].

2.8 CHAPTER 8: DATA SHEETS
In chapter 8 of the manual, were listed as an attachment, 124 fact sheets with various types of projects and works undertaken in Brazil and abroad, developed by Brito (2010), which illustrate examples of structural systems and construction using pieces of round wood treated especially species of wood from forest (eucalyptus and pine), usual in the development of structural projects in construction, such as piles of foundations, walkways, bridges, gazebos, sheds rural, residential, commercial, hotels, churches, institutions education, headquarters environmental and ecological parks, local event structures, roofs, structures, bleachers, tourist parks and children's toys, airport terminal, observation towers, structures, ways of shoring for concrete structures, road fenders, noise barriers, among others, presented in data sheets, in CALIL & BRITO (2010) [4].

3 CONCLUSIONS
The LaMEM is one of the most important research laboratories of wooden structures in Brazil and in recent years has held several lines of theoretical and experimental research of structural elements and connections between structural elements with roundwood, especially from reforestation, as species of eucalyptus and pine.

In May 2010, Engineer Civil Leandro Dussarrat Brito, presented the dissertation of master in structures engineering "Recommendations for the design and construction of structures with roundwood from reforestation", directed by teacher Dr. Carlito Calil Junior in the Department of Structural Engineering School of Engineering of São Carlos University of São Paulo.

Based on the research dissertation of BRITO (2010) [3], a technical book entitled "Manual for design and construction of structures with roundwood from reforestation", was developed.

The principal aim was to offer to students and professionals in the fields of Civil Engineering and Architecture, several models of information technology systems and structural and constructive suggestions of types of connections between structural elements. It is also indicated in Chapter 7 of the book, the guidelines for the design and dimensioning of roundwood structures, including tables practices of pre-sizing of bridges composed of roundwood beams, bridges and concrete mixed roundwood, and sheds type porch.

Chapter 8 has already been cataloged in 124 fact sheets with various types of projects and works carried out in Brazil and abroad, with various models of structural systems and construction, using roundwood.

As the purpose of this publication was to make the theme in order to spread the use of structural systems with roundwood from reforestation wood within technological criteria and non-profit, the full and original digital version of the book, ISBN: 978-85-8023-000-0,
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REFERENCES

INTERIM REPORT ON LONG-TERM TEST ON TIMBER–CONCRETE COMPOSITE BEAMS

Mulugheta Hailu¹, Christophe Gerber², Rijun Shrestha³, Keith Crews⁴

ABSTRACT: A long term laboratory investigation started in August 2010 at the University of Technology, Sydney. The test is conducted on four 5.8m span LVL-concrete composite beams (referred to as TCC beams here onwards) beams with four different connector types: Type 17 screws, four and six notches with coach screw and SFS screws. The materials used are Laminated Veneer Lumber (LVL) for the beams and 32 MPa concrete for the flanges. The investigation is still continuing. The specimens have been under sustained loads of (1.7kPa) whilst the environmental conditions have been cyclically alternated between normal and very humid conditions whilst the temperature remains quasi constant (22 °C) – typical cycle duration is six to eight weeks. With regard to EC 5, the environmental conditions can be classified as service class 3 where the relative humidity of the air exceeds 85% and the moisture content of the timber samples reaches 20%. During the test, the mid-span deflection, moisture content of the timber beams and relative humidity of the air are continuously monitored. The paper presents the results and observations of the long-term test to date. It will discuss numerical models found in the literature and examine their fitness to predict the long-term behavioural responses of the specimens.

KEYWORDS: Timber-concrete composite, mechano-sorptive creep, creep factor

1 INTRODUCTION

Timber-concrete composite (TCC) floor system is a construction technique where by the concrete is placed on top of the timber and connected with shear connectors such that timber and concrete work as a composite element and are essentially in tension and compression. Respectively. The three components of TCC floors, timber, concrete and connection, are characterized by different time dependent behaviour. This behaviour is affected by, the stress level, moisture content, temperature and relative humidity of air. Concrete is characterized by significant creep and shrinkage phenomena, timber by creep, mechano-sorptive and shrinkage/swelling [1-16], and connection by creep and mechano-sorptive effect. Due to complexity of the composite action, a series of experimental tests is desirable in order to investigate the real behaviour of the composite structure.

Few long-term tests performed on TCC structures have been performed so far. [17-20, 23-28] Yeoh [17] tested three eight-meter T-section floor beams with three types of connectors in uncontrolled, unheated indoor environment, classified as service class 3 as per Euro code 5. The test result showed largest creep coefficients for plate connection and least with rectangular notch connection. The long-term deflection for beams with normal concrete was larger than the beams with low shrinkage concrete.

Ceccotti [18] tested a TCC floor system six-meter span with glue in connection in outdoor unsheltered well ventilated conditions and protected from direct radiation from the sun. The test lasted for five years and it was classified as service class 3 as per Euro code 5. The maximum average deflection monitored throughout the test was 3.36 mm, which was almost four times the instantaneous elastic deflection and is well below the limiting value L/300 as per the Euro code 5 (2008). Buo Said [23] monitored for 2 years a composite beam with glue-in mechanical connectors loaded in sheltered outdoor conditions. The short term deflection estimated

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using Eurocode 5 was exceeded and it also exceeded the long-term deflection limits. Ahmadi [20] tested a simply supported 3.9 meters long TCC slab loaded with a full sustained live load of 2kN/m² for 140 days. The ultimate long-term deflection under this 100% sustained live load was in acceptable limits set by the building codes AITC and ACI 318-89 and the relative dimensional changes in the timber joists due to variations of humidity and temperature caused cracks in the concrete. The tested repeated with longitudinal movement of wood joists restrained no cracks were observed.

This paper reports the outcomes of up to date long-term tests on four T-section floor beams subjected to quasi-permanent loads. This research will produce simplified guidelines and prediction model for determination of long-term performance of TCC beams and floors. Such information will be useful for development of design guidelines and methods for TCC beams and floor systems to meet ultimate and serviceability limit states which is practical and easy to use for design engineers through design charts, graphs and/or span tables. The application is targeted specifically for non-residential buildings but can be equally applied for domestic buildings.

## 2 EXPERIMENTAL PROGRAM

Four TCC beams with identical geometry (Figure 1), each with a span of six meters, are currently being tested under long-term test in the civil engineering structures laboratory at University of Technology Sydney which was started on 04-August-2010 and 200 days after the concrete pour.

### 2.1 TEST SPECIMEN

Each specimen consists of three components, namely, the concrete topping, LVL joist and shear connectors as shown in Figure 1.

![Figure 1: Cross section of the TCC beam (Typical) (measured in mm)](image)

The top slab is made of a normal weight concrete with a characteristic comprehensive strength of 32 MPa. Results for the concrete compressive strength are summarised in Table 1. A standard reinforcing mesh of 7 mm diameter normal strength reinforcement bars at 200mm spacing was provided in the concrete slab to prevent shrinkage cracks.

### Table 1: Concrete compressive strength (Pham [28])

<table>
<thead>
<tr>
<th>Days</th>
<th>Concrete strength (MPa)</th>
<th>Mean drying shrinkage (microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>39.54</td>
<td>0.0012</td>
</tr>
<tr>
<td>56</td>
<td>44.32</td>
<td>0.0008</td>
</tr>
<tr>
<td>91</td>
<td>50.33</td>
<td></td>
</tr>
</tbody>
</table>

The joists are made of 250 x 45 mm LVL. The mean young’s modulus of the LVL was 13.2 GPa and the characteristic bending strength was 48 MPa. The material property values of LVL are supplied by the manufacturer (Carter Holt Harvey).

A short term static test was done to determine the modulus of elasticity of the LVL according to AS/NZS 4357:2:2006 and the results are given in table below.

### Table 2: The Modulus of Elasticity of the Timber (LVL) (Pham [28])

<table>
<thead>
<tr>
<th>Specimen</th>
<th>MoE LVL after carving (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B_NS</td>
<td>12402</td>
</tr>
<tr>
<td>B_4N</td>
<td>13153</td>
</tr>
<tr>
<td>B_6N</td>
<td>13482</td>
</tr>
<tr>
<td>B_SFS</td>
<td>12312</td>
</tr>
</tbody>
</table>

### 2.1.1 Connectors

Four different types of shear connector configurations were used in the four beams. Normal (Type 17) wood screws at 500mm spacing were used as shear connector in the first beam (B-NS). For beam B-4N, four triangular notch connections with minimum 600 mm and maximum 1700mm spacing between notches were used while for beam B-6N six triangular notch connections were used with minimum 500 mm and maximum 1400mm spacing between the notches – all notches to include a Ø 16 mm x 200 mm coach screw. The fourth beam (B-SFS) was constructed with a pair of SFS crews at ±45° angles at a spacing of 300 mm as shear connectors. The slip modulus for each type of connector obtained from the push out test on specimens (AS1649-2001) with similar shear connectors are also summarised in Table 3. The geometry of the triangular notches and SFS screws used is shown below in Figure 2.

### Table 3: Type of connector and slip modulus

<table>
<thead>
<tr>
<th>Type of connector</th>
<th>Slip modulus, (K_{slip}) (kN/mm)</th>
<th>Characteristic strength, (Q_k) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B_NS</td>
<td>45.0</td>
<td>10.9</td>
</tr>
<tr>
<td>B_4N</td>
<td>36.9</td>
<td>59.5</td>
</tr>
<tr>
<td>B_6N</td>
<td>36.9</td>
<td>59.5</td>
</tr>
<tr>
<td>B_SFS</td>
<td>54.9</td>
<td>32.6</td>
</tr>
</tbody>
</table>
A four point bending test was done on each TCC beam to determine their stiffness. It is summarised in Table 4.

**Table 4: TCC beams stiffness (Pham [28])**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stiffness (kN/m)</th>
<th>EI (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B_NS</td>
<td>810.7</td>
<td>2.77E12</td>
</tr>
<tr>
<td>B_4N</td>
<td>1210.4</td>
<td>4.19E12</td>
</tr>
<tr>
<td>B_6N</td>
<td>1276.9</td>
<td>4.61E12</td>
</tr>
<tr>
<td>B_SFS</td>
<td>1344.9</td>
<td>4.66E12</td>
</tr>
</tbody>
</table>

2.2 LOADING

The TCC beams were relocated to the fog-room for long-term test in semi-controlled environmental conditions. The room is indoor and the beams are not exposed to direct radiation from the sun. Figure 3 shows the beams under sustained loads in the fog room. And Figure 4 shows schematic diagram of the test set up.

2.3 INSTRUMENTATION AND DATA MEASUREMENT

In order to monitor the long-term behaviour of the TCC beams, a number of instruments were used to measure and record deflection, temperature and humidity. The mid-span vertical displacement is measured using Linear Voltage Displacement Transducers (LVDT’s) with a range ±25mm with 0.01mm resolution. The deflection is recorded every hour. The moisture content of the timber beams are measured using a small LVL test samples every week and larger test samples at the end of each climate cycle. The temperature and relative humidity of the environment are measured using climate data loggers.

3 TEST RESULTS

3.1 ENVIRONMENTAL CONDITIONS

The relative humidity (RH) and temperature (T) of the room are measured regularly every hour. The changes in RH, T and timber moisture content (MC) are shown in Figure 5. An air humidifier is used to increase and maintain high RH during the wet periods.
3.2 MOISTURE CONTENT

To monitor the variation of moisture content in LVL beams, separate moisture content samples with 100x100x45mm sizes cut from same batch of LVL were kept in the fog-room and the changes in the level of moisture are measured regularly. The test samples for the moisture content are shown in Figure 6.

Figure 6: LVL MC Test samples

The moisture content obtained from the test samples was around 9% at the start of the test increased and reached above 20% after the humidifier was operated. It is observed from the result that it takes at least two weeks for the moisture content samples to attain 10% additional moisture content.

Since the moisture content samples used are too small to represent accurate MC variations of the LVL beams, larger sized MC samples 600x250x45mm cut from the same batch of the beams are also kept in the room and the moisture variations at the end of every wet and dry period is monitored with these samples. The result of the moisture content measurement from both groups of MC samples had a significant variation. The moisture content values are the mean values of moisture content obtained from the four large test samples. The locations of the test samples in the fog-room affects the moisture content of the samples and hence are positioned considering the variations of the room humidity.

3.3 MID-SPAN DEFLECTION

The mid-span deflection was measured every minute during loading of the specimen for the initial 24 hours and every hour for the remaining of the long term test. Deflections were measured using LVDT’s mounted on metal frame.

The instantaneous elastic deflections of the beam immediately after the application of the loads were as shown in Table 5. The beam with normal screw as shear connector deflected twice more than the other three beams.

Table 5: Instantaneous deflection after the application of load

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Instantaneous elastic deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-NS</td>
<td>8.24</td>
</tr>
<tr>
<td>B-4N</td>
<td>4.67</td>
</tr>
<tr>
<td>B-6N</td>
<td>4.39</td>
</tr>
<tr>
<td>B-SFS</td>
<td>4.17</td>
</tr>
</tbody>
</table>

As shown in Figure 7, the deflection increased sustainably for the first two weeks. Thus the beams experience some magnitude of creep. During this time period the relative humidity of the air and the moisture content of the timber were fairly constant that is RH = 50% and T = 20 °C.

Several phenomena have been observed from the long-term test result shown in Figure 7 these are described below;

When the specimens were loaded on the dry state, despite the instantaneous deflection due to the loads, moderate creep occurred within the first week of the test. After approximately one week (First cycle) water vapour was sprayed, in this first adsorption stage the deflection increased rapidly, that is, creep rate accelerated sharply. In a similar manner on re-drying (first dry cycle) the specimens, the deflection increased abruptly reaching twice the initial deflection of the specimens, but the rate of deflection decreased after a while.

During subsequent wet or humid cycles (Second cycle and all other cycles), the specimens experience some recovery. While each desorption is followed by increase in deflection and each absorption is followed by recovery of the beams.

Deflection several times the instantaneous elastic deflection was thus ultimately reached. Such large deflection could not generally be obtained under short-term loads at constant moisture content without breaking the specimen. Similar results have been reported in Armstrong [1] and Hearmon [2].

3.4 DISCUSSIONS

The results of these long-term investigations indicate that the rate and magnitude of deflection for the TCC beams under sustained load is affected by a change in moisture content. Most of the change in deflection occurs within period during which moisture content cycle takes place.

The up-to-date experimental result shows steady increase in creep for all specimens with time. Beam B-NS with normal screws as shear connectors showed highest creep while the other three beams had similar but lower creep.

A comprehensive plot of the test result showing the mid-span deflection, moisture content is shown in Figure 7.
The moisture content of the beams under load is cycled from dry to wet and back to dry again, the deformation also followed a cyclic pattern. However the recovery in each cycle is only partial and over four cycles the total amount of creep is very large as shown. The greater the moisture difference is in each cycle, the higher is the amount of creep. The air humidity cycled between about 50% to 100% It should be noted that creep increased during drying (desorption) and decreased during the wetting cycle (adsorption) with the exception of the initial wetting when creep increased and this trend is reported in several research works [1-3],[16] and represents a typical creep behaviour of wood in bending and wood based materials [4]. The changes in deflection during each complete cycle would have been much less if the range of change of vapour pressure was made narrower during moisture content cycling under load. Nevertheless, a decrease in deflection for each absorption and an increase for each desorption were still to be found for each cycle after the first cycle as was also reported by Armstrong [1].

Depending on the load and environment history, $K_{\text{creep}}$ values between 0.5 and 2.0 may be expected for timber at 10 years according Eurocode-5. However, it seems that corresponds to a gross underestimating of the creep magnitude of TCC structures. In Figure 8, the relative creep of the TCC beams is shown. It indicates that creep coefficient up to 6.0 should be considered. In case of poor composite action this creep factor should be increased to 9.0.

The creep mechanism in a timber concrete composite is known to be complex and the results after one year are limited to make definitive conclusions. However, the results up to date show that all the three beams deflected approximately six times their instantaneous deflections while non-notched beam (B_NS) deflected eight times the initial deflection. All the beams exceeded the limit of span/200 generally accepted by EC-5.

The test result showed largest Creep coefficient for beam with a normal screw connection and least with birds’ mouth notch and SFS screw connections.

ACKNOWLEDGEMENT
This investigation is sponsored by the Structural Timber Innovation Company (STIC) which is gratefully acknowledged.

REFERENCES
[8] Hunt, D.G., 'Linearity and non-linearity in mechano-sorptive creep of softwood in compression and


ABSTRACT: The stiffness of load spreader has a direct impact on the failure mode and lateral resistance of wood frame shear walls in shear wall tests. Furthermore, it is more affected when hold-downs are not fixed at the ends of the walls and openings. A new load spreader setup is designed and used to conduct the test research under monotonic and cyclic protocol. The test specimens are three full size wood frames shear walls; one is a standard wall, one with door opening and the other one with a window opening. The failure mode, shear strength, ultimate displacement and lateral displacement stiffness are studied. The test results are compared with the same configures specimens by using flexibly and rigid load spreaders.

KEYWORDS: light wood structures; wood frame shear walls; performance

1. ABSTRACT

In this paper, three types of load spreader beams were designed to carry out monotonic and cyclic loading tests on three full -scale wood frame shear walls with different openings. This paper focuses on their failure modes, load displacement curves, shear strength, ultimate displacements, elastic lateral stiffness, as well as the comparative analysis of the lateral force test results of wood frame shear wall by virtue of three different types of load transfer beams.

2. Test Overview

2.1 SPECIMEN DESIGN

Three specimens, namely, Wall-A, B and C, have been designed. According to structural specifications[1], there is one bottom plate, two top plates and two end studs for a wood frame shear wall. The top and bottom plate with the studs are vertically connected using 3.9 mm diameter (90 mm long) spiral nails which are made in China. The 9.5 mm oriented strand board (OSB) was connected to the framing members with 3.0 mm diameter spiral nails (60 mm long) spaced 150 mm along the panel perimeter and 300 mm elsewhere. For test parameters of the three specimens, please refer to Table 1.

2.2 TEST SETUPS

There are three types of load transfer beams using in this test. The first one is to design a load transfer beam with five segments, 1.2m long each, which is the same as the width of the OSB panel. Each segment is hinged to another segment so that it can transfer horizontal load and rotate. Hereinafter, it will be referred to as hinged load transfer beam[2].

The second is to weld six 12mm nuts to the position near the hinging points of the hinged load transfer beam, and use the 12mm screws to mount the two 80×80mm continuous square steel beams that have been pre-welded into an integral body to the top of hinged load transfer beam. Therefore, the stiffness of the load transfer beam will be greatly improved, hereinafter, it will be referred to as continuous load transfer beam.

The third loading setup (see figure 1), which is...
mounted at one side of the wood shear wall so that it can ensure that the cantilever force transmission setup would not affect the rotation and uplift of wood shear wall after being loaded while it transfers the force applied by the hydraulic servo loading equipment to the wood shear wall. Meanwhile, the force transmission setup won’t affect the force transmission when it deforms in a manner that is the same as the wood shear wall.

Table 1. Test Matrix of Wood Frame Shear Wall

<table>
<thead>
<tr>
<th>Wall</th>
<th>Wall No.</th>
<th>Opening (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall-A</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>—</td>
</tr>
<tr>
<td>Wall-B</td>
<td>3</td>
<td>1.2×2.1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.2×2.1</td>
</tr>
<tr>
<td>Wall-C</td>
<td>5</td>
<td>1.2×2.1</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.2×2.1</td>
</tr>
</tbody>
</table>

The load protocol of ISO-16670 is applied for this test.[3]

3. FAILURE MODES

The studs will be slightly bent when the wood shear wall reaches the ultimate load, the wood frame changes into a parallelogram from a rectangle and there is a significant relative dislocation between the panel and the wood frame. Such damage is mainly caused by nail connection failure as well as the separation between the studs and the bottom slab. Extrusion failure of wall panels and bottom slab cracking can also be observed occasionally.

There are four different nail connection failures, namely, the nails withdraw from the wall panel, the nail heads pull through the wall panel, panel chipped out at the edges of the wall panel and nail fatigues under the reverse load.

For the walls without holdown, when the uplift force is larger than the resistance provided by the nail connection between the studs and bottom plate as well as that between the wall and bottom plate, the studs and bottom plate will separate from each other and cause stud uplift. The uplift will first occur at the edge stud. Along with the worsening failures and the increasing lateral displacement, the central studs will successively separate from the bottom plate. In a monotonic test, such a separation only occurs at the tensile sides. In a cyclic test, the stub separation will occur at both sides of the wall. (see figure 2a-2d,3a,3b)

![Figure 1](test_setup.png)

![Figure 2](failure_nail_joints.png)

![Figure 3](uplift_studs.png)
4. TEST RESULTS AND ANALYSIS

4.1 TEST RESULTS

Table 2 presents the test results of the specimen under the monotonic and cyclic load.

Table 2. Result of the test

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Load</th>
<th>( f_{d} ) (KN/m)</th>
<th>( \Delta u ) (mm)</th>
<th>( K_e ) (KN/m/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T4</td>
<td>C4</td>
<td>Av.</td>
<td>T4</td>
</tr>
<tr>
<td>Wall-A</td>
<td>1</td>
<td>6.36</td>
<td>-</td>
<td>6.36</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>5.81</td>
<td>5.31</td>
<td>5.56</td>
</tr>
<tr>
<td>Wall-B</td>
<td>3</td>
<td>4.84</td>
<td>-</td>
<td>4.84</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>5.19</td>
<td>5.43</td>
<td>5.31</td>
</tr>
<tr>
<td>Wall-C</td>
<td>5</td>
<td>8.88</td>
<td>-</td>
<td>8.88</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>7.19</td>
<td>7.03</td>
<td>7.11</td>
</tr>
</tbody>
</table>

Notes: 1. \( f_{d} \) – unit shear strength, kN/m, which is the shear strength divided by the total lengths of the test wall; 2. \( \Delta u \) – ultimate displacement of the shear wall, which is the displacement at 80% of ultimate load in the descending portion of the load-displacement curve (ISO, 2003); 3. \( K_e \) – unit initial stiffness, kN/mm/m, which is defined as the secant stiffness between starting point and the point where displacement equals to 9.6mm, equivalent to 1/250 of the wall height.; 4. T—tension; C—compression; 5. Test is stopped before the specimen reaches the ultimate displacement.

4.2 SHEAR STRENGTH

The shear strength was shown in Figure 4. It can be obtained by virtue of the average values of the shear strength as shown in Table 2.

The unit shear strength of Wall-C is significantly higher than Wall-A and B since the plate over the opening has been ignored while calculating the shear strength.

After the specimen with the same construction added with a rigid load transfer beam, the shear strength will be increased to some extent. With regard to the impact of stiffness of transfer beam on shear strength, continuous steel beam > hinged steel beam > out-of-plane steel beam. After Wall-A is added with a out-of-plane steel beam, the shear strength will be reduced by 13% compared with that of hinged steel beam and by 26.8% compared with that of continuous steel beam; After Wall-B is added with a out-of-plane steel beam, the shear strength will be reduced by 11.5% compared with that of hinged steel beam and by 30% compared with that of continuous steel beam; After Wall-C is added with a out-of-plane steel beam, the shear strength will be reduced by 33% compared with that of hinged steel beam and by 39% compared with that of continuous steel beam.

For out-of-plane load transfer beam, since it can not restrict the wood shear wall specimen, the deformation of top plate in the shear wall specimen is larger than that of the specimen with the same construction equipped with hinged load transfer beam or rigid load transfer beam. Therefore, the shear strength of the out-of-plane load transfer beam specimen is smaller than that of the other specimens.

![Figure 4. The shear strength of test walls using three different load spreaders](image)

4.3 ULTIMATE DISPLACEMENT

Figure 5 shows the average value of ultimate displacements of the three different specimens with different loading beams in the cyclic load test. The ultimate displacement of the wood shear wall with out-of-plane load transfer beam is lower than the ultimate displacement of the wood shear wall with hinged steel beam at the wall top. For example, the
ultimate displacement of Wall-A with out-of-plane load transfer beam is 34% lower than that with hinged steel beam at the wall top and 28% lower than that with rigid transfer beam; for Wall-B, due to the instrument, the value bias is too large. Therefore, it is not mentioned here.

The main reason why the ultimate displacement of out-of-plane load transfer beam is lower than that of hinged steel beam and continuous steel beam is that there is no confinement from the loaded steel beam on the wood shear specimen. During the loading-bearing process of the specimen, the partial deformation of the wall panel is much larger than its overall deformation. It has been found that, in the test, the rotation of all OSB panels and stud uplift are both larger than that of the specimen with hinged steel beam at the wall top as well as the one with rigid load transfer beam. Therefore, after being loaded to some extent, the specimen will have been completely damaged when the ultimate displacement (overall deformation) of the specimen is still not very large. Under the same test conditions, the ultimate displacement will become larger and larger along with the enlargement of the opening size.

The limit displacement of test walls using three different load spreader.

**Figure 5** The limit displacement of test walls using three different load spreader.

### 4.4 INITIAL STIFFNESS

Figure 6 shows the average elastic lateral stiffness of the three different specimens with different loading beams in the cyclic load test. Seen from the elastic lateral stiffness values of all specimens listed in Table 2, except Wall-A, the elastic lateral stiffness of the wood shear wall with hinged load transfer beam and that of the wood shear wall with rigid load transfer beam are almost the same, which is inconsistent with the expected effect of the rigid load transfer beam. In theory, the elastic lateral stiffness of the wood shear wall with rigid load transfer beam will be larger than that of the wood shear wall with hinged load transfer beam. The reason for such an inconsistency might be that the connecting bolts for rigid load transfer beam and hinged load transfer beam are moderately tightened by hand and there is only six connection points. When the displacement of the wall specimen is small, the rigid load transfer beam only has a limited role in preventing stub uplift. Therefore, there are only a few differences between the elastic lateral stiffness values of the specimens with different boundary conditions.

It can be seen from Figure 6 that the elastic lateral stiffness value of the wood shear wall specimen with out-of-plane load transfer beam is smaller than that of the wood shear wall specimen with hinged load transfer beam or the one with rigid load transfer beam. The reason can be drawn from the definition of elastic lateral stiffness, since the lateral stiffness is associated with the shear strength and deformation properties of the wood shear wall.

It can be found out in Figure 4 and Figure 5, that the shear strength and ultimate displacement of the wood shear wall specimen with out-of-plane load transfer beam are both smaller than the shear strength and ultimate displacement of the specimen with hinged steel beam or with continuous steel beam. Therefore, the elastic lateral stiffness of the wood shear wall specimen with out-of-plane load transfer beam is smaller than the elastic lateral stiffness of the specimen with hinged steel beam or with continuous steel beam. This result is consistent with the expected conclusion.

**Figure 6.** The rigid stiffness of test walls using three different load spreader.

### 4.5 THE LOADING-DISPLACEMENT CURVE

The loading-displacement curve in the tests was shown in the figure 7-9. For the monotonic loading test, the curve in the third quadrant is obtained by the mirror method. One can see that the cyclic curve looks thin and shaped like S in reverse. The loading-displacement curve of monotonic test is similar to the envelope of the first cycle of reversed loading test.
5 CONCLUSIONS

This paper describes specimen design, test setups, test content and load protocol, etc. It focuses on the impacts of different load spreader beams on the shear strength and displacement as well as elastic lateral stiffness of different wood frame shear walls and failure modes of wood shear walls. This paper also presents the load-displacement curve test result of wood shear wall in monotonic and cyclic load tests. Based on the above analysis, we can draw the following conclusions:

Adding rigid steel beam and new-type loading setup have no significant impact on the failure modes of the wood shear wall. The failures of wood shear wall are still divided into two categories: nail connection failure and the separation between the bottom of the studs and bottom slab. The failure positions are mainly distributed at the bottom edges and the two sides of the wall panel, the middle part of the wall panel is basically intact during the test.

Compared with the specimen only with hinged load transfer beam, adding rigid load transfer beam can effectively improve the shear strength of the wood shear wall. Since the out-of-plane load transfer beam cannot confine the wood shear wall specimen, the specimen shear strength is smaller than that of the other specimens with the same construction.

The elastic lateral stiffness of the wood shear wall specimen with out-of-plane load transfer beam is smaller than the elastic lateral stiffness of the wood shear wall specimen with hinged steel beam or with continuous steel beam.

The load-displacement curve of monotonic load test is in agreement with the envelop of the first cycle of cyclic load test for the same wood shear wall specimen that there is no significant difference. The load-displacement curve of cyclic test is not plump that it shows a significant antithetic s shape. Through several cyclic loads, the stiffness of the specimen will be degraded and there is rheostiction and hysteretic behavior in deformation restoration.

REFERENCES


EXPERIMENTAL STUDY ON BENDING CREEP BEHAVIOUR OF REINFORCED GLULAM BEAM

Weidong Lu\textsuperscript{1}, Erwei Song\textsuperscript{2}, Min He\textsuperscript{3}, Kong Yue\textsuperscript{4}, Weiqing Liu\textsuperscript{5}

ABSTRACT: This investigation is performed to see the creep effect on the performance of glulam beam under different conditions, including different stresses (i.e. 30% and 50% of ultimate strength), different experimental environments (i.e. constant temperature and humidity, indoor and outdoor) and different reinforced methods. A detailed study on bending creep behaviour of glulam beam is presented. Through the experiment, creep law of the glulam beam was analyzed, and the experimental results were discussed. After building the bending model, the creep data was fit, the creep deflection curve and relative creep deflection curve were drawn, followed by a prediction about the relative creep deformation in 50 years. The results showed the creep performance can be greatly improved with the reinforced material.

KEYWORDS: Glued Laminated Timber; Beam; Creep Testing; Creep Deflection; FRP

1 INTRODUCTION

During the recent years, the widely use of fibre-reinforced polymer (FRP) and rebar planting for reinforcement of both strength and stiffness of glulam structure members and components has been developed as an economical way. More and more researches have been conducted to further improve the structural performance of glued-laminate (glulam) beams. As an important influence factor to the structure performance, the creep effect of the glulam structure\textsuperscript{[1],[2]} also played a significant role. It has been found that with the help of FRP or rebar, the bending strength of the glulam could be increased considerably or even doubled. Based on this\textsuperscript{[3],[4]}, the experimental study on bending creep behaviour of reinforced glulam beam has been conducted. From a modelling point of view, and for final engineering use, a standardization of creep testing is recommended. The test specification of the specimen is $50 \text{mm} \times 150 \text{mm} \times 2850 \text{mm}$, which combined with 5 pieces of $50 \text{mm} \times 30 \text{mm} \times 2850 \text{mm}$ laminates. As to the mechanical test, it is known that four-point (or three-point) bending test method is the most frequently performed for beams. In this paper, the four-point loading method was chosen, with both ends of beams hinged. Besides, the climatic conditions and the internal variables are also under control. It’s generally accepted that the constant standard humidity environmental conditions are the “air dry” (65% RH, 20°C) and the “humid” (85%RH, 20°C), and in this study it is between 60~65% RH, 18~22°C.

The experiment is performed by testing 10 different types of glulam beams under different environment for 6 months, and one of the specimens is used as a comparison. The experimental device and experimental level table were shown in figure 1 and table 1 respectively for detail.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{experimental_device.png}
\caption{Experimental device}
\end{figure}
2 EXPERIMENT ON BENDING CREEP BEHAVIOUR OF GLULAM BEAMS

2.1 INTRODUCTION OF THE EXPERIMENT

2.1.1 Experimental purposes
Through imposing constant stresses (under different percentages of ultimate strength) on unreinforced or reinforced glulam beams, the bending creep behaviour were tested. The influences of different factors (i.e. stress, environment condition and reinforced method) were summarized. Based on experimental datas, the authors fitted the creep curves to predict the relative creep deformation in the future so as to optimize the design of glulam beams.

2.1.2 Specimen design
The specimen design of this experiment followed the “Code for design of timber structures” and relevant foreign standards. The test parameters were different stresses, environment conditions and reinforced methods. Specific test parameters are shown in Table 1 below.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Stress</th>
<th>Environmental condition</th>
<th>Reinforced method</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>--</td>
</tr>
<tr>
<td>L2</td>
<td>50% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>--</td>
</tr>
<tr>
<td>L3</td>
<td>30% of ultimate strength</td>
<td>Indoor</td>
<td>--</td>
</tr>
<tr>
<td>L4</td>
<td>30% of ultimate strength</td>
<td>Outdoor</td>
<td>--</td>
</tr>
<tr>
<td>L5</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>One layer of FRP in the tensile area</td>
</tr>
<tr>
<td>L6</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>Two layers of FRP in the tensile area</td>
</tr>
<tr>
<td>L7</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>One layer of FRP in the compression area and two layers in the tensile area</td>
</tr>
<tr>
<td>L8</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>Φ6 in the compression area</td>
</tr>
<tr>
<td>L9</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>Φ6 in the tensile area</td>
</tr>
<tr>
<td>L10</td>
<td>30% of ultimate strength</td>
<td>Constant temperature and humidity</td>
<td>Φ8 in the tensile area</td>
</tr>
</tbody>
</table>

2.1.3 Load device and measurement program
In this experiment, the four-point bending test method was taken. A point load was plused to the third point of the simply supported beam. The imposed load were 30% and 50% of the ultimate strength of the glulam beams. Experimental timber properties including density, moisture content, tensile strength, compressive strength, shear strength, short-term flexural strength and elastic modulus had been tested in advance. Shear properties of adhesive and steel bar pullout test were in the same way. The gravity of the concrete beams acted as the vertical load. The loading weight of a glulam beam with 30% of ultimate strength was 460kg and 50% of ultimate strength was 765kg.

The tests were conducted in the constant temperature and humidity laboratory, basement and outdoor respectively.

It is generally accepted that the constant standard humidity environmental conditions are the “dry air” (65%RH, 20°C) and the “humid”(85%RH, 20°C).and in this experiment it was between 60~65%RH, 18~22°C. The temperature and humidity data of basement and outdoor were taken in the manual way. The displacement records of both ends and crosses of the glulam beams also took the manual-reading method. The sketch of the test device is shown in Figure 2. The loading process is shown in Figure 3.

![Creep test device](image)

*Figure 2: Creep test device*

![Load process](image)

*Figure 3: Load process*
2.2 EXPERIMENT RESULTS AND ANALYSIS

2.2.1 Experimental comparison under different magnitude of stress

L1 and L2 were unreinforced glulam beams. The test was conducted in the constant temperature and humidity laboratory under different stress (i.e. 30% of ultimate strength and 50% of ultimate strength). Creep laws of unreinforced glulam beams under different stress are shown in Figure 4.

![Figure 4: Time-deformation curve and relative creep deformation curve under different magnitude of stress](image)

It is concluded that creep deformation has an obvious relativity with the magnitude of stress according to creep curves of the glulam beams under different stresses. Under the acting of a constant load, the deformation of the glulam beam is increasing with the passage of time. Creep processes under two different percentages of ultimate strength are much similar in a short time. But the relative creep deformation under a high stress is obviously higher. However, creep curves under a low stress show more smoothly.

2.2.2 Experimental comparison under different reinforced methods

Glulam beams L5-L7 were reinforced with FRP and L8-L10 with rebar-planting. The test was conducted in the constant temperature and humidity under the 30% of ultimate strength. L5-L10 took different reinforced methods: L5 with one layer of FRP in the tensile area, L6 with two layers of FRP in the tensile area, L7 with one layer of FRP in the compression area and two layers of FRP in the tensile area, L8 with 1Φ6 in the compression area, L9 with 1Φ8 in the tensile area, L10 with 1Φ8 in the tensile area. The creep deformation of reinforced glulam beams are shown in Figure5 and Figure6.

![Figure 5: Time-deformation curve and relative creep deformation curve under different rebar planting ways](image)

![Figure 6: Time-deformation curve and relative creep deformation curve under different FRP reinforced methods](image)

It is analyzed that the initial deformations of reinforced glulam beams are lower than the unreinforced ones, especially the FRP reinforced ones [5-7]. It is concluded
that the initial stiffness of the reinforced glulam beam is greatly improved and the stiffness would increase with the reinforcement ratio in a certain scope. The results also showed that the relative creep deformations of rebar-planting glulam beams increases with the reinforcement ratio while the FRP reinforced ones differ little with the reinforcement ratio. But it also can be seen that the reinforced beams tend to have a more obvious upward tending with time.

2.2.3 Experimental comparison under different environment conditions

L1, L3 and L4 were all unreinforced glulam beams with 30% of the ultimate strength. L1 was tested in a constant temperature and humidity laboratory while L3 at an indoor condition (for unexpected reason, this beam broke two month after it was loaded) and L4 at an outdoor condition. The creep curves are shown in Figure 7.

2.3 SUMMARY

The experimental results showed by using FRP-reinforced glulam beam, due to the improved initial stiffness, the initial deformation rigidity has reduced by 27% and the relative creep deformation after 50 years has reduced by 80%, which means FRP is a great help to improve the structure performance. Besides, due to different environments, a great influence on creep performance has been observed when the specimens were tested outdoor. Finally, it is suggested the magnitude of stress should be controlled in a low level about 30% of the ultimate strength during engineering design in case of creep damage.

3 STUDY ON BENDING CREEP MECHANISM OF REINFORCED GLULAM BEAMS

3.1 INTRODUCTION TO MODEL ESTABLISHMENT OF REINFORCED GLULAM BEAMS’ BENDING CREEP

During the reinforced glulam beams’ bending creep process, primary and secondary creep deformation especially the latter one play an important role in engineering application and show a significant regularity. Therefore, primary and secondary deformation are analysed and discussed in this study. The authors attempt to describe the bending creep behaviour of reinforced glulam beams with a neat 4-element model [8-10]. The 4-element model is composed of a Hooke body, a Kelvin model and a viscous model.

3.2 PARAMETER FITTING AND PREDICTION OF BENDING CREEP MODEL OF REINFORCED GLULAM BEAMS

According to the experimental results, the constitutive equation for reinforced glulam beams is established with the neat power function equation based on the empirical equation. The equation is shown as follows:

\[ \varepsilon(t) = \frac{\sigma}{E} (1 + at^b) \]  

This constitutive equation only takes primary and secondary creep processes into consideration. The experiment was not destructive and the third stage had not yet appeared according to the creep curves, so the constitutive equation is reasonable ignoring the third stage of bending creep. Furthermore, it also takes the first two stages into consideration during engineering the design. A large number of engineering examples has proved that the creep constitutive equation and its parameters obtained with this method agree quite well with actual situation when used for scene analysis and calculation.

The experimental data was fitting analysed by Origin Software. Origin Software has automatic function to fit the power function \( y = ax^b \) and the power function can be called directly.
The test results showed a high accuracy of the relative creep deformation fitting curves and the correlation coefficient R² is approaching 1. The fitting curve also verifies the correctness of the creep model. The factor a, b and prediction of relative creep deformation are shown in Table 2. According to Table 2, the test value and the fitted value of the relative creep deformation in 60 days differed little. It also verified the correctness of the creep model.

**Table 2**: Factor a, b and Prediction of relative creep deformation

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Fitting factor</th>
<th>60 days relative creep %</th>
<th>Fitting prediction %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>Test value</td>
</tr>
<tr>
<td>L1</td>
<td>3.854</td>
<td>0.258</td>
<td>24.00</td>
</tr>
<tr>
<td>L2</td>
<td>3.497</td>
<td>0.293</td>
<td>28.00</td>
</tr>
<tr>
<td>L3</td>
<td>2.372</td>
<td>0.237</td>
<td>13.22</td>
</tr>
<tr>
<td>L4</td>
<td>2.501</td>
<td>0.229</td>
<td>13.04</td>
</tr>
<tr>
<td>L5</td>
<td>3.789</td>
<td>0.141</td>
<td>10.45</td>
</tr>
<tr>
<td>L6</td>
<td>2.890</td>
<td>0.304</td>
<td>25.11</td>
</tr>
<tr>
<td>L7</td>
<td>3.344</td>
<td>0.298</td>
<td>27.04</td>
</tr>
<tr>
<td>L8</td>
<td>2.176</td>
<td>0.274</td>
<td>15.71</td>
</tr>
<tr>
<td>L9</td>
<td>3.128</td>
<td>0.234</td>
<td>16.21</td>
</tr>
<tr>
<td>L10</td>
<td>3.052</td>
<td>0.177</td>
<td>11.13</td>
</tr>
</tbody>
</table>

4 NUMERICAL SIMULATION

In order to investigate the creep response of the glulam unreinforced and reinforced with FRP or rebar, a series of finite element models were developed in the Abaqus environment. It can be seen that the results of the finite model were accordance with the test results, which proved the correctness of the finite model. Figure 8 shows the results between test and numerical simulation. Based on this, the prediction about the relative creep deformation is made in 50 years, which can provides basis for engineering use.

(c): Test and finite results of rebar planting beams

5 CONCLUSIONS

Through the experimental results of bending creep behaviour of glulam beams, the establishment of bending creep model and the fitting of the experimental data, we can draw the following conclusions:

1. The bending creep curves of glulam beams under low stresses present the two significant stages of creep deformation: instantaneous creep and stable creep.
2. The creep deformation under a low stress approaches to an ultimate value while the creep deformation under a higher stress tends to make the beam fail. Therefore, it is suggested that the stress should be controlled in a low level, about 30% during engineering design, in case of creep damage.

3. The glulam beams with reinforced methods play an important role in reducing the initial deformation and controlling the bending creep deformation. Reinforced glulam beams make full use of timber strength and can provide a reference for practical engineering application.

REFERENCES

AN EXPERIMENTAL STUDY ON RESISTANT MECHANISM OF PLYWOOD PANEL-STEEL COMPOSITE MEMBER

Takumi Ito¹, Wataru Kambe² and Saki Kondo³

ABSTRACT: This paper proposes a new structural system, the plywood panel - steel composite member. To investigate the resistant mechanism, the compression test has been conducted. From the test results, the behaviour of this member is clarified. It is found that the maximum strength is determined by the elastic flexural buckling. From the results of the distribution of the strain in the section, this composite member behaves as the layered beam. Furthermore, the composite flexural rigidity is obtained, and the buckling strength is calculated based on the layered beam model.

KEYWORDS: Composite member, Structural plywood, Combined flexural rigidity, Stiffening effect

1 INTRODUCTION

This paper proposes a new structural system, the plywood panel - steel composite member as shown in Figure 1. This composite member is consisted that the slender steel column is sandwiched with two sheets of plywood. The plywood panel is used as the stiffener for the steel member in this system. The concepts for this structural system are producing the simple constructional process and reducing the volume of steel material for lightweight building structures. And also, in Japan, the thick plywood with high stiffness has been developed in last decade [1]. These are used for the floor, the structural walls and the roof in the general wooden houses. Our new constructional system is to provide a new application method for this material.

In these days, some composite members of steel and wooden member are proposed in Japan (Sakata, et al [2], Takagi, et al [3], Nakashima, et al [4]). In these proposal systems, the custom-build connection-parts are used, so its construction method is getting a little difficult. So then, it is necessary to develop the simple and easy construction or fixing methods.

In this paper, to investigate the resistant mechanism and structural performance of this system, the compression loading test is done. The parametric study is performed to investigate the effects of the thickness and width of plywood panels. And also, the analytical model for vertical strength is proposed based on the test results.

Figure 1: Proposed composite member
properties of the plywood and the steel are summarized in Tables 1 and 2.

To investigate the resistant mechanism of this composite member subjected to compression, the compression loading tests are conducted. Figure 3 shows the test setup and the measurement positions. The specimens are pin-supported at the both ends, and the distance between the pins of top and bottom, that is, the buckling length is 1,700mm. As shown in Figures 2 and 3, the strain on the plywood and the steel, and the deflection of the specimen are measured continuously.

From the conditions mentioned above, the slenderness ratio of steel member is 183, and the critical slenderness ratio is 111.

3 TEST RESULTS AND CONSIDERATIONS

3.1 OUTLINES OF TEST RESULTS

The test results of monotonic compression loading are summarized as followings.

The ultimate state of test specimen after the maximum strength is shown in Figure 4.

The load-deflection curves are shown in Figure 5, the distributions of strain in section are shown in Figure 6.

The relations of the curvature-axial strain of the steel and the plywood are shown in Figure 7. Furthermore, the load-deflection curve of steel is also shown in Figure 5 to compare with not or reinforced by plywood. The test results of maximum strength are summarized in Table 3. The relation of maximum strength and thickness and width of plywood is shown in Figure 8.

3.2 OBSERVATIONS AND CONSIDERATIONS

From the Figures 5, 8, and the Table 3, the maximum strength increases as the thickness of plywood increases. However, the relation of width of plywood and the strength is not confirmed. And also, from the Figure 5, it can be observed that the rigidity shows almost equal.

From the Figure 6, the steel and the plywood almost remain within the elastic range until the maximum strength. From the Figure 7, the curvature of the steel and the plywood suddenly increases at the maximum strength, and the appearance out-of-plane bending deformation was observed as shown in Figure 4. After these points, the load-deformation curves show the load deterioration as shown in Figure 5.

And also, it is confirmed that the strain is not transmitted between the steel and plywood as shown in Figure 6.

From these results, the maximum strength is determined by the elastic flexural buckling.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Young’s modulus (N/mm²)</th>
<th>Bending strength (N/mm²)</th>
<th>Yield strain (µ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12mm</td>
<td>6,860</td>
<td>51.6</td>
<td>3,032</td>
</tr>
<tr>
<td>24mm</td>
<td>4,860</td>
<td>38.4</td>
<td>3,173</td>
</tr>
<tr>
<td>28mm</td>
<td>3,100</td>
<td>19.3</td>
<td>2,497</td>
</tr>
</tbody>
</table>

Table 1: Mechanical properties of plywood

<table>
<thead>
<tr>
<th>Yield strength (N/mm²)</th>
<th>Tensile strength (N/mm²)</th>
<th>Yield strain (µ)</th>
<th>Young’s modulus (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>271</td>
<td>431</td>
<td>1,340</td>
<td>202,000</td>
</tr>
</tbody>
</table>

Table 2: Mechanical properties of steel

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Test</th>
<th>Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>12mm</td>
<td>80mm</td>
<td>128.0 kN</td>
</tr>
<tr>
<td>24mm</td>
<td>80mm</td>
<td>147.0 kN</td>
</tr>
<tr>
<td>28mm</td>
<td>80mm</td>
<td>138.3 kN</td>
</tr>
</tbody>
</table>

Table 3: Comparison of maximum strength of test results vs. analytical results

*: The outline of analysis is explained in Ch.4
Figure 4: Ultimate state of test specimen

Figure 5: Test results of load-deflection curves

Figure 6: Distribution of strain in the section

Figure 7: Relations of curvature-axial strain
4 ANALYSIS AND COMPARISON
In the previous chapter, the resistant mechanism and ultimate behaviour are summarized based on the test results. From the test results, this composite member subjected to vertical loads behaves as follows:
1) The steel and the plywood almost remain within the elastic range until the maximum strength
2) The curvature of the steel and the plywood suddenly increases at the maximum strength.
3) The appearance out-of-plane bending deformation is occurred after the maximum strength. And also, the load – deformation curves show the load deterioration.
4) The strains of steel and plywood are not transmitted between the steel and plywood.

From these results and observations, and this composite member behaves like as the layered beam [5,6] as shown in Figure 9.

Herein, the maximum strength \( P_a \) is calculated from the equation of Euler’s buckling load as follows:

\[
P_a = \frac{\pi^2 (EI)_{cmb}}{L^2} \quad (1)
\]

Where, \((EI)_{cmb}\) is the combined flexural rigidity of composite member as following, and \(L\) is the buckling length.

It is assumed that the combined flexural rigidity \((EI)_{cmb}\) is obtained by use of the layered beam model. That is, the combined flexural rigidity is calculated by the sum of rigidity of plywood and steel as follows:

\[
(EI)_{cmb} = \sum_i (EI)_i \quad (2)
\]

The calculated buckling strength compares with the test results in Table 3. Form the Table 3, the analytical results overestimate around 2 - 20% of the test results, however, its show good agreements with test results.

5 CONCLUSIONS
This paper proposes a new composite structural system of steel and plywood. To investigate the resistant mechanism, the compression loading test is done. From the test results, the behaviour of this member is clarified. Furthermore, the calculated buckling strength based on the layered beam model shows good agreement with test results.

ACKNOWLEDGEMENT
This study was supported by Grain-in-Aid for Advanced Research, Tokyo University of Science, 2010.

REFERENCES
NUMERICAL AND EXPERIMENTAL ANALYSIS OF THE VERTICAL VIBRATIONS ON SEVERAL DESIGNS OF TIMBER FOOTBRIDGES

Vanessa Baño¹, Julio Vivas², Soledad Rodríguez³ and Keith Crews⁴

ABSTRACT: Numerical and experimental vertical natural frequencies of simply-supported, two hinged arch and truss timber footbridges were analyzed. The footbridges were designed to conform to Eurocode 5 and each typology presented different spans (12, 27 and 14 m, respectively). The theoretical values of natural frequency for different mode shapes were calculated using a numerical model. The experimental modal parameters were then measured using accelerometers attached underneath the bridge girders and the impact was induced by hitting with an instrumented impact hammer. The results of the acceleration were measured at five points on each girder, in order to obtain the first and second modal shape in bending and in torsion. The experimental results obtained for first bending modal shape (9.9, 6.4 and 7.3 Hz, respectively) and first torsion modal shape (12.8, 6.9 and 15.7 Hz, respectively) presented no risk of resonance according to Spanish regulations (IAP 2011). Furthermore, the relationship between experimental results and numerical simulation were analyzed. Once the numerical model was validated, new footbridges were designed based on one typology to study the effect of span on natural frequency. The paper also presents a frequency range classification corresponding to the risk of resonance due to pedestrian loading.

KEYWORDS: timber footbridges, accelerometers, natural frequency, modal shapes, risk of resonance.

1 INTRODUCTION

The demand of timber pedestrian bridges in Spain has recently increased with respect other materials like concrete or steel. Timber bridges, compared to those made of other materials, has lower natural frequencies, so this means an increased risk of resonance due to pedestrian traffic and a lower comfortability to step. The recent review of the Spanish regulation and rules relating to loads in the design and planning of road bridges (IAP-11) includes a section about the limit state of vibration in footbridges as a serviceability limit state [1]. The IAP defines a range for vertical frequencies (Table 1).

<table>
<thead>
<tr>
<th>Table 1: Frequency range classification according to IAP-11 for vertical vibration.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency (Hz)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>1st frequency &lt; 1.25</td>
</tr>
<tr>
<td>1.25 - 4.60</td>
</tr>
<tr>
<td>1st frequency &gt; 4.60</td>
</tr>
</tbody>
</table>

According to the IAP, the natural frequencies of pedestrian bridges must be outside this range (1.25-4.60 Hz) to comply with the serviceability limit state. Furthermore, the IAP requires verification by specific dynamic tests when materials different from concrete or steel, such as timber, are used. The determination of the natural frequencies in timber bridges has been studied by several authors [2, 3].

Furthermore, in France the Service d'Études techniques des routes et autoroutes has a similar classification (Table 2) where frequencies are situated to assess the risk of resonance entailed by pedestrian traffic and to verify the comfort criteria [4].

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⁴ Keith Crews, Centre for Built Infrastructure Research, University of Technology, Sydney, Australia. Email: keith.crews@uts.edu.au
Table 2: Frequency range classification according to SETRA for vertical vibration

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>0-1</th>
<th>1.7-2.1</th>
<th>2.1-2.6</th>
<th>2.6-5</th>
<th>&gt;5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range 4</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

where,
Range 1: maximum risk of resonance
Range 2: medium risk of resonance
Range 3: low risk of resonance for standard loading situations
Range 4: negligible risk of resonance

Three typologies of timber footbridges were studied: simply-supported bridge, two hinged arch bridge and truss bridge, in order to study the natural frequencies of vibration. Once the numerical and experimental results were compared, new pedestrian timber bridges were designed and calculated according to Eurocode 5 [5] using finite element software, in order to study the influence of span in the vibration.

2 METHODOLOGY

2.1 BRIDGES TESTED

The following figures show the three typologies studied: simply-supported bridge (figure 1), two hinged arch bridge (figure 2) and truss bridge (figure 3). The table 3 shows the dimensions, cross section of main girders and mass of three bridges studied.

<table>
<thead>
<tr>
<th></th>
<th>L</th>
<th>w</th>
<th>w_g</th>
<th>h_g</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m)</td>
<td>(m)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(kg)</td>
</tr>
<tr>
<td>A</td>
<td>12</td>
<td>2</td>
<td>190</td>
<td>594</td>
<td>2845</td>
</tr>
<tr>
<td>B</td>
<td>27</td>
<td>2</td>
<td>190</td>
<td>variable</td>
<td>8305</td>
</tr>
<tr>
<td>C</td>
<td>14</td>
<td>2.7</td>
<td>190</td>
<td>360</td>
<td>3822</td>
</tr>
</tbody>
</table>

where,
T Typology of footbridge
A Simply-supported straight bridge
B Two hinged arch bridge

Figure 2: Typology B. Two hinged arch bridge
Figure 3: Typology C. Trussed bridge
3 RESULTS

3.1 RESULTS OF THE EXPERIMENTAL TEST

The dynamic frequency analysis test was accomplished for the three typologies of bridges. The experimental values of natural frequencies for first and second bending and torsion mode are shown in Table 4.

Table 4: Experimental results of natural frequencies of the first and second modal shape in bending and torsion

<table>
<thead>
<tr>
<th>Type</th>
<th>L (m)</th>
<th>W (m)</th>
<th>$f_{1e}$ (Hz)</th>
<th>$f_{1e}$ (Hz)</th>
<th>$f_{2e}$ (Hz)</th>
<th>$f_{2e}$ (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>12</td>
<td>2</td>
<td>9.95</td>
<td>12.81</td>
<td>24.89</td>
<td>36.94</td>
</tr>
<tr>
<td>B</td>
<td>27</td>
<td>2</td>
<td>6.42</td>
<td>6.93</td>
<td>7.53</td>
<td>11.05</td>
</tr>
<tr>
<td>C</td>
<td>14</td>
<td>2.7</td>
<td>7.29</td>
<td>15.70</td>
<td>18.31</td>
<td>24.21</td>
</tr>
</tbody>
</table>

where,

A Simply-supported straight bridge
B Two hinged arch bridge
C Trussed bridge

$L$ Span (m)
$W$ Width (m)
$f_{1e}$ Experimental natural frequency of first bending modal shape
$f_{1e}$ Experimental natural frequency of first torsion modal shape
$f_{2e}$ Experimental natural frequency of second bending modal shape
$f_{2e}$ Experimental natural frequency of second torsion modal shape

The lower natural frequency corresponded to the first bending modal shape for the three bridges, being this the value to consider in the study of comfortability due to vibration caused by the pedestrian traffic.

The natural frequency of the first torsion modal shape was also studied because of its proximity to the natural frequency of the first bending modal shape.

Once the footbridges were tested experimentally, they were classified based on the risk of resonance due to the pedestrian traffic according to the Spanish standard for loads on road bridges IAP-2011, Table 5, and according to recommendations of SETRA, Table 6.

Table 5: Classification of the risk of resonance according to IAP-2011 [1]

<table>
<thead>
<tr>
<th>Type</th>
<th>$f_{0d}$ (Hz)</th>
<th>Risk of resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>&gt;4.60 Hz</td>
<td>No risk</td>
</tr>
<tr>
<td>B</td>
<td>&gt;4.60 Hz</td>
<td>No risk</td>
</tr>
<tr>
<td>C</td>
<td>&gt;4.60 Hz</td>
<td>No risk</td>
</tr>
</tbody>
</table>
3.2 NUMERICAL RESULTS

A numerical model was designed, using Robot software, to obtain the natural frequencies for the typologies A and B, that present a width of 2m. The values of the numerical natural frequencies and the relative error with respect to the experimental values are presented in Table 7.

Table 7: Numerical results of natural frequencies of the first and second modal shape in bending and torsion.

<table>
<thead>
<tr>
<th>T</th>
<th>f₁bn (Hz)</th>
<th>E (%)</th>
<th>f₁tn (Hz)</th>
<th>E (%)</th>
<th>f₂bn (Hz)</th>
<th>E (%)</th>
<th>f₂tn (Hz)</th>
<th>E (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>9.88</td>
<td>0.7</td>
<td>13.77</td>
<td>18.8</td>
<td>30.6</td>
<td>6.9</td>
<td>34.28</td>
<td>9.7</td>
</tr>
<tr>
<td>B</td>
<td>6.64</td>
<td>3.3</td>
<td>6.73</td>
<td>9.6</td>
<td>8.33</td>
<td>2.9</td>
<td>9.40</td>
<td>17.5</td>
</tr>
</tbody>
</table>

where,
- T: Typology of footbridge
- E: Relative error (%)
- $f_{1bn}$: Numerical natural frequency of first bending modal shape
- $f_{1tn}$: Numerical natural frequency of first torsion modal shape
- $f_{2bn}$: Numerical natural frequency of second bending modal shape
- $f_{2tn}$: Numerical natural frequency of second torsion modal shape

The numerical model has presented good results for the prediction of the natural frequency of the first bending mode, with a relative error less than 3.3%. On the determination of remaining frequencies, the relative error is not so low, with values, in all cases, lower than 18.8%.

New footbridges of several length were designed based on the typology A in order to study the effect of bridge span on the natural frequency. The bridges were calculated, according to Eurocode 5, to get the cross-section of main girders, keeping the same percentage of use in bending strengths than in the model A (87.5% with respect to 100%). Table 8 shows the results of the numerical frequencies for the typology A, considering span of 12, 16, 20 and 27 m for a width of 2 m.

The risk of resonance was evaluated for the new bridges designed, resulting in there is no risk of resonant due to vertical loads as the pedestrian crossing to the spans studied. In ease of the span of 27 m, the value of frequency for the first bending mode is close to the limit permitted by the IAP-2011. Therefore, for larger spans may involve risks of resonance for the typology of bridge A.
ACKNOWLEDGEMENT

This work was possible as a result of the financial support given by the Council of Education and Science of the Principality of Asturias (Research and Innovation Support Program for Companies for 2000-2012) and the Operative Program FEDER of the Principality of Asturias 2007-2013, which we thankfully acknowledge.

REFERENCES


A STUDY ON HORIZONTAL RESISTANCE MECHANISM OF WOODEN FLOOR WITH OPENING

Haruhiko Ogawa\textsuperscript{1}, Hisamitsu Kajikawa\textsuperscript{2}, Hiroyuki Noguchi\textsuperscript{3}

ABSTRACT: This study verifies whether sheathing area ratio method is applied to horizontal diaphragm in timber structure by finite element method analysis. Sheathing area ratio method is the evaluation method of strength and shear stiffness in shear wall of timber structure with opening which is proposed Sugiyama et al. As the result, shear strength and shear stiffness shows smaller by sheathing area ratio is larger. However, analysis results showed a slight variation for the sheathing area ratio.

KEYWORDS: Wooden floor with opening, Sheathing area ratio, Finite element method

1 INTRODUCTION

The current seismic design verifies structure with regarding horizontal diaphragm as established rigid floor assumption. However, sometimes horizontal diaphragm in timber structure can’t be regarded as established rigid floor assumption from the effect of type of opening (such as staircase and skylights, atrium, construction method, etc.). And there have been only a few studies on the evaluation method of strength and stiffness of the horizontal diaphragm.

This study verifies whether sheathing area ratio method is applied to horizontal diaphragm in timber structure by finite element method analysis. The parameter is form of opening and construction method (wood frame construction, wooden panels construction), pitch of a roof. Sheathing area ratio method is the evaluation method of strength and shear stiffness in shear wall of timber structure with opening which is proposed Sugiyama et al.\textsuperscript{[1]}

2 OUTLINE OF ANALYSIS

2.1 ANALYSIS MODELS

Figure 1 shows the outline of analysis model. Horizontal diaphragm is modeled in the plane elements (plywood) and spring elements (nail), the line elements (frame). At the end of the horizontal diaphragm is placed spring elements (wall) which is assumed shear wall. Figure 2 shows parameter and model name. Figure 3 shows form of opening. In addition, Figure 3 shows the sheathing area ratio (γ) proposed by Sugiyama et al. Parameter is form of opening of 26 types (A- to Z-type) and construction method of 2 types (wood frame construction, wooden panels construction), pitch of a roof of 4 types (0, 1/3, 1/2, 1). The total number of analysis models is 208. And, the model of wood panel construction is modeled as infinite in stiffness of nails.

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2.2 INPUT-DATA

Table 1 shows the input data used in the analysis.

Table 1: Input Data

<table>
<thead>
<tr>
<th>element</th>
<th>construction</th>
<th>range</th>
<th>module</th>
<th>size</th>
</tr>
</thead>
<tbody>
<tr>
<td>plane element</td>
<td>ZK4, AP</td>
<td>0°~20°</td>
<td>0.015</td>
<td>1500</td>
</tr>
<tr>
<td>spring element</td>
<td>wall</td>
<td>ZK4</td>
<td>0°~20°</td>
<td>0.015</td>
</tr>
</tbody>
</table>

2.3 ANALYSIS METHOD

Analysis method is incremental load analysis. Analyses is elasto-plastic analysis by changing the stiffness. Changing the characteristics of the displacement or shear stress in the range of degrees. The analysis was completed at the time beyond the range shown in Table 1.
Figure 4: Relationship between Shear Force and Deformation Angle
Figure 5: Deformation characteristics and In-plane shear stress distribution (nail joint type)

Figure 6: Deformation characteristics and In-plane shear stress distribution (adhesively-bonded joint type)
Figure 7: Relationship between shear strength ratio and sheathing area ratio at 1/600[rad.]

Figure 8: Relationship between shear strength ratio and sheathing area ratio at 1/150[rad.]

Figure 9: Relationship between shear strength ratio and sheathing area ratio at Qmax
2.4 ANALYSIS RESULT

2.4.1 The Relationship Between Shear Force and Shear Deformation Angle
Shows the relationship between shear force and deformation angle of each form of opening in Figure 4. Shear stiffness of horizontal diaphragm with opening is smaller by pitch of a roof and sheathing area ratio becomes larger. Shear strength is smaller by sheathing area ratio becomes larger. Shear strength of wood frame construction is larger by pitch of a roof becomes larger. And, shear strength of wooden panels construction show roughly the same value regardless of the difference of pitch of a roof. But the experiment of slope roof shows that shear strength is smaller by pitch of a roof becomes larger. Therefore, we consider that this analysis method is difficult to evaluate the shear strength.

2.4.2 Deformation Diagram and Degree of Stress Distribution
Deformation diagram and degree of stress distribution shown in figures 5 and 6 at the time of the 1/150rad.(pitch of a roof is flat.) Deformation is represented by 20 times. Deformation behavior is not seen a big difference in the difference of construction. Degree of plywood shear stress of wooden panels construction is very large compared with wood frame construction. In addition, the degree of shear stress near the opening, which shows a negative value as well as shear wall with opening[3].

3 SHEAR STIFFNESS AND SHEAR STRENGTH
Shows a comparison between sheathing area ratio method proposed by Sugiyama et al and analysis results of horizontal diagram with opening in Figure 7-9. Figure 7 shows the time 1/600rad. Figure 8 shows the time 1/150rad. Figure 9 shows the maximum load. According to Figure 7-9, shear strength and shear stiffness shows smaller by sheathing area ratio is larger. However, analysis results showed a slight variation for the sheathing area ratio. Possible reason for this is that, horizontal floor with opening is affected by the two-way sheathing area ratio, unlike in the case of bearing walls.

4 ADD-UP
Shear strength and shear stiffness shows smaller by sheathing area ratio is larger. However, analysis results showed a slight variation for the sheathing area ratio. As future work, proposal of evaluation method considering the effect of the two-way sheathing area ratio.

APPENDIX

\[ \gamma = \frac{1}{1 + \frac{\alpha}{\beta}} \]
\[ F = \frac{3}{8-5} \gamma \]
\[ \alpha = \frac{A_0}{H \times L} = \frac{\Sigma A_i}{H \times L} \]
\[ \beta = \frac{L_0}{L} = \frac{L-\Sigma L_i}{L} \]

Figure A: Sheathing area ratio method

REFERENCES
END-OF-LIFE WOOD QUALITY OF MOORING POLES.

Wolfgang Gard¹, Nadine Montaruli¹, Jan-Willem van de Kuilen¹²

ABSTRACT: The wood quality of an out-of-service timber mooring pole (wood species *Dicorynia guianensis*) has been assessed after 40 years in service. The resistance level of wood species against fungi determines predominantly the propagation of degradation of timber in hydraulic structures in the critical area around the water-line. Several tests have been performed in order to determine the residual strength and the natural durability. Results of the tests showed that the natural durability of the timber was still comparable with fresh timber. Also, the test results show that the mechanical properties have most probably not changed over time and are comparable with new basralocus. The results of this investigation can be used for decision making regarding possible re-use of structures in the same condition or increase the service life of same timber structures.

KEYWORDS: natural durability, decay tests, hydraulic structure, residual strength, basralocus

1 INTRODUCTION

The use of timber in marine structures gives satisfactory performance both in sea- and brackish water environments, particularly in construction elements continuously submerged below the lowest water level. In the Netherlands, hydraulic structures are often from tropical hardwoods. They are normally used for fenders, sheet pile walls, lock gates, mooring poles (Figure 1), etc. The most widely used wood species are azobe (*Lophira alata* ex Gaertn.f.), basralocus (*Dicorynia guianensis* Amsh.) and demerara greenheart (*Chlorocardium rodiei* Rohwer, Richter&Werff).

Due to economic and ecological reasons the service life of these marine structures has to be maximised by reliable assessment methods in order to predict residual strength and biological durability.

For example several thousand mooring poles (ca. 20 meter in length) are being replaced based on visual inspection every year just in the Netherlands [1]. The visual inspection criteria are not distinctive enough in order to predict the residual strength of the mooring poles. This research were initiated by the responsible Dutch ministry (RWS) to investigate the residual wood quality of the mooring poles which were scheduled to be replaced. The intention was to reconsider the initial service life of the poles.

Figure 1: Critical degradation zone (water-air) of Mooring pole Zwartsluis/Netherlands indicated by the arrow.

Strength properties of the poles can be seriously affected by biological, chemical, photochemical and mechanical degradation of the wood. In this investigation the wood quality has been assessed with respect to the residual biological durability and mechanical properties. Even if

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basralocus belongs to the most resistant wood species against fungi and marine borers, own inspections and research (not published) of structural members from basralocus in water have showed partially decay caused by fungi.

2 MATERIALS AND METHODS

2.1 MATERIAL

The mooring pole be investigated, had been in service for 40 years, exposed to harsh conditions in Dutch waters. The pole was considered for replacement. The wood basralocus (*Dicorynia guianensis* Amsh) originated from Suriname is assigned to durability class 2 according to EN 350-2 [2].

The pole had a cross section of 0.32x0.32 m² and a length of 18 m. Visually most decayed zone was the water-air line (Figure 1).

For investigation, the pole has been cut into blocks of 1 m length considering the air, water and soil zones (Figure 2 and 3).

**Figure 1:** Mooring pole blocks for investigation from Zwartsluis/Netherlands

Sampling

The cross section of the pole had been divided in different zones of heart- and sapwood along the radial direction, because after the service period a wider spread of wood quality was expected (Figure 3).

For compression tests 74 specimens were taken from the pole considering different vertical positions so that they covered all different zones of conditions (air, water, soil). Along the cross section, inner and outer heartwood zones were sampled (Figure 3). The dimensions of the specimens were 44x44x260 mm³.

For biological durability tests, about 670 specimens were prepared considering vertical positions and all cross section zones indicated in Figure 3. Depending on the test method, two sets of dimensions were made, 10x5x30 mm³ and 25x15x50 mm³.

2.2 METHODS

The natural durability of the pole has been assessed by 2 different tests, a standard method according the Technical Specification CEN/TS 15083-1 [3] and an accelerated method, originally developed by Bravery [4].

The first test method was used to determine the durability class of the basralocus pole. The second one was used to investigate the ‘resistance behaviour’ of the test specimens over a longer period up to 41 weeks of exposure to fungi. The basralocus test samples and the reference samples were exposed to two different fungi: the brown rot fungus *Coniophora puteana* (Schumacher ex Fries) Karsten (BAM Ebw. 15) and the white rot fungus *Coriolus versicolor* (Linnaeus) Quélet (CTB 863A). Basic test setup is shown in Figure 4.

**Figure 2:** Mooring pole blocks for investigation from Zwartsluis/Netherlands

**Figure 3:** Drawing (left) of the upper 5 m of the pole showing the sections used for the compression test (C), durability test (D). Distinct zones along the cross section (at the right), indicating inner-, intermediate- and outer heartwood and sapwood for testing.

**Figure 4:** Setup of durability test against fungi: CEN/TS 15083-1 (left) and accelerated test (miniblock test) developed by Bravery (right).

In addition the degradation mechanism was investigated with the Fourier Transform InfraRed (FT-IR) spectroscopy. It was applied at the most critical zone, the water-air line, in order to investigate if this zone had been degraded by fungi or other chemical actions. The FT-IR spectrometer used was Perkin Elmer Spectrum 100 fitted with the Universal Attenuated Total Reflectance (UATR) accessory. Each spectrum was collected with a resolution of 2 cm⁻¹. The wavenumber range covered by the instrument is from 600 cm⁻¹ to 4000 cm⁻¹.
Compression strength, modulus of elasticity and density of the specimens were determined following EN 408 [5].

3 RESULTS

3.1 BIOLOGICAL DURABILITY

The highest mass loss registered after 16 weeks was 21.9% for the inner heartwood zone when the samples were subjected to the attack of Coniophora puteana (brown rot). For all other sections the mass loss was less than 5%. The median mass loss for heartwood of the whole pole did not exceed 1.2% for both fungi (Figure 5). Therefore, the durability class of the timber tested may be assigned to durability class 1 (very durable), according to CEN/TS 15083-1.

![Figure 5: Median mass loss of basralocus and beech (reference) after 16 weeks exposure to two fungi types according to CEN/TS 15083-1. The median mass loss of beech lies outside the chart.](image)

Also the results from the acceleration test confirm the high grade of durability after the reference exposure time of 6 weeks (Figure 6). The threshold value being assigned to grade ‘very durable’ is a median mass loss of less than 5% according to CEN/TS 15083-1 [3].

![Figure 6: Median mass loss of basralocus over 290 days exposure to fungi according to the Bravery setup. Most values regarding mass loss for sapwood caused by white rot lie outside of the chart because of the high mass loss.](image)

During the extended accelerated decay test, the median mass loss of the basralocus test specimens from heartwood slowly increased up to 1.9% and 3.6% for the brown rot and white rot fungus respectively. Even prolonging the test duration for a remarkably longer period than commonly used in fungal decay tests, the mass loss recorded was still low enough to classify the tested timber as durability class 1. The mass loss for sapwood was considerably higher than for heartwood. After the exposure time of 290 days to white rot the median mass loss had reached 37%. Although the decay caused by brown rot showed the same tendency as white rot, after 290 days the median mass loss of the residual specimens didn’t follow. One of the reason could be explained by origin of the specimens position close to the heartwood, which could be affected by the gradient of higher natural durability.

FT-IR spectroscopy

The samples analysed with the infrared spectrometer were small chips cut from the inner part and the surface of the pole at the water-line level. Figure 7 shows the reference spectra and the spectra collected both in the inner part and on the surface of the section. The surface of the water-line zone presented a lower relative amount of carbohydrates. This suggest that, on the surface at the water-line level a soft-rot decay could have occurred. The relevant absorption peaks are, in this case, at 1050 cm\(^{-1}\), 1370 cm\(^{-1}\) and 1730 cm\(^{-1}\) (Figure 7). The first 2 peaks have been attributed to C-O stretching and to CH\(_2\) bending, respectively, in cellulose and hemicellulose, respectively. The absorption peak at 1730 cm\(^{-1}\) has been attributed to the C=O stretching in xylan.

![Figure 7: FT-IR spectra of samples cut from the inner part and the surface of the pole at the water-line.](image)

3.2 COMPRESSION STRENGTH

The compression strength and the modulus of elasticity in compression parallel to the grain (\(E_{c0}\)) have been measured using the set-up shown in Figure 8. To measure the modulus of elasticity, two extensometers with a length of 75 mm were attached to the specimens.
Figure 8: Set-up for the compression test (left). Specimens having defects (a) knot on the edge and cracks with discoloration (b) insect holes (c) knot on one face (right). Dimension of the specimens are 44x44x260 mm³.

The quality and degradation level of the specimens had a wide scatter, from almost clear to severe damages such as cracks (Figure 8).

The mean values and the standard deviations of the compression strength ($f_{c,0}$) were calculated for the different characteristic zones of the pole. They were found to be (62±5) N/mm², (55±8) N/mm² and (59±3) N/mm² for the air zone, the water-line and the soil-line respectively (Figure 9). The lowest value of $f_{c,0}$ with 34 N/mm² was recorded for a specimen cut from the water-line zone with a knot (Figure 8 right, c). The mean strength for the sample from the pole was 61 N/mm² with a standard deviation of 4 N/mm² and a 5-percentile value of 46 N/mm².

The mean $E_{c,0}$ for all specimens was 18000 N/mm² with a standard deviation of 1600 N/mm². The lowest value was 7600 N/mm² and the 5-percentile value was 13000 N/mm².

Figure 9: Mean compression strength ($f_{c,0}$) and modulus of elasticity ($E_{c,0}$) for the 3 areas of the pole. The error bars represent 2 times the standard deviation.

Density

The density values ranged from a minimum of 652 kg/m³ to a maximum of 813 kg/m³. The mean density over the whole cross section was 764 kg/m³ with a standard deviation of 29 kg/m³ at 0% wood moisture content or around 855 kg/m³ at the reference moisture content of 12%.

The minimum density was measured for the blocks around the pith. The latter was off-centered respect to the pole cross section, being located towards one corner at a distance of 80 mm from the center of the pole. The maximum density was registered for the blocks located in the outer part of the section, close to the sapwood.

4 CONCLUSION

- Natural durability of the wood
The results of the decay tests showed that the durability of the pole could be classified as very durable according to CEN/TS 15083-1, even after 40 years of service as a mooring pole. Relative long exposure of the miniblocks to fungi revealed that even under optimal conditions for fungal growth, the wood of this pole undergoes only very slow mass loss, compared with less durable or non-durable timbers such as beech.

- Degradation at the water-line
The infrared investigation indicated that only on the surface of the water-line zone chemical changes have occurred. The major mechanism of degradation in this zone could be explained as a combined effect of microbiological action and cavitation erosion.

- Mechanical properties
The results of the mechanical tests indicate that the timber after 40 years of service shows no significant strength reduction compared to new material. Even though the material did not allow for bending tests according to EN 408, the results of the compression tests indicate mechanical properties equivalent to at least strength class D60, with the density being close to the required value of 840 kg/m³, according to the EN 338 [6]. The strength values can be used for further evaluation of the residual strength of the pole with partly reduced cross section.

REFERENCES

MECHANICAL PROPERTIES OF REAL-SIZE BEAMS SAWN UP FROM SUGI (Cryptomeria japonica D.Don) CURVED LOGS

Shiro Aratake¹, Atushi Shiiba² and Hideki Morita³.

ABSTRACT: There are many sugi trees (Cryptomeria japonica D.Don) with considerable curves in Japan, but they have scarcely been used as the logs for building materials since the curves were considered to affect properties of lumber after sawing. From this background, the properties of sugi sawn lumber obtained from the curved logs were investigated to know if their qualities meet the need as a beam mainly based on the Japanese Agricultural Standard (JAS). For that purpose, bending test and creep test along with the measurement of warp due to sawing and drying were carried out. In the first place, 52 of curved logs with the maximum curved rate of 0.8-2.9% were picked out. Then they were cut into the size of about 120mm in width, 230mm in height, and 4000mm in length, and divided into two groups for green and dried condition with the same numbers (26 for each). The results were summarized as follows: As far as the logs with maximum warp of less than about 100mm in 4000mm length go (so-called “B log” among the relevant industries in Japan), there is almost no practical problem to apply the lumber obtained from the logs for beams considering the warp due to sawing and drying, bending properties, and creep behaviour including mechano-sorptive deflection during the desorption process. In terms of creep behaviour, it can be better to set the beams making the convex upward to reduce long-term deflections when they are used as a beam. What was remarkable in this case was the creep tendency was very stable regardless of the moisture content even when the lumber was in green condition.

KEYWORDS: Sugi, Curved logs, Beams, Bending performance, Bending creep

1 INTRODUCTION

In order to expand the demand of sugi (Cryptomeria japonica D.Don), which has been the leading species for constructions in Japan, promoting them for beams must be the most effective way since they have the highest share of structural members in Japan (beams: 28%, columns: 16%, sills: 7%, other construction materials: 49% [1]), while the percentage of domestic lumber for all beams has been merely one digit (5-7%) for many years [1]. However, in the existing circumstances, a large amount of curved logs has reportedly accounted for as much as 30% of all harvested logs [2]. This makes the situation difficult to promote sugi for beams since curved logs were considered to be far inferior in properties to the straight ones, whereas this idea was not necessarily based on experimental data. From this background, the purpose of this study is to clarify the properties of sugi sawn lumber obtained from the curved logs to know if their qualities meet the need as a beam mainly based on the Japanese Agricultural Standard (JAS). The experiments carried out were bending test and creep test along with the warp measurement after sawing and drying conducted beforehand.

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Figure 1: Sugi curved logs.
2 MATERIALS AND METHODS

2.1 SPECIMENS AND PROCEDURE

There is no united criterion for curved logs, but the ones with 1-2% at maximum curved rate are generally called “B log” and the others with over 2% at maximum curved rate are called “C log” in the relevant industries [2]. Based on this criterion, 52 logs with the range of 0.8-2.9% at maximum curved rate [the maximum warp to the log’s length (4000mm) was 31-116mm] were sorted for the experiment. The procedure of the experiment was as follows. (1) Measuring the properties of logs used for the experiment [diameter of bottom ends (Db) and that of tip ends (Dt), densities, average annual ring width (ARW), moduli of elasticity due to longitudinal vibration (Et), and maximum warp to length (MWL)]; those values are shown in Table 1. (2) Sawing the logs into the dimensions of about 120mm in width and 230mm in height. (3) Measuring densities, the maximum diameter of knots (K), ARW, Et and MWL. (4) Dividing them into the following 4 groups, (a) Green lumber loaded from the convex for warp direction of logs [in the case of creep test though, the side was for the warp direction of lumber itself. Following (b)-(d) are also the same], (b) Green lumber loaded from the concave for the warp direction of logs, (c) Dried lumber loaded from the convex for the warp direction of logs, (d) Dried lumber loaded from the concave for the warp direction of logs.

(5) Bending test for 11 pieces of each group (a) and (b) (total = 22), (6) 3 weeks’ Kiln-drying treatment with the condition of 90-120°C (DBT), 70-90°C (WBT) for 26 pieces of group (c) and (d), then measuring Et and other properties; those values along with the ones of green condition are shown in Table 1. (7) Bending test for 13

Table 1: The properties of sugi sample logs.

<table>
<thead>
<tr>
<th>D_b (cm)</th>
<th>D_t (cm)</th>
<th>Density (g/cm³)</th>
<th>ARW (mm)</th>
<th>E_t (kN/mm²)</th>
<th>MWL (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>41.5</td>
<td>31.9</td>
<td>0.678</td>
<td>4.70</td>
<td>5.21</td>
</tr>
<tr>
<td>SD</td>
<td>4.86</td>
<td>2.91</td>
<td>0.0721</td>
<td>0.860</td>
<td>0.813</td>
</tr>
</tbody>
</table>

D_b: Diameter of bottom ends, D_t: diameter of tip ends: ARW: Average annual ring widths, Et: moduli of elasticity due to longitudinal vibration, MWL: Maximum warp to length

Table 2: The properties of lumber obtained from the sample logs for the bending test.

<table>
<thead>
<tr>
<th>Loading direction</th>
<th>Density (g/cm³)</th>
<th>MC (%)</th>
<th>ARW (mm)</th>
<th>E_t (kN/mm²)</th>
<th>MWL (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dried lumber</td>
<td>0.386</td>
<td>27.5</td>
<td>14.7</td>
<td>5.93</td>
<td>5.90(5.03)</td>
</tr>
<tr>
<td>Green lumber</td>
<td>0.390</td>
<td>30.5</td>
<td>14.2</td>
<td>6.13</td>
<td>5.80(4.95)</td>
</tr>
<tr>
<td></td>
<td>0.719</td>
<td>27.5</td>
<td>133</td>
<td>5.83</td>
<td>5.09</td>
</tr>
<tr>
<td></td>
<td>0.675</td>
<td>25.8</td>
<td>123</td>
<td>6.24</td>
<td>5.11</td>
</tr>
</tbody>
</table>

: Loaded from the concave for the warp direction of logs, : Loaded from the convex for the warp direction of logs, K: Maximum diameter of knot, MC: Moisture content measured by oven drying method, ARW, E_t, MWL: See Table 1, Figures in the parentheses show E_t before drying.

3 RESULTS AND DISCUSSIONS

3.1 WARP BY SAWING AND DRYING

Figure 3 shows relationships between the MWL of logs and that of lumber before and after drying. From this figure, MWFs of lumber before drying were not that significant (in the left figure), while the values of lumber after drying become extremely high (in the right figure) when the MWL of logs is higher than about 100mm (2.5% at the maximum curved rate). Here, JAS stipulates the limit of maximum warp rate of beams, which must be lower than 0.5% for the Second Grade (20mm in this condition shown in Figure 3). On checking the results against this standard, only 2 pieces of lumber after drying, of which maximum warp at the state of logs were higher than 100mm (2.5% at the maximum curved rate), do not satisfy the Second Grade. This result shows that as far as “B log” go, there is almost no practical barrier.
Figure 3: Relationships between MWL of logs and that of lumber (before and after drying). JAS: Japanese Agricultural Standards, MWL: See Table 1

to pick out them for beams considering the warp caused by sawing and drying.

Figure 4 shows relationship between the MWL of lumber before and after drying. This figure gives us the practical information which shows MBL becomes roughly three times of green condition when the MC reaches around air-dried condition by drying treatment.

3.2 BENDING STRENGTH

Figure 5 shows the frequency of bending strength (MOR) for dried lumber and green one. According to this figure, the MOR of dried lumber is lower than that of green one (with the 5% level of significance). This means the drying treatment deteriorated the properties of lumber in this condition with lumber obtained from logs with significant warps. Usually, drying treatment increases mechanical properties on average, so this tendency does not necessarily be common at least in the case of general lumber in Japan. The following is the discussion mainly to infer the reason.

Figure 6 shows the frequency of MOR for the lumber loaded from concave ( ) and convex ( ) for the warp direction of logs. In this figure, it is noticeable that the distribution of MOR resembles to that in Figure 5, which suggest there are some links between the effect of drying treatment and the loading direction to concave or convex of lumber obtained from the curved logs. In particular, the two lowest values of MOR in both figures (Figure 5 and 6), one of which falls below the characteristic value for bending (22.2N/mm²) stipulated by Notification No.1452 of the Ministry of Construction in Japan, are the case when the lumber was dried and also loaded from the convex.

Figure 7 shows the relationship between the Eₐ at the stage of logs and MOR. This figure shows the lower the Eₐ at the stage of logs is, the lower the MOR of dried lumber becomes. And the lumber with the lowest MOR, which falls below the characteristic value, has been obtained from the log with the lowest Eₐ as shown in this figure.

Considering these results, curved logs with extremely low Young’s modulus, which seems to be less than about 4kN/mm², should not be sawn into the size of structural lumber, dried, and set up making the convex upward when it is used as a beam. However, only 1 out of 44
specimens which has the maximum warp of 100mm (2.5% at the maximum curved rate) at the stage of log does not satisfy the characteristic value.

This result shows that as far as the beams obtained from “B log” go, loading directions to the warp ones could be negligible considering the required bending strength.

Figure 7: Relationships between $E_i$ at the stage of logs and MOR. MOR: See Figure 5, $E_i$: See table 1 $r$: See Figure 4

### 3.3 BENDING CREEP

Figure 8 shows changes of deflection when the applied load was 685kg and moisture content measured by radio-frequency-type moisture meter (MC$_r$) with loading time. According to this figure, the creep behaviour of dried lumber and green one is fairly similar to each other until the first summer [in the case of green lumber, this time is nearly accordant with the one when the MC$_r$ reaches the fibre saturation point (FSP) as shown in the lower right in Figure 8]. Then for the dried lumber, this increase stops gradually and almost reaches so-called “creep cessation” as general tendencies (See the upper left in Figure 8). However for the green lumber, there is a conspicuous difference between the deflections when the beams were set making the convex upward (Convex direction) and the concave upward (Concave direction) from FSP (See the upper and lower right in Figure 8). The followings are the discussion mainly to infer this reason.

In the case of Concave direction in the upper right in Figure 8, the creep deflection extremely increases from FSP till air-dried condition (about 15%). This seems to be caused by mechano-sorptive phenomena [3] in the process of desorption, which is rather general when green lumber is used as a member [4]. On the other hand, in the case of Convex direction, this creep increase dwindles and then almost reaches “creep cessation” like the tendency of dried condition, which is extremely peculiar for the lumber in green condition, in the respect that mechano-sorptive phenomena in the process of desorption can hardly be recognised and there is no clear difference between the lines of dried lumber and this green one.

Figure 8: Changes of deflection when the applied load was 685kg and moisture content measured by radio-frequency-type moisture meter (MC$_r$) with loading time. MC$_r$: See Table 2

Figure 9 gives some clues why this peculiar phenomena in the case of Convex direction was induced for green lumber. This figure shows the changes of deflection when the applied load was 0 kg and MC$_r$ with time. In this figure, the deflection of the dried lumber with both loading directions (See the upper left in figure 9) and the green one loaded from the Concave direction (See the upper right in figure 9) are small or almost nothing. But in the case of Convex direction for green lumber (See the upper right in Figure 9), the deflection remarkably goes toward the recovery direction. In the end, the amount of recovery in Figure 9 (Load = 0kg) becomes almost the same as that of creep difference between Concave direction and Convex one in Figure 8 (Load = 685kg). From this result, the fact that the deflection almost reaches “creep cessation” in the case of Concave direction for the green lumber must have caused by the
fact that the upward deformation due to drying offset the downward mechano-sorptive deflection. This result shows the predominance of lumber sawn up from sugi curved logs in terms of long-term performance when it is used as a beam considering the loading direction. What is remarkable in this case is creep tendencies can be fairly stable regardless of the moisture content even if the beams are in green condition.

4 CONCLUSIONS
The result of this experiment shows there is almost no practical barrier to pick out curved logs (with less than about 2.5% curved rate) for beams considering the warp caused by sawing and drying, and required bending strength. And this result also shows that it can be better to set the beams making the convex upward when they are used as a beam to reduce long-term deflections. In this case, the creep curve was fairly stable regardless of the moisture content.

ACKNOWLEDGEMENT
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REFERENCES
EFFECTS OF THERMAL TREATMENT BASED ON HIGH-TEMPERATURE SETTING METHOD ON FRACTURE ENERGY OF WEDGE SPLITTING SPECIMEN MODELED

Koji MURATA¹, Yuji KURENUMA², Takato NAKANO³

ABSTRACT: Wedge-splitting tests were performed to obtain the fracture energy, and determine how thermal treatment affects the fracture toughness of a timber. Two sets of timber specimens were used: the first set to test the degradation of green wood caused by initial heating in saturated-humidity air and the second set to examine further degradation during the drying process in low-humidity air after initial heating. Air-dried spruce specimens were impregnated with sufficient distilled water to imitate green wood. As a result, the fracture toughness of almost all specimens was less than that of the control specimen. In particular, the fracture toughness of the heated wet specimens became lower with increasing process temperature. The specimens dried at 55 °C displayed the least toughness.

KEYWORDS: high-temperature setting method, wedge splitting test, fracture toughness, thermal decomposition.

1 INTRODUCTION

The high-temperature setting method may be put to practical use in Japanese sawmills for drying square timbers that include pith. Timber that include pith may develop surface splits during conventional kiln drying [1, 2]. In order to prevent splits and cracks, green timber is softened by heating and then rapidly dried at high temperature in low-humidity air, so as to generate compression stress on the surface [3]. It has been reported that the strength and toughness of timber dried by this new method is lower than timber subjected to conventional air drying [4]. The reduction in these mechanical properties may be due to thermal degradation during kiln drying, possibly as a result of both the initial heating and rapid drying at high temperature.

The high-temperature setting method has two processes: an initial steaming process before drying and a rapidly drying process under low-humidity air at high temperature. In the previous literature on the effects of kiln drying on strength, although heat treatment in the drying process has been discussed, degradation caused by heating wet timbers has not been discussed sufficiently. When a wood specimen is heated at high temperature, it becomes more brittle. It is said that wood is quasi-brittle material under stress perpendicular to the fibre direction, and we have focused on a stress-strain relationship with the strain-softening branch [5, 6]. The wedge-splitting test is thought to be suitable for analyzing strain-softening effects [7]. In this study, we performed wedge-splitting tests to obtain the fracture energy and determine which thermal treatment affects the degradation of the mechanical properties of timber.

2 EXPERIMENTAL

2.1 SPECIMEN

Air-dried spruce wood (Picea sp.) was prepared for the test. Spruce specimens were shaped by a numeric controlled router. The thickness of the specimen was 40 mm in the tangential direction, and the width was 80 mm in the radial direction. The curved air-dried specimens were impregnated with sufficient distilled water to imitate green wood.

Two sets of timber specimens were used: the first set to test the degradation caused by the initial heating process and the second set to examine the further degradation effects attributable to the so-called drying set process. A starter notch was made in the first batch of specimens before water impregnation, but the notch was only made in the second batch after the drying process to avoid stress concentration caused by kiln drying.

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³ Takato NAKANO, Graduate School of Agriculture, Kyoto University, Kitashirakawa oiwake-cho, Sakyo-ku, Kyoto, 606-8502, Japan.
2.2 HEAT TREATMENT

The wet specimens were heated for 12 h at one of the four temperatures (105 ºC, 85 ºC, 75 ºC, and 65 ºC) in saturated humidity air. The hottest wet air (105 ºC) was produced by a steamer (T-Fal 616070), and the others were heated in an environmental chamber (EYELA KCL-2000). For the drying set process, specimens were initially heated at 85 ºC and then dried for 12 h at four temperatures (120 ºC, 90 ºC, 75 ºC and 55 ºC) in low humidity air (33% of relative humidity). The two higher temperatures (120 ºC and 90 ºC) were produced in an oven drier (YAMATO DX602) with a water pan and the lower temperature (75 ºC and 55 ºC) were produced in an environmental chamber. Following this, specimens were slowly dried for three weeks to achieve an air-dried condition.

2.3 WEDGE-SPLITTING TEST

The fracture toughness of a specimen was defined as the fracture energy obtained by the wedge-splitting test (Figure 1). The angle of the wedge was of 17º. In this test, since the specimen did not split completely into two parts, the fracture energy could not be estimated by the fracture area and the total work required for splitting. Therefore, the fracture energy was estimated from the shift displacement of the strain distribution measured using the digital image correlation (DIC) technique in a manner similar to that of Matsumoto and Nairn [8]. For the strain analysis, both surfaces of the specimens were captured by a video camera (Imagine Source DMK41) during wedge splitting. The fracture toughness $G$ was calculated as

$$G = \frac{\Delta J}{B \Delta a}$$

where $\Delta a$ is the shift displacement of the strain distribution, $\Delta J$ is the work done in creating $\Delta a$, and $B$ is the width of the specimen. The horizontal component ($P_H$) of the splitting force and the crack opening displacement (COD) were calculated from the vertical component ($P_V$) of the load and the displacement of the crosshead, respectively. Because a slitting crack developed unstably using a clip gauge, the wedge-splitting test was performed without a clip gauge for COD. The work $\Delta J$ was obtained from the horizontal component ($P_H$) of the applied load and the COD.

2.4 CRACK BEGINNING

Elastic energy is stored in the specimen during the splitting test. When fracture toughness is estimated from the start of loading, the total work of the crosshead may include the elastic energy. In this analysis, the work to begin the crack was omitted in order to reduce adulteration of the elastic energy. The crack beginning was defined as follows. It is assumed that a decrease in stiffness is caused by micro cracks. The probability of the formation of a micro crack was defined as a function of $D(\delta)$, where $\delta$ indicates the crack opening displacement. The survival function $S(\delta)$ was defined as Equation (2):

$$S(\delta) = \frac{E [1 - D(\delta)]}{E}$$

(2)

Since the probability is assumed to obey a Weibull distribution, the scale and shape parameters were estimated using a double logarithm using Equation (3):

$$S(\delta) = \exp \left[ \left( \frac{\delta}{\theta} \right)^m \right]$$

(3)

where $\theta$ and $m$ indicate the scale and shape parameter, respectively. A change in the shape parameter related to the fracture mode was defined as a crack beginning (Figure 2).

3 RESULT AND DISCUSSION

3.1 $P_H$-COD CURVE

Examples of $P_H$-COD curves for wet heating treatment are shown in Figure 3. A gradual degreasing in the load was observed in this splitting test. The curve for the heated specimen was similar to that of the control specimen. Maximum load tended to decrease with
increasing temperature. In the drying set specimen, which was dried in low-humidity air, a similar curve was also obtained (Figure 4). Drying set specimens were initially heated in humid air at 85 °C. However, the maximum load tended to be larger than that of the control specimen.

3.2 CRACK BEGINNING

Figures 5 and 6 indicate relative stiffness at the crack beginning. The relative stiffness is the total stiffness divided by the elastic stiffness, which is in the linear region of the $P_H$-COD curve. There were two specimens for each condition. The error bars indicate the high and low value for the two specimens. The non-linear region, from the linear proportional limit to the crack beginning, was the same for the wet and dry specimens. The effect of temperature was not obvious from these data.
3.3 MAXIMUM LOAD

Figure 7 shows the changes in the relative maximum load which is the maximum load for the sample divided by that of the control specimen. ‘Wet heat’ indicates wet specimens heated in saturated-humidity air and ‘drying set’ indicates specimens dried in low-humidity air after initial wet heating at 85 °C. The maximum load of all wet specimens decreased with increasing temperature (solid circles). Since wet hemicelluloses decompose at 80–90 °C [9, 10], the tendency toward lower maximum loads in the heated wet specimens was probably due to thermal decomposition of hemicelluloses. Even though the maximum load was decreased by initial set heating, that of the drying set specimens was larger than that of the control specimen (open circles). The drying set process generated compression stress on the surface of the specimen [11, 12]. In the transverse tension test, the crack started at the edge of the wood specimen where tension stress was concentrated [5, 6]. It was thought that the compression stress in the surface increased the maximum load of the drying set specimen.

![Figure 7: Maximum load comparison](image)

3.4 FRACTURE TOUGHNESS

Figures 8 and 9 show the changes in the fracture toughness of the timber specimens measured using DIC. The fracture toughness of the wet heated specimen increased as the crack expanded (Figure 8). A fracture process area was observed in the stress concentration part of the wood specimen during the transverse tension test [6]. The increase in fracture toughness probably depended on the progress of the fracture process area. The fracture toughness of the drying set specimen decreased as the crack expanded (Figure 9). As stated above, compression stress was on the surface of the drying set specimen. The stress probably increased the initial toughness in the splitting test in Figure 9. In this test, the starter notch was not finished with a razor blade, and the notch did not have a sharp top edge. The fracture in this splitting test may not have been under the ideal conditions of mode I. The fracture toughness was found to be converged with shift displacement of strain distribution. The obtained data (G) were fitted by Equation (4), and the converged fracture toughness ($G_{\text{inf}}$) was defined as the fracture toughness of the specimen. The fitting was performed using the solver function of Microsoft Excel®.

$$G = G_{\text{inf}} \left(1 \pm \exp(-\omega a) \right), \hspace{1cm} (4)$$

where $a$ is the shift distribution of strain distribution, and $\omega$ is the coefficient for fitting. Fracture toughness $G_{\text{inf}}$ was divided by that of control specimen to obtain relative fracture toughness.

Figure 10 indicates the relationship between relative fracture toughness and heat-treatment temperature. The fracture toughness of almost all the specimens, with the exception of those heated at 65 °C in a wet condition, was less than that of the control specimen. This also seems to be due to the thermal decomposition of wet hemicelluloses [9, 10]. The effect of decomposition was larger than that of maximum load, and high-temperature treatment (105 °C) decreased the fracture toughness of
the wet specimen by 26% compared to the control specimen. All drying set specimens were initially heated at 85 °C in a wet condition. The fracture toughness of the specimen dried at higher than 85 °C was approximately 80% of the control. The specimen dried at 55 °C displayed the lowest fracture toughness which was 8% of the control. Since the glass transition temperature of wet lignin is approximately 60 °C [13, 14], this specimen was insufficiently softened to prevent splits and cracks from occurring on the surface.

4 CONCLUSIONS

The high-temperature setting method may be put to practical use in Japanese sawmills for drying square timbers that include pith. In this study, we performed wedge-splitting tests to obtain the fracture energy and determine how thermal treatment affects the fracture toughness of the timber in order to elucidate the degradation mechanics of high-temperature drying.

As a result, the fracture toughness of almost all the specimens was less than that of the control specimen. In particular, the fracture toughness of the wet specimen heated in saturated-humidity air became lower with increasing process temperature. This seemed to be due to the thermal decomposition of wet hemicelluloses, since wet hemicelluloses are thought to decompose at 80–90 °C. The specimen dried at 55 °C under low humidity displayed the lowest toughness of all. Since the glass transition temperature of wet lignin is approximately 60 °C, this specimen was probably insufficiently softened to prevent splits and cracks from occurring on the surface.

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REFERENCES


Microwave pretreatment of Norway spruce (Picea abies) for preservative treatment

Wilfried Beikircher¹, Christian Lux², Andreas Saxer³

ABSTRACT: The use of Norway spruce (Picea abies) is limited by its natural resistance against biological organism as insects and fungi. Thus the impregnation with preservatives would broaden the use of this most available wood species within Europe. Low permeability of Norway spruce does not allow for good impregnation with preservatives and fire retardant treatment. Some research results show that the microwave (MW) wood modification increases the permeability and improves the preservative distribution and uptake and other studies says that the effect could be neglected. This study should enlarge the knowledge about this topic.

The MW wood modification (pre drying) was carried out by using laboratory equipment. The effect of the MW modification will be compared on laboratory scale dried, commercial dried and natural dried wood. After drying the samples were impregnated with water and Potassium silicate and the uptake and the penetration depth was determined. The results show that the mild MW wood modification could not increase significantly the uptake and penetration depth of liquids.

KEYWORDS: microwave wood modification, timber, permeability, Picea abies

1 INTRODUCTION

The use of Norway spruce is limited by the low natural resistance against biological organism as insects and fungi. Therefore Norway spruce must be impregnated with preservatives to have a high durability. But the low permeability of this wood species does not allow adequate penetration of preservatives. MW modification of wood structure can increase the wood permeability [1, 2] and open new opportunities for Norway spruce. Other studies show that the effect of the microwave pretreatment on the moisture diffusion coefficient could be neglected [3, 4].

The target of this investigation was to use a mild MW process for avoiding mechanical losses and to get advantage of the higher impregnability as presented by [1, 2].

2 MATERIAL AND METHODS

2.1 MATERIAL

Freshly sawn Norway spruce from upper Austria was used for the investigations. The material was cut in the way to get twin samples for optimal comparison of the microwave drying effect. All drying samples had the dimension of 30 x 30 x 500 mm³. Due to the fresh material from the sawn mill the starting moisture content (mc) varied between 15 to 60 %. The mean value of the starting mc was 29 %. For the determination of the water uptake the samples were cutted and planed after the drying to the final dimension of 20 x 20 x 100 mm. For the determination of the Potassium silicate uptake the samples were prepared to 20 x 20 x 500 mm³. For the determination of the penetration depth the samples were cutted to 20 x 20 x 5 mm³.

2.2 METHODS

The methods applied were vacuum-microwave drying, microwave and laboratory oven drying under atmospheric pressure and industrial dried and natural dried material was used as reference material.
Experimental equipment
Drying experiments were carried out with a laboratory MW-drying equipment (see Figure 1). For the drying tests the first five Magnetrons of the MW-kiln were used. The laboratory plant is equipped with 12 spirally positioned magnetrons at the whole length of 3 m. Each magnetron has a maximum power of 800 W and the working frequency is 2.45 GHz.

During the drying the core temperature was measured with a fibre optic sensor (FOTEMP1 Fa. OPTOcon GmbH), which was inserted to the core of one of the samples and the surface temperature was measured with an infrared spectral pyrometer. The core temperature was used for the manual regulation of the intensity of the MW-power. The drying started by using 50% of the MW-power and then the power was manually regulated for holding the target core temperature. The core temperatures were regulated in the range of 100 ± 2 °C for the atmospheric drying process and 65 ± 2 °C for the vacuum drying process at 200 mbar.

For comparison industrial dried, natural dried and in a laboratory oven dried samples were used. The natural dried material was stored in a storage room at about 15 - 25° C until the mc of about 12% was reached. The drying steps for the laboratory oven were 24 hours with 50 °C then 24 hours with 80 °C and finally until the target mc was reached the temperature of 103°C were settled. For the industrial processes the target drying temperature was 65 °C and a vacuum of 200 mbar and 50 °C for the convectional drying under ambient pressure.

The different drying experiments were carried out to get final target moisture content of 12% and 0% (see Table 1).

After drying all samples were conditioned in a climate chamber at 20 °C and 65 % relative humidity prior to testing.

Determination of the liquid uptake and penetration depth
For the determination of the water uptake 12 samples (20 x 20 x 100 mm³) from each series were sealed with epoxy resin at the end sides and then were dipped for 48 hours in water at 20 °C. The absolute uptake of water was measured after 0,5, 1, 2, 4, 8, 24, and 48 hours. Additionally the water uptake was determined from the 12 twin samples in comparison to the previous samples, using a vacuum impregnation process. For this process similar prepared samples were given in an exsikkator and were evacuated for 30 min with a prevacuum of - 650 mbar (≈ 360 mbar). Afterwards water was inserted and the samples were dipped for 180 min and then the endvacuum of - 650 mbar was hold for 15 min as suggested in the research studies [5]. Before and at the end of the impregnation process the samples were weighted and the absolute water uptake was calculated by the mass difference.

For the determination of the uptake of Potassium silicate, K$_2$SiO$_3$ at a concentration of 75 %, 19-20 samples (20 x 20 x 500 mm³) were dipped for 72 hours. The solution uptake in g was measured from the difference of the mass before and after dipping. After dipping the samples were dried in the microwave to the mc before dipping. The penetration depth of the Potassium silicate was measured after storing the impregnated material in the clima chamber at 20 °C and 65 % rh until the equilibrium moisture content was reached. The penetration depth was measured from the tangential and radial direction in the middle of the 2 cm sample by using the pH-indicator neutral red. After changing the colour the penetration was determined with the Bresser LCD microscope using a 10x magnification (see Figure 2).

In order to observe, with Low vacuum scanning electron microscopy (LVEM), the information about the damage of pit structure on the cell wall, samples of MW dried (MWA_100°C_0%) and laboratory oven dried samples were used.
(TSA_100°C_0%) samples were prepared at the dimension of 20 x 20 x 30 mm³.

3 RESULTS AND CONCLUSION

The water submersion test of spruce samples show hardly any differences in water uptake compared to naturally or conventionally dried samples (see Figure 3 and Figure 4).

![Figure 3: Water uptake of MW dried Norvay spruce in comparison to conventional dried samples during dipping test.](image1)

![Figure 4: Uptake of MW- and conventional dried spruce wood under vacuum after 180 min.](image2)

![Figure 5: Potassium silicate uptake of Norvay spruce after 72 hours dipping](image3)

Even though the mean value from the MWV_65°C_12% - series show higher uptake of Potassim silicate the absolute uptake could not be significantly improved by the microwave drying as the standard deviation overlaps (see Figure 5).

The measurements of the penetration depth show similar results between the different pretreatments as the standard deviations overlap. No significant difference between the drying processes could be determined in this investigation see Figure 6.

![Figure 6: Penetration depth of Potassium silicate of Norvay spruce after different drying processes](image4)
In [3] was presented that the pith membrane structure from Masson Pine can be destroyed during microwave pretreatment. In this studies the pith membrane from Norvay spruce show no difference between the drying processes. Both pretreatments show similar damage effects on the pith membrane structure (see Figure 7 and Figure 8).

![Figure 7: LVEM photo of Norvay spruce showing the pith structure on the radial cell wall for microwave pretreated wood (MWA_100°C_0%)](image1)

In this investigation no significant improvement for the uptake of liquids from Norway spruce after MW drying in comparison to conventional dried material could be determined.

Regarding the results further investigations are necessary for improving the permeability of Norvay spruce. More intense microwave drying processes could possibly lead to major effects, but the wood should not be damaged due to the process. The use of an impregnation process by using a pressuring step should also be investigated.

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REFERENCES
CHARACTERIZING INFLUENCE OF LAMINATE CHARACTERISTICS ON ELASTIC PROPERTIES OF SINGLE LAYER IN CROSS LAMINATED TIMBER

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ABSTRACT: Cross laminated timber (CLT) has potential for load transfer in two directions when loaded in out of plane direction. In order to make effective use of this potential it is necessary to evaluate layer properties and their effects on the final product performance. This paper presents the approach and outcomes of an on-going study dealing with the evaluation of material and structural characteristics of components and their effects on overall CLT properties using modal testing. Laminate characteristics were first determined and “homogenized” single-layer plates were edge-glued. Single-layer panel characteristics were determined by modal tests. Measured natural frequencies and mode shapes were compared with finite element results. Relationships between laminate properties, like width and growth ring orientation, and single-layer elastic properties, like elastic and shear modulus, were found.

KEYWORDS: Cross laminated timber, Modal testing, Two-way bending, Elastic properties, Laminate aspect ratio

1 INTRODUCTION

In order to compete against other building materials, like steel and concrete, it is necessary for the timber industry to develop new engineered wood products. Cross laminated timber (CLT) is one of these new products. As a result of the alternating grain direction of the adjacent layers, CLT shows not only material dependent anisotropy, but also structural anisotropic behaviour. Due to their lay-up, CLT panels show potential for two-way resistance action and therefore an economical use in floor construction. Current design procedures for CLT under out-of-plane loading however are based on one-dimensional beam models. This does not utilize the full potential of CLT panels. Two-way plate models based on advanced laminated plate theory have been developed to predict normal and shear stress distributions, as well as deflection and natural frequencies of CLT panels under transverse loading. These models have the potential to be adopted for design use. However, a major challenge for using these advanced laminated plate models is the determination of appropriate input properties for individual layers, especially in the direction transverse to the grain of the laminates. This is because each layer is not a continuous plate. Rather, it is formed by connecting laminates edge-to-edge to form the layer. Influence of edge-gluing characteristics, width to thickness ratio of laminates (aspect ratio), and layer thickness will affect input properties. The objective of this on-going study is to develop relationships between laminate properties and layer elastic properties for input into two-way plate models. This paper presents some preliminary results of the study.

2 METHODOLOGY

2.1 MATERIAL AND GEOMETRICAL ASPECTS

Wooden boards, mainly spruce with various growth ring patterns were randomly selected and conditioned to a moisture content of about 13%. In order to maintain the achieved moisture content during further processing stages, the material is stored in a conditioning chamber with a constant climate. To facilitate further processing, all boards were sized to constant dimensions after the conditioning process has been completed. The boards were cut to a length of 1500mm, a width of 128mm and were planed to a thickness of 19mm.

2.2 EVALUATION OF LAMINATE CHARACTERISTICS

The modulus of elasticity (E) of the boards and their shear modulus (G) were determined by use of a modal testing technique as described in [1,2]. In order to obtain the elastic properties of the boards, the first and second natural frequencies in free-free support conditions were determined. Based on their natural frequencies, the board dimensions and density, the elastic modulus and the

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shear modulus were calculated. All boards were labelled and their dimensions, density, elastic properties and natural frequencies were documented.

2.3 LAMINATE GROUPING AND LAYER GLUING

The boards were sorted into different groups with similar characteristics, namely mean elastic properties (E and G values) and growth ring orientation. The boards were separated into three groups based on growth ring orientation: flat-sawn, quarter-sawn and boards with a growth ring orientation of about 45°. After grouping, the laminates were cut from the boards. In order to investigate the influence of laminate width on layer characteristics, three laminate widths were included: 120mm, 76mm and 32mm. The three laminate widths led to three laminate aspect ratios of approximately 2:1, 5:1 and 8:1. Boards with major defects were either excluded, exchanged or ripped to a smaller laminate width in order to allow removal of the defects or to distribute the defects over the final layer.

The laminates within each group were edge-glued by use of a structural polyurethane adhesive. The order of laminates within a layer was randomly chosen. To minimize surface distortion and cupping the laminates were edge-glued together with alternating pith location. In case of changes in the moisture content the alternating pith location of adjacent laminates leads to less surface distortion and cupping of the single-layer panel and therefore better dimensional stability. After glue application, a layer of laminates was clamped in a mechanical multi-jack press. The pressure of each jack can be controlled by an adjustable maximum torsional moment which is applied to threaded rods. The laminates were held down and aligned by moveable pneumatic dies while the pressure for the edge-gluing was applied. The layer gluing process was undertaken in two steps in order to prevent buckling and pop out of the laminates during edge clamping. First, two layers, half the final layer width, were edge-glued and then these two layers were edge-glued together to the final full width.

The selected characteristics for the grouping of the laminates, namely the elastic modulus, shear modulus, aspect ratio and growth ring pattern, provided the basis for the investigation of the influence of these laminate characteristics on the layer overall characteristics. Minimizing the variation of the selected properties within each group and therefore within each layer led to “homogenized” layers with similar laminate characteristics. After the gluing process the layers were re-sized to uniform dimensions. First they were sanded to a thickness of about 15.4mm, then cut to a width of 588mm and a length of 1220mm. In total 55 single-layers were produced, 27 representative single-layer panels were tested in this first step of the investigation. The group of selected single-layer panels included panels with 9 combinations of aspect ratio (3) and growth ring orientation (3).

2.4 EVALUATION OF LAYER CHARACTERISTICS

The 27 selected single-layer plate specimens were tested to determine their elastic parameters using modal tests developed by Sobue and Katoh [3] and Gsell et al [4]. The modal testing method described in [3] was initially developed for the determination of the orthotropic elastic constants of plywood boards. The elastic modulus in face grain direction, $E_{11}$, the elastic modulus perpendicular to the face grain direction, $E_{22}$, and the in-plane shear modulus $G_{12}$ are determined simultaneously by the determination of three natural frequencies. In the method, the plate-shaped specimen is vertically erected, simply supported along the bottom edge with the other three edges free. Simply supported boundary condition was achieved by clamping the specimen edge with two steel pipes. The test setup can be seen in Figure 1. The elastic properties were calculated using the equations given in [3] for the three selected frequencies. In this study the natural frequencies $f_{11}$, $f_{12}$ and $f_{31}$ were selected. In theory, any 3 natural frequencies can be used. However the sensitivity of calculated results is dependent on values of the elastic properties and specimen geometry. The natural frequencies used in the calculation were selected based on a sensitivity study.

In an attempt to evaluate the validity of the test method [3], a finite element (FE) modelling exercise was also conducted. The elastic properties measured using method proposed by [3] were used as input for a FE model as material properties. The FE model was designed to replicate the test setup of [3]. The single-layer panel was modelled as a shell element. The boundary condition of the test setup was simulated as a pinned support in the FE model. As shown in Figure 2, two single pinned supports are located at the vertical supports in the test setup, and a series of pinned support points along the supported edge are used to model the steel pipe support in the test setup. The natural frequencies of single-layer panels determined by laboratory tests were compared with natural frequencies calculated by a FE model.

The method described in [4] was first developed for the determination of elastic properties of CLT panels, although as for method [3] it is equally applicable to any orthotropic plate. In theory all nine elastic constants for orthotropic plate can be calculated using this approach.
However for CLT panels, the interested properties are the elastic moduli $E_{11}$ and $E_{22}$ and the shear moduli $G_{23}$, $G_{13}$ and $G_{12}$. The method is based on iterative calculation within an advanced plate theory model until the calculated natural frequencies and mode shapes match with the measurements from laboratory tests. Laboratory tests and plate theory model are based on free-free boundary conditions achieved by suspending the panels with strings in a vertical position. The physical test setup can be seen in Figure 3. At this stage of the current research project only the measurement of the natural frequencies and the related mode shapes of the single-layer panel was completed. The calculation of elastic properties using advanced plate model is yet to be performed.

As for the case of the Sobue and Katoh method [3] a FE model of the test setup was developed. The single-layer panel is modelled as a shell element, the free-free boundary conditions were achieved by two roller supports at the locations of the strings. The roller supports allow movement in direction 2 (minor axis) and 3 (out-of-plane) and restraints the in-plane movement in direction 1. The FE model can be seen in Figure 4. The measured natural frequencies and mode shapes were then compared with FE model predicted values based on material properties estimated using the method by [3] previously.

The two modal test setups were used in order to compare and verify the results for the single-layer elastic properties gained by the use of the method by [3]. The two setups were selected since their basic approach and their boundary conditions differed. To evaluate the influence of aspect ratio and growth ring orientation on the overall characteristics of single-layers, $E_{11}$ and $G_{12}$, are generally presented in this paper relative to the corresponding average elastic and shear modulus of all the laminates used in forming the “homogenized” layer, $E_{\text{average}}$ and $G_{\text{average}}$.

3 RESULTS AND DISCUSSION

A comparison of natural frequencies and mode shapes of the laboratory tests with ones calculated by FE models, using the characteristic properties determined by the method proposed by [3], shows good correlation in general. For the test setup with a simple support along one edge and free boundary conditions for the other edges [3], the three selected frequencies and mode shapes were compared with the ones from the related FE model. The deviation of the FE model based results to the laboratory results is less than 2% for frequency $f_{3,1}$, which is mainly
driven by $E_{11}$. The other frequencies $f_{1,1}$ and $f_{1,2}$, which are dominate by $G_{12}$ and $E_{22}$, show a maximum deviation of about 11%. For the free-free condition test setup based on [4] 14 frequencies and their corresponding mode shapes were calculated for each layer using the FE model. At least 10, generally 12 modes and their frequencies were identified from the laboratory tests of each specimen. The reasons for the difficulties in identifying all 14 modes are the closeness of adjacent modes in the spectrum, and the inevitable selection of an impact or response point that is close to the node of the mode shape of some of the modes. This raises the question of applying the test method to estimating CLT elastic properties. Nevertheless frequencies and mode shapes related to $E_{11}$ and $G_{12}$ show good correlation between laboratory test and FE model results. The deviation for $E_{11}$ related frequencies is less than 6%, while $G_{12}$ related frequencies show a maximum deviation of less than 10%. Only $E_{22}$ related frequencies show higher deviations.

Figure 5 shows the relationships between the laminate width and the increase of $E_{11}$ relative to $E_{\text{average}}$, and $G_{12}$ relative to $G_{\text{average}}$, respectively. The trend lines are also shown. Figure 6 shows the relationships between the laminate growth ring orientation and the increase of $E_{11}$ to relative to $E_{\text{average}}$, and $G_{12}$ relative to $G_{\text{average}}$, respectively, and the trend lines.

Although at first glance of Figure 5 and Figure 6, there is no clear trend regarding the influence of either laminate aspect ratio or growth ring orientation on elastic properties of single plate, some specific relationships between these elastic properties and laminate characteristics can be observed. Figure 7 shows selected $E_{11}$ results. It can be seen that almost all layers with a laminate width of 32mm show an increase in $E_{11}$ in relation to $E_{\text{average}}$. This could be explained by the stiffening effects caused by the extra glue lines in the plate with narrow laminates. A similar influence can be observed for layers with a growth ring orientation of about 45° irrespective of laminate width. The reason for this is unknown. On the contrary it appears that layers made from quarter-sawn laminates tend to show a reduction in $E_{11}$ in relation to $E_{\text{average}}$, except for the narrowest laminates. This would mean that there may be counter-acting effects of laminate width and growth ring orientation. All other laminate parameter combinations show less pronounced effects $E_{11}$. 

**Figure 5:** Influence of laminate width on elastic properties $E_{11} \& G_{12}$

**Figure 6:** Influence of growth ring orientation on elastic properties $E_{11} \& G_{12}$
Figure 7: Influence of laminate characteristics on $E_{11}$

Figure 8 shows selected results for $G_{12}$. It can be seen that all layers made by laminates with a width of 120mm show an increase in $G_{12}$ in relation to $G_{\text{average}}$. Layers produced from quarter-sawn laminates also show consistent increase in $G_{12}$ relative to $G_{\text{average}}$. This also applies to most of the single-layers made with flat-sawn laminates. All other combinations of investigated characteristics show less obvious trend on $G_{12}$ in relation to $G_{\text{average}}$. These results may indicate that wider laminates and quarter-sawn laminates could be beneficial in enhancing $G_{12}$ of CLT.

4 CONCLUSIONS

The small differences between measured frequencies and FE model predictions for the two test methods lead to the conclusion that property values for $E_{11}$ and $G_{12}$ of single-layer panels can be reliably estimated using modal testing method proposed by [3]. Further work is required to evaluate method proposed in [4]. From the comparison of the evaluated layer properties ($E_{11}$ and $G_{12}$) with averaged laminated properties ($E_{\text{average}}$ and $G_{\text{average}}$) for various combinations of laminate aspect ratio and growth ring orientation the following conclusions can be drawn:

- A smaller laminate width or aspect ratio lead to an increase in $E_{11}$ compared to $E_{\text{average}}$
- A growth ring orientation of about 45° leads to an increase in $E_{11}$ compared to $E_{\text{average}}$
- A quarter-sawn growth ring orientation leads to a reduction in $E_{11}$ compared to $E_{\text{average}}$
- Wider laminates or large aspect ratio lead to an increase in $G_{12}$ compared to $G_{\text{average}}$
- A quarter-sawn growth ring orientation leads to an increase in $G_{12}$ compared to $G_{\text{average}}$

The authors are aware that the amount of test samples is rather small at this point. The findings will be monitored during the analysis of the remaining single-layer panels and future research.

5 FURTHER RESEARCH

After the single-layer panel modal tests have been completed and analysed, selected single-layer panels will be tested in static tests to verify the results from the modal testing analysis. Static test will be conducted with different boundary (one- and two-way bending) conditions. The results from modal and static tests will be analysed with respect to the laminate grouping characteristics, elastic modulus ($E$), shear modulus ($G$) and laminate growth ring orientation. Their influence on the layer characteristics will be evaluated. It is expected that relationships between the different grouping characteristics and the single-layer properties can be derived.

After the single-layer panels have been analysed, scaled CLT panels will be formed by face gluing single-layer panels together. The formed CLT panels will be tested using modal testing methods [3,4] and static tests in order to evaluate their overall elastic properties and the influence of layer properties on CLT panel properties. Starting with 3-layer CLT panels the number of layers will be increased successively after modal and static test have been completed. In order to gain information about internal interlayer behaviour strain gauges will be employed within the interlayer sections at selected locations in some panels. Through this approach the relationship between laminate properties and characteristics on elastic properties of an edge-glued layer will be established. These layer properties will be used as input into an advanced laminate plate analytical model and finite element model to generate predicted
responses (deflection and stress distribution) caused by out-of-plane loading, and natural frequencies under the same boundary conditions as the modal tests. These predicted responses will be compared with measured responses from the static CLT plate tests to evaluate the validity of the advanced laminate plate model.

In addition to edge-glued CLT panels, CLT panels without edge-gluing will also be fabricated using the laminates from the same source. These CLT panels will be also tested in modal and static tests, and the results will be compared with those of edge-glued CLT panels. This would allow the influence of edge-gluing of laminates on CLT panel properties to be evaluated.

In order to gain more information about the applicability of the derived relationships on full scale CLT panels it is planned to carry out modal and static tests on full scaled CLT panels using a similar approach as for the scaled specimens. Samples of laminates will be obtained from the production line of an industry partner and tested to obtain model input properties. Full scale CLT panels will be produced and tested using modal testing method to evaluate their overall properties and to evaluate the validity of the derived relationships for full scale CLT panels.

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REFERENCES
MOISTURE INDUCED DEFORMATIONS IN GLULAM MEMBERS - EXPERIMENTS AND 3-D FINITE ELEMENT MODEL

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ABSTRACT. Experiments were performed on scaled spruce glued-laminated timber (glulam) bending specimens to observe time dependent development of deformations during drying and wetting. Measurements determined changes in the average moisture content and external shape and dimensions, between when specimens were placed into constant or variable climates and when they had responded to those climates. Alterations in the external shape and dimensions reflected changes in the average value and distribution of moisture and mechanosorptive creep in the glulam. Two experimental arrangements were used, with the first representing horizontal members with nominally zero external loading effects (i.e. supported underneath so that self-weight could not produce bending deformations), and the second arrangement representing horizontal members with external loading effects (self-weight plus external loads that cause bending deformation). Test results are being used to develop a sequentially-coupled three-dimensional hygrothermal Finite Element (FE) model for predicting temporally varying internal strains and external deformations of dried or wetted structural components. The model implies temporally varying and eventual steady state internal stress distributions in members, based on elastic and creep compliances that represent wood within glulam as a continuous orthotropic homogenised material. Thus, predictions are consistent with smeared engineering stress analysis methods, rather than being a physically correct analogue of how glulam behaves. This paper discusses limitations of and intended improvements to the FE model. Complementary investigations are underway to address other aspects of the hygrothermal behaviour of structural components of glulam and other materials (e.g. reinforced concrete) embedded within multi-storey hybrid building superstructures. The eventual goal is to create the capability to predict, and therefore be able to counteract, adverse deformations and material incompatibilities that can exist within hybrid building systems.

Keywords: Beam, creep, finite element model, glulam, heat transfer, hydrothermal strain, mechanosorptive strain, moisture.

1 INTRODUCTION

Glued-laminated timber (glulam) is an engineered wood product manufactured by gluing together timber laminates to create relatively large components for structural applications. Like any other structural parts, glulam elements are commonly exposed to wetting by liquid water or air humidity and drying by surrounding air. Usually in building applications direct exposure to liquid water is the result of preventable causes like faulty design, construction errors, lack of maintenance and spills. Therefore, focus here is what happens in well designed and constructed buildings within which glulam components are wetted or dried according to temporally varying moisture fluxes at surfaces of components that are exposed to air during and after construction [1, 2]. In such cases the primary external factors determining the current average moisture content and internal distribution of moisture in glulam are relative humidity (RH) and temperature (T) of surrounding air [3]. Temporal variations of RH and T of the air cause temporal variations of average moisture content ($u_{av}$) and gradients within glulam, and affect its physical and mechanical properties. Adverse consequence of initial drying of glulam after installation in a building, or moisture cyclic moisture fluxes in service, include component distortions and cracking at exposed surfaces. The distortions reflect development of three-dimensional strain gradients that in the extreme case of cracking have reached critical tensile strain values perpendicular to grain [4]. To be noted is that although for convenience engineering literature mostly discusses the phenomena in terms of stress levels and gradients, the physical local failure processes are driven by differential strains.

It is suggested in the literature that moisture related distortions of components should be taken into account during design of timber structures to avoid serviceability and safety related problems [4]. For example shrinkage and swelling of timber may cause damage to structural or non-structural parts of systems, including loss of...
integrity of connections in critical load paths [4]. Sundström et al [5] have shown that heat transfer and mass transfer have coupled effects on behaviours of wood-based construction materials under high-temperature conditions (as can occur during material manufacturing). However, in service temperatures and temperature gradients in glulam components are not likely to be severe, and coupling effect can be expected to be relatively weak. This permits assumption of weak coupling between heat transfer and mass transfer, as implemented by Baronas et al [6] in a two-dimensional numerical model of moisture transfer in timber during outdoor storage. In that study temperature everywhere in the timber was assumed to be the same and to vary simultaneously with ambient conditions of surrounding air. Therefore, although the effect of temperature on moisture transfer at the surface was accounted for, moisture flux was the only boundary condition considered. Also, the two-dimensional nature of the representation meant that only moisture diffusion in the transverse direction was considered as a function of the porosity of timber, in the process smearing differences that exist in the moisture diffusion in radial and tangential directions that characterise the physical structure of timber, Figure 1.

Past above mentioned work by other investigations has demonstrated the importance and conceptual viability of modelling moisture induced deformations of timber construction materials like glulam. However, so far the models created remain preliminary in nature. The remainder of this paper presents initial ideas for a sequentially-coupled three-dimensional hygrothermal Finite Element (FE) model for deformation of glulam, and describes some initial experiments supporting creation of such a model.

2 THEORETICAL FRAMEWORK

Methods and discussion in this section are predicated on acceptability of engineering stress representations of glulam/timber as a homogenous continuum, and assuming use of softwood species to make glulam.

2.1 TIMBER AND GLULAM CHARACTERISTICS

Timber/glulam has directionally dependent physical and mechanical properties in directions that correspond to the arrangement of cells in tree stems, Figure 1 [7]. The anisotropy in the cross-sectional plane of stems is due to the growth in radial cell rows which determines unbroken cell wall lines called wood rays. Based on the concept of cylindrical orthotropic structure the radial direction radiates from the pith of any stem. The tangential direction lies normal to the radial direction and tangential to the swept radii that define growth rings in cross-sections. The longitudinal direction corresponds to the pith of any stem. Although trees are imperfect in geometry and other features, the associated true anisotropy is typically ignored. For the purposes of property definitions and for stress analysis it is often assumed that wood is rectilinearly orthotropic with radial, tangential and longitudinal directions being axes of material symmetry. Importance of ignoring the differences between actual tree anisotropy, cylindrical orthotropic structure and rectilinearly orthotropic structure varies from case to case, but can be important for materials like glulam [8]. Structural design codes for timber make even greater simplifications such as ignoring directionality of material behaviour in the radial-tangential (cross-section) plane and only distinguishing between properties parallel to grain and those perpendicular to grain [e.g. 9 ]. These factors relate, amongst other things, to selection of the way directionality in properties needs to be recognised in prediction of heat and mass transfer, distortions and development of damage in glulam components.

Moisture in wood components can be found as free water in the cell cavities and as bound water within the cell walls. When only bound water is present, the timber material exchanges moisture with its surroundings continually [10]. With change in the air humidity wood absorbs or desorbs moisture at unsealed surfaces to the surrounding air or to contacting porous materials. The relationship between the RH and T and moisture content of wood (u) at contact surfaces is a kinetic process that under normal building service conditions is characterised as isothermal sorption with either wood or air the adsorbent and adsorbate mediums. Rates of adsorption or desorption are influenced by the rate of air movement across surfaces of timber/glulam components and if conditions inside building do not approach still air conditions that is a factor to be considered. When RH of surrounding air approaches 100 percent for a sustained period u will reach the fibre-saturation point (FSP). For many wood species the FSP is about 28 percent MC. Conceptually the fibre saturation point defines the state beyond which any increase in u will begin to add only free water to cell cavities, i.e. the point where the amount of bound water held in cell walls is at maximum [11]. However, these are simple heuristic concepts and the true behaviour of timber is much more complex. The question from in an engineering perspective is always where the correct balance lies between simplicity of concepts and need to realistically model the true behaviour, and that is part of the contextual conundrum being addressed by the authors.

Figure 1: Principal axes of wood with respect to tree stem structure
2.2 MOISTURE MOVEMENT AND HEAT TRANSFER

Moisture transport in timber is a slow diffusion process that results in moisture and strain gradients. Rates of diffusion primarily depend on $u$, $T$ and density ($\rho$), with rate being higher parallel to the grain than perpendicular to grain [12]. Assuming that timber/glulam is a homogeneous and continuous material, Fick’s first law applies:

$$ q = -D \frac{dy}{dx} $$

(1)

where: $q$ is the diffusion flux (kg m$^{-2}$ s$^{-1}$), $D$ is the diffusion coefficient of the material (m$^2$ s$^{-1}$), $y$ is the moisture concentration (kg m$^{-3}$), and $x$ the position along the transport direction axes (m). Because when $u$ is less than the FSP physical and mechanical properties of timber change, moisture transport occurs as a non-steady state process according to Fick’s second law. Hence, the one dimensional formulation of the moisture diffusion equation is:

$$ \frac{du}{dt} = -\frac{d}{dx}D(u) \frac{du}{dx} $$

(2)

where moisture content $u$ is the weight of water as a percentage of the dry weight of wood, $D(u)$ is the moisture dependent diffusion coefficient (m$^2$ s$^{-1}$), and $t$ is time (s). At surfaces where moisture can pass between timber and air, equivalence can be assumed to exist between the moisture content of the material surface and the equilibrium moisture content (EMC) associated with RH and $T$. This leads to the boundary condition:

$$ u_{surf} = u_{air} $$

(3)

where $u_{surf}$ and $u_{air}$ are the moisture contents of the timber and air, accounting if appropriate for the circulation conditions of the air. This is called the Drichlet boundary condition [12]. However, the alternative Cauchy boundary condition that takes into account mass flux at a material surface is arguably more realistic:

$$ \frac{qn}{\rho} = s_u(U_{air} - U_{surf}) $$

(4)

where $q_n$ represent the moisture flux across the boundary, $\rho$ is the density of wood in absolute dry conditions (kg m$^{-3}$), $s_u$ is the emission coefficient of moisture on the surface (m s$^{-1}$), $s_u = 3.2 \times 10^8$ m s$^{-1}$ [12]. According to [13]:

$$ U_{air} = 0.01 \left( -T + 273.15 \right) \ln \left( 1 - h \right) \frac{1}{0.13 \left( 1 - T + 273.15 / 647.1 \right) \rho_u \tau} $$

where $T$ is in degrees Celsius and $h$ is the relative vapour pressure of the air.

A modified Fourier equation can be assumed to govern conductive temperature changes in wood:

$$ \rho c_T \frac{dT}{dt} = \nabla (\lambda_T \nabla K) $$

(5)

Where: $\nabla$ is the gradient operator, $\lambda_T$ the specific heat of wood, $\rho$ the density of wood, $c_T$ the specific heat capacity of the wood, $T$ the wood temperature (°C ). Effects of convective and radiant heat are normally neglected but can be important in some timber construction situations, and for processes like kiln drying of timber.

Above governing equations are commonly solved using numerical methods and based on finite element approximation representations of timber components [e.g. 13, 14]. In modelling presented later in this paper, equation (3) was adopted rather than equation (4) because moisture transfer resistance at glulam surfaces was not measured by the authors. Importance of this simplification will be evaluated later.

2.3 STRAIN AND DEFORMATIONS IN BEAMS

Under constant climatic conditions the total deformation of glulam beams consists mainly of elastic and creep deformation induced by mechanical forces like self-weight and external loads. The intensiveness of creep deformations depends on the temperature and relative humidity of the surrounding climate at any surfaces. If climatic conditions are not constant, i.e. surrounding RH and $T$ change, two phenomena occur. The first phenomenon is dilation associated with change in $u$, $u_{air}$, and glulam temperature, and distortion associated with internal gradients $u$ and glulam temperature. The second phenomenon is coupling effects of mechanical forces and changes in $u$, i.e. what are often termed mechanosorptive effects. Because of the temporal nature of the involved processes resulting strains and deformations are referred to as mechanosorptive creep [14, 15]. The coupled nature of physical processes in glulam components subjected to variable climates can be analysed based on prediction of effects of non-stationary heat and water transfers within them.

Total strain vector $\varepsilon_{total}$ can be divided into elastic strain $\varepsilon_e$, thermal induced strain $\varepsilon_T$, moisture induced strain $\varepsilon_u$, creep strain $\varepsilon_{cr}$, and mechanosorptive stain $\varepsilon_{ms}$:

$$ \varepsilon_{total} = \varepsilon_e + \varepsilon_T + \varepsilon_u + \varepsilon_{cr} + \varepsilon_{ms} $$

(6)

The component $\varepsilon_e$ is creep strain that would occur under constant climatic conditions, and component $\varepsilon_{ms}$ is the incremental strain associated with climatic variations.

Following from Hooke’s law, the elastic strain is:

$$ \varepsilon_e = S_d \sigma $$

(7)

Where $\sigma$ is the stress vector and $S_d$ the compliance matrix.
Where in $E_r$, $E_t$ and $E_l$ are of elastic moduli of wood in radial, tangential and longitudinal directions respectively; $G_{rt}$, $G_{rl}$, $G_{lt}$ are shear moduli; and $v_{rt}$, $v_{rl}$ and $v_{lt}$ are Poisson’s ratios. Elastic moduli can seem to be assumed to proportional to the density, temperature, and $u$ of wood [7]:

$$E_i = E_{i,ref}[1 + \alpha_1(P - P_{ref}) + \alpha_2(K - K_{ref}) + \alpha_3(u - u_{ref})]$$

(9)

where $E_{i,ref}$ is the modulus of elasticity for direction $i$ at reference conditions of $P$, $K$ and $u$; and $\alpha_i$ are that adjust for unitary deviations from reference conditions of the variables that affect elastic moduli. The same logic applies to adjustment for $G_{ij}$ values from reference to actual conditions.

Thermal strain can be calculated as:

$$\varepsilon_T = \alpha_T \Delta K$$

(10)

where $\alpha_T$ is the thermal expansion coefficient vector, and $\Delta K$ is the change in temperature.

Moisture induced strain can be estimated by:

$$\varepsilon_u = \alpha_u \Delta u.$$  

(11)

where $\alpha_u$ is the dilatory coefficient vector, and $\Delta u$ is change of moisture content.

Creep strain can be calculated as:

$$\varepsilon_{cr} = \int_0^t S_{cr}(t) \frac{\partial \sigma(t)}{\partial t} dt$$

(12)

where $S_{cr}(t)$ is the creep compliance matrix, terms of which are calibrated by various methods based on experiments [14].

The mechanosorptive creep rate can be assumed to depend on the rate of change of RH and the stress level and the histories of those variables [14, 15], leading to:

$$\varepsilon_{ms} = \int_0^t S_{ms}(t) \frac{\partial u}{\partial t} dt$$

(13)

where $S_{ms}$ is the mechanosorptive compliance matrix. Again terms of the matrix are calibrated by various methods based on experiments, and it has been suggested that values in $S_{ms}$ should be conditional to $u$ [17].

Above equations follow the approach of Mirianon et al [14, 15]. The assumption of uncoupled affects and linearity of adjustments underpinning equations (6) to (14) are not believed to be crucial departures from actual behaviour of glulam installed in building having normal interior climates.

2.4 SHRINKAGE AND SWELLING OF WOOD

Moisture shrinkage/expansion coefficients of glulam depend on the positions and directions relative to the pith of tree stems from which laminations were cut, and the homogenizing effects of assembling laminations to create components. Coefficients depend on many factors including the wood species, the value of $u$ at which moisture change began, and the increment of moisture change involved. For spruce species, which are commonly used in Canada, and elsewhere, for making glulam, the average tangential shrinkage from the FSP to oven-dry is around 7 to 8%, while corresponding average radial shrinkage and longitudinal shrinkages are about 4% and between 0.1 to 0.2% respectively [7]. Such values can be used as the basis of the dilatory coefficient vector $\alpha_u$ in equation (11). Refinements are possible to recognize factors like departure of the shrinkage/expansion from linear dependence on the current value of $u$ and dependence on the moisture history by, for example, applying equation (11) piecewise.

3 EXPERIMENTS

As already indicated, experiments described here were designed to support the development of a three-dimensional finite element model capable of predicting temporally varying internal strains and external deformations of drying or wetted glulam structural elements with or without external applied forces. Those experiments were relatively simple and small scale, and are a precursor to complex experiments necessary to rigorously establish how glulam components interact with components of other materials commonly found in large hybrid building superstructures.

Eight scaled straight glulam members of 60 mm depth by 40 mm width by about 2.2 m long were made from 10 mm thick spruce laminates glued together with Phenol Resorcinol Formaldehyde (PRF) adhesive. The laminations were cut from commercially produced lumber that was nominally air dry ($u_{av}$ around 15 %). After fabrication specimens were stored in a conditioning chamber for four weeks at 20°C and 65% RH until they attained an EMC of about 12% prior to testing. This was to nominally simulate conditions in which full-size glulam components typically arrive at construction sites.

Two control specimens were kept in the constant environment (i.e. 12% EMC) and the six others were tested in a specially built environmental chamber. The special chamber has the capability to alter the climate ($T$...
and RH) of air surrounding specimens while they were supported in different ways and/or subjected to effects of external loads. Table 1 lists the combinations of test conditions investigated.

Table 1: Combination of test variables

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Loaded</th>
<th>Climate</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULC1, ULC2</td>
<td>No</td>
<td>Constant*</td>
</tr>
<tr>
<td>LC1, LC2</td>
<td>Yes**</td>
<td>Constant</td>
</tr>
<tr>
<td>ULV1, ULV2</td>
<td>No</td>
<td>Variable</td>
</tr>
<tr>
<td>LV1, LV2</td>
<td>Yes</td>
<td>Variable</td>
</tr>
</tbody>
</table>

* 20°C, 65% RH, ** As Figure 3.

Two experiment arrangements were employed. One arrangement represented horizontal members with nominally zero external loading effects, i.e. supported underneath so that self-weight could not produce bending deformations, Figure 2 (a). This situation determined the unloaded response of the scaled glulam under constant and variable climates. The other arrangements represented horizontal members with external loading effects, i.e. self-weight plus external loads that caused bending deformations, Figures 2(b) and 3. Magnitudes of the applied loads were self-weight of 11.06 N per m and concentrated forces of 50 N each (Figure 4). This resulted in a stress levels (circa 0.15 to 0.20 of ultimate capacity) comparable to those existing in superstructure systems controlled by deflections, as is expected for many large and tall hybrid buildings.

Specimens tested in a variable climate were subjected to cyclical variations in the RH of surrounding air at a constant air temperature of 20°C. Initially average daily relative humidity of the air varied from 55 to 95 percent based on a 1 week cycle length for 5 cycles. Then RH was then varied from 65 to 90 percent over one longer cycle of about 2 months, shown in Figure 4. Finally, specimens were allowed to equilibrate in a constant climate of 20°C and 68% RH. The logic of the chosen variable climate was to simulate climatic fluctuations that might occur inside heated buildings.

As appropriate, vertical displacements, and longitudinal and transverse dilatory deformations of specimens were measured every minute using electronic devices, as illustrated schematically in Figure 3. Average dilations were measures by strain gauges glued to specimens as shown in the figure. The RH of air in the chamber with variable climate was taken to be the average of measurements from four relative humidity probes at different locations within the chamber (with minimal variations observed between measuring locations). Air temperature was measured at specimen surfaces using four thermocouples as shown in Figure 3. Both RH and T were also recorded every minute. As shown in Figure 4, the actual air temperature in the chamber with a variable climate was slightly above the target value of 20°C. Figure 5 and 6 shows respectively selected total strain results for the two unloaded and loaded specimens in the variable climate. Those responses represent longitudinal strain at the tension face near mid-span, and perpendicular to grain strain at the mid-span. In the case of the perpendicular to grain strains the measuring direction was parallel to the depth. The windowed time interval encompassed the five weeks during which RH cycled between 65% to 90%, which was period of net wetting of the specimens. As previously indicated, ε_total contains a number of important components, that with the exception of ε_e are not separable without a supporting analytical framework/model. For that reason, it is not meaningful to draw any generalised conclusions about temporally varying strains and external deformations of glulam components.

![Figure 2](image-url)  
**Figure 2:** (a) Unloaded and (b) Loaded test arrangements in the variable climate

![Figure 4](image-url)  
**Figure 4:** Air temperature (T), relative humidity (RH) and glulam average moisture content (u_ave) variable climate experiments
Experiments done so far are not sufficient on their own for robust verification of any hydrothermal or creep response model. Instead, as previously alluded they are simply providing insights into approaches that need to be adopted during modelling. Test data was still being analysed at the time this paper was written and more results will be presented at the WCTE 2012.

4 FINITE ELEMENT ANALYSIS

Two classes of models appear in the literature for predicting deformation of timber/wood. The first class is an orthotropic continuum mechanics model that smears the effects of microstructure, and the second class is discrete mechanics models that realistically represent microstructural behaviour [15]. The approach taken here is the former as an implementation of concepts discussed in Section 2.

4.1 MODEL DESCRIPTION

The ABAQUS finite element software package [16] was used as the modelling medium for coupled mass diffusion analysis, consisting of a coupled temperature-displacement analysis followed by a mass diffusion analysis. ABAQUS was selected because it can perform sequentially coupled thermo-mechanical mass diffusion analysis. Material properties in Table 2 were selected based on assuming a three-dimensional rectilinear orthotropic material structure [16]. Diffusivity of mass was assumed to be the same in longitudinal and transverse directions, as an initial approach with recognized limitations.

Table 2: Assumed glulam material properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_u$</td>
<td>0.13</td>
</tr>
<tr>
<td>$D(u)$</td>
<td>$2.886 \times 10^{-7}$ m/s</td>
</tr>
<tr>
<td>$C_t$</td>
<td>$1170[15(1+T/100)]^{0.2}$ J/kg/°C.</td>
</tr>
<tr>
<td>$\rho$</td>
<td>450 kg/m$^3$</td>
</tr>
<tr>
<td>$E_l$</td>
<td>12 000 MPa</td>
</tr>
<tr>
<td>$E_T$</td>
<td>900 MPa</td>
</tr>
<tr>
<td>$G_{ll}$</td>
<td>500 MPa</td>
</tr>
<tr>
<td>$G_{tr}$</td>
<td>40 MPa</td>
</tr>
<tr>
<td>$G_{tt}$</td>
<td>700 MPa</td>
</tr>
<tr>
<td>$G_{tt}$</td>
<td>700 MPa</td>
</tr>
<tr>
<td>$\nu_{ll}$</td>
<td>0.55</td>
</tr>
<tr>
<td>$\nu_{tr}$</td>
<td>0.038</td>
</tr>
<tr>
<td>$\nu_{tt}$</td>
<td>0.015</td>
</tr>
</tbody>
</table>

FE mesh

The finite element mesh used to represent the scaled glulam members described in Section 3 is shown in Figure 7. Type C3D8T eight-node linear hexahedron thermally coupled brick elements with tri-linear displacement and temperature variations for coupled displacement and temperature analysis were used, with 4800 elements in total. This element was selected to exploit the similarity of governing equations for temperature and moisture transport, which enabled the governing constant for a temperature field to be replaced by the effective coefficient of diffusion for mass transport. Physical boundary conditions used in the analysis discussed here were simple end supports and concentrated loads shown in Figure 3. Boundary conditions for moisture transfer were that the glulam surface moisture flux and temperature matched temporally varying values measured during the variable climate experiments. Initial glulam moisture content and temperature were taken to be uniform and equal to 12 percent and 20°C respectively.
Equivalent stresses and temperatures from the temperature-displacement analysis were written to result files as averaged nodal and nodal values respectively. Subsequently those results were used during a mass diffusion analysis to provide ‘driving mechanisms’ for mass diffusion. The primary outputs were prediction of the evolution of moisture gradients, bending deflections and specimen dilations.

To note is that at present ABAQUS has no library modelling capabilities specific to time dependent analysis of wood components. It is possible for users to create their own material specific subroutine models, but so far the authors have not done so. What is reported here is therefore simply the first stage of analysing coupled mass diffusion analysis in glulam. One of the next steps will be for the authors to either write their own material specific subroutines or undertake that task in collaboration with international experts.

Currently identifiable deficiencies in the modelling include:

- Neglect of glue layers at interfaces between laminations.
- Neglect of systemic (e.g., growth rings, density) and random (e.g., knots, finger joints) material homogeneity and discontinuities (e.g. drying cracks) characteristic of commercial glulam.
- Neglect of directional dependence of mass diffusion relative to axes of material symmetry.
- Simplification of initial moisture and temperature conditions.
- Presumption that glulam constituent laminations have no memory of influences that altered their physical and mechanical responses prior to installation of glulam components in buildings. Such effects are widely disregarded but can be significant [17].

Addressing each of these, and other, model shortcomings will be considerable tasks.

4.2 PRELIMINARY RESULTS

Figure 8 shows typical predicted surface strains and deflection shapes, for a loaded specimen in a variable climate.

Figure 9: Bending displacement due to self weight and external loading

Figure 9 shows a typical bending deflection distribution obtained from FE modelling. Interestingly, it was found that the calculated mid-span elastic displacement was 1.19mm and the complete ABAQUS model prediction was 1.62mm. The difference in the values reflects the influence of creep strains. Preliminary comparison of FE predictions with experimental data gives some confidence in the modelling techniques. However, all FE model results presented here represented only the initial step of thermo mechanical analysis. Because of that, and given various uncertainties about the robustness of the model and the limited nature of the experiments to date, drawing other than the broadest of conclusions now would be less than circumspect.

Complementary investigations are underway to address other aspects of hydrothermal behaviour of structural members of glulam and other materials (e.g. reinforced concrete) embedded within superstructure frameworks of multi-storey hybrid buildings. The storyline here is therefore only the beginning of an epic technical adventure.

5 CONCLUSIONS

Even though what has been done so far by the authors and other pioneers is limited, it is clearly feasible to predict temporally varying internal strains and external deformations of drying or wetting glulam structural components via continuum finite element modelling techniques. The employed framework of sequentially-coupled three-dimensional hydrothermal modelling shows promise as the basis of a robust engineering tool for predicting, and therefore being able to counteract, adverse deformations and material incompatibilities that can exist within hybrid building systems.

ACKNOWLEDGEMENTS

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REFERENCES


BENDING PERFORMANCE OF SUGI (CRYPTOMERIA JAPONICA) LUMBER SAWN PARALLEL TO THE EDGE OF LARGE DIAMETER LOGS

Atsushi Shiiba¹, Shiro Aratake² and Hideki Morita³

ABSTRACT: Material characteristics of large diameter sugi logs grown in Miyazaki and flat squares sawn from them were investigated and their bending performances were examined. Modulus of elasticity from longitudinal vibration ($E_t$) of first logs was about 19% lower than that of second logs. However, modulus of rupture in bending (MOR) was only 2% different between lumber without pith sawn from first logs by taper rule sawing and lumber with pith sawn from second logs by center rule sawing, and the difference was not significant. In relationships between $E_t$ of logs and $E_t$ of flat squares, $E_t$ increased only 2% for lumber with pith obtained from second logs, while $E_t$ increased 25% for lumber without pith obtained from first logs. It shows that taper rule sawing is an effective method to improve the bending performances.

KEYWORDS: large diameter logs, sugi, taper rule sawing, bending performance

1 INTRODUCTION

Although production volume of sugi logs in Miyazaki prefecture is the highest in Japan, proportion of over 24 cm ones are quadruple of 20 years ago according to the research result of each diameter class. It shows that Miyazaki sugi logs grow in size at high speed due to climate of moist and humid. Meanwhile, domestic beams are merely less than 10% of lumber usage in conventional method of construction [1]. Therefore, it is very important to increase the usage proportion of beams. As for sugi structural lumber, it is often the case that lumber with pith is thought to be stronger than lumber without pith in relative market, and there is a difference in the price between both sides. It can be better to obtain 2 pieces of lumber without pith sawn from first logs considering yield ratio in this situation. However, there is an anxiety that degradation of strength with cross-grain and deformation after drying by using the center rule sawing which is current standard sawing method. On the other hand, one of the co-author showed that taper rule sawing could prevent degradation of strength since it was less affected by cross-grain in the case of lamina experiment [2].

In this study, material characteristics of large diameter sugi logs grown in Miyazaki and flat squares sawn from them were investigated. Based on the result, lumber without pith sawn from first logs by taper rule sawing and lumber with pith sawn from second logs by center rule sawing were examined, then their bending performances were compared.

2 EXPERIMENTAL METHOD

Sugi first and second logs grown in northern Miyazaki (diameter at bottom end ($D_b$) 33.3 ~ 50.0 cm, diameter at top end ($D_t$) 28.8 ~ 39.0 cm, length 430 cm, 40 years old) which number was 24 for both were used in experiment. First of all, $D_b$, $D_t$, length, weight, annual ring number and first natural frequency from longitudinal vibration ($F1$) were measured. Next, density in green state ($\rho_g$), $E_t$, annual ring width (ARW) and taper rate ($T_r$) were calculated. After that, 2 pieces of lumber without pith were sawn from first logs by taper rule sawing and 1 piece of lumber with pith was sawn from second logs by center rule sawing as shown in Fig.1; total numbers of them were 72.

[Figure 1: Sawing method of logs]

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³ Hideki Morita, Miyazaki Prefectural Wood Utilization Research Center, Miyakonojo 885-0037, Japan. Email: morita-hideki@pref.miyazaki.lg.jp
Then, they were kiln dried (hygrothermal condition: 90 ~120 degrees (DBT), 60~95 degrees (WBT), drying span: 10days) and they were shaped into width 105mm, height 210mm by planer. Finally, their size of parts, weight were measured and experimented on bending test. In experiment, actual size bending test machine (Tokyokoki WU-1000 TK21) was used, and experiment was conducted following “strength test method of structural lumber” edited Japan Housing and Wood Technology Center [3].

3 RESULT AND DISCUSSION

3.1 MATERIAL CHARACTERISTICS OF LOGS AND FLAT SQUARES

The properties of logs and flat squares are shown in Table 1. $E_t$ of first logs was about 19% lower than that of second logs, and 5% level of significance was showed between both sides. Then $T_r$ of first logs was over twice higher than that of second logs, and 5% level of significance was showed between both sides like $E_t$. MOE of lumber with pith was slightly higher than $E_t$ of lumber. Many squares were used for past similar experiment, and edge of ones are immature with low modulus of elasticity. In contrast to it, height of flat squares are higher than that of squares, and edge of those are mature with high modulus of elasticity. For this reason, it is thought that MOE is higher than $E_t$. In comparison between lumber with pith and lumber without pith, MOE of the former was about 8% higher than that of the latter. However, the difference between MOR and $E_t$ was only about 2%.

Table 1: The properties of logs and flat squares

<table>
<thead>
<tr>
<th>Logs</th>
<th>Avg. ($g/cm^3$)</th>
<th>$\rho_T$ ($g/cm^3$)</th>
<th>$T_r$ (mm)</th>
<th>$E_t$ (kN/mm²)</th>
<th>$\rho_A$ (g/cm³)</th>
<th>$g$ (kN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st logs</td>
<td>0.407</td>
<td>0.011</td>
<td>103</td>
<td>7.51</td>
<td>0.023</td>
<td>34.6</td>
</tr>
<tr>
<td>2nd logs</td>
<td>0.391</td>
<td>0.010</td>
<td>105</td>
<td>7.41</td>
<td>0.023</td>
<td>33.8</td>
</tr>
<tr>
<td>Total</td>
<td>0.396</td>
<td>0.011</td>
<td>104.5</td>
<td>7.45</td>
<td>0.023</td>
<td>34.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Flat squares</th>
<th>Avg. ($g/cm^3$)</th>
<th>$\rho_T$ ($g/cm^3$)</th>
<th>$T_r$ (mm)</th>
<th>$E_t$ (kN/mm²)</th>
<th>$\rho_A$ (g/cm³)</th>
<th>$g$ (kN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without pith</td>
<td>0.743</td>
<td>0.011</td>
<td>102.8</td>
<td>7.33</td>
<td>0.023</td>
<td>32.8</td>
</tr>
<tr>
<td>Lumber with pith</td>
<td>0.731</td>
<td>0.011</td>
<td>105.1</td>
<td>7.51</td>
<td>0.023</td>
<td>34.6</td>
</tr>
<tr>
<td>Total</td>
<td>0.737</td>
<td>0.011</td>
<td>104.45</td>
<td>7.45</td>
<td>0.023</td>
<td>34.1</td>
</tr>
</tbody>
</table>

3.2 RELATIONSHIPS BETWEEN MOR AND MOE FOR FLAT SQUARES

Relationships between MOR and MOE are shown in Fig.2. MOR is positively 1% level correlated with MOE though there is no clear difference between the lumber with pith and without pith. Then, only one of the lumber without pith (21.9N/mm²) was slightly less than characteristic value of normal structural lumber (22.2N/mm²). From this and preceding section result, there isn’t much difference between lumber without pith sawn from first logs and lumber with pith sawn from second logs for bending strength, and it is considered that both are basically equality.

3.3 RELATIONSHIPS BETWEEN $E_t$, $T_r$ FOR LOGS AND EACH FACTORS FOR FLAT SQUARES

Relationships between $E_t$ for logs and $E_t$, MOE for flat squares are shown in Fig.3 and Fig.4. For lumber with pith, $E_t$ is about 2%, MOE is about 3% higher than that of logs and slope of regression line close to 1. On the other hand, for lumber without pith, each differences are as much as 25% and 19%.

Figure 2: Relationships between MOR and MOE for flat squares

Figure 3: Relationships between $E_t$ of logs and MOE of flat squares

Relationships between $T_r$ for logs and MOR, MOE for flat squares are showed in Fig.5 and Fig.6. In relationships between $T_r$ and MOE, there is negative correlation of 5% level for lumber with pith, whereas there is no correlation for lumber without pith. In relationships between $T_r$ and MOR, there is no correlation for lumber without pith as MOE. However, there is negative correlation of 1% level for lumber with pith and the difference prominently manifested.
3.4 RELATIONSHIPS BETWEEN FLAT SQUARES, LOGS BY STRESS GRADING AND EACH FACTOR

Flat squares were graded according to the Japanese Agricultural Standard of lumber (JAS)\textsuperscript{[4]} based on the measurement result of MOE, then relationships between this result and crack of end grain section were investigated. Composition ratio of grade was shown in Table 2.

<table>
<thead>
<tr>
<th>Flat squares</th>
<th>E50</th>
<th>E70</th>
<th>E90</th>
</tr>
</thead>
<tbody>
<tr>
<td>First logs (Lumber without pith)</td>
<td>17%</td>
<td>58%</td>
<td>25%</td>
</tr>
<tr>
<td>Second logs (Lumber with pith)</td>
<td>8%</td>
<td>54%</td>
<td>38%</td>
</tr>
</tbody>
</table>

Relationships between MOR and crack length rate of end grain section are shown in Fig.7 and Fig.8. In these figures, test pieces were obtained from about 50cm from both specimens’ ends after bending test. The total crack length of end grain section were calculated, and this value was divided section area.\textsuperscript{[5]} Relationships between MOR and crack length rate of end grain section are unclear for lumber with pith. However, specimen without pith with lower crack length rate of end grain section broadly show the higher value of MOR for this lumber. But, the further research needs to be conducted because clear relationship wasn’t obtained from the compression test of post, which is examined at the same time.\textsuperscript{[6]}

These results show that large diameter logs which is able to be sawn into 2 lumber without pith can maximally bring out the performance by taper rule sawing, since this method can reduce the angle between axial direction and grain direction of flat square, and can reduce the loss of sapwood which mechanical strength is higher than heartwood. It shows that lumber without pith sawn from first logs possesses equal strength performance as lumber with pith sawn from second logs which can’t be eliminated the influence of cross-grain by the sawing method.

3.4 RELATIONSHIPS BETWEEN FLAT SQUARES, LOGS BY STRESS GRADING AND EACH FACTOR

Logs were graded according to the Japanese Agricultural Standard of material (JAS)\textsuperscript{[7]} based on the measurement result of $E_t$, then relationships between this result and $T_s$ were investigated. Composition ratio of grade was shown in Table 3.

<table>
<thead>
<tr>
<th>Logs</th>
<th>E50</th>
<th>E70</th>
<th>E90</th>
</tr>
</thead>
<tbody>
<tr>
<td>First logs (Lumber without pith)</td>
<td>58%</td>
<td>42%</td>
<td>-</td>
</tr>
<tr>
<td>Second logs (Lumber with pith)</td>
<td>4%</td>
<td>79%</td>
<td>17%</td>
</tr>
</tbody>
</table>
Figure 8: Relationships between MOR and crack length rate of end grain section (second logs: lumber with pith)

Relationships between MOR of flat squares and $T_r$ of logs are shown in Fig.9. In lumber with pith, on the whole, the lumber with higher $T_r$ has lower MOR. This tendency is remarkable in the case of low grade, while the number of specimen is a little small. On the other hand, in lumber without pith, lower grade show tendency to higher MOR of flat squares with $T_r$ of logs is high. From these results, taper rule sawing turned out to be effective for logs which have lower $E_t$ and higher $T_r$. And it is suggested that this method have potential to get good lumber with better quality.

Figure 9: Relationships between $T_r$ of logs and MOR of flat squares

4 CONCLUSIONS

Material characteristics of large diameter sugi logs grown in Miyazaki and flat squares sawn from them were investigated. At the same time, lumber without pith was sawn from first logs by taper rule sawing and lumber with pith sawn from second logs by center rule sawing were experimented, and their bending performances were compared. The followings are the results.

1. $E_t$ of first logs was about 19% lower than that of second logs. Then $T_r$ of first logs was over twice than that of second logs. As for flat squares, MOE of lumber with pith (second logs) was slightly higher than $E_t$ of lumber.

2. It is considered that lumber without pith sawn from first logs and lumber with pith sawn from second logs are basically equal because there isn’t much difference (no significant difference) between them for bending performance.

3. $E_t$ of flat squares was significantly higher than that of logs for lumber without pith (first logs). $T_r$ of logs is positively correlated with MOR and MOE of flat squares for lumber with pith (second logs). It was confirmed that taper rule sawing contributes to advancement of bending performances.

4. It was suggested that taper rule sawing is effective for logs which have lower $E_t$ and higher $T_r$.

ACKNOWLEDGEMENT

A part of the study was conducted with the support of Horizontal affiliation fabrication system encouragement project of local material in 2009 (Japan Federation of Wood-industry Associations), and Japan Society for the Promotion of Science Grants-in-Aid for Scientific Research (Grants-in-Aid for Scientific Research (C), No.22580191). Then, this experiment was conducted by significant collaboration from Koji Sato, Yuka Hagiwara and working staff belong to Mokumi Co., Ltd. We make a most cordial acknowledgement to them.

REFERENCES


DEVELOPING OF WOOD AND WOOD-ASH BASED HYDROXYAPATITE COMPOSITES: THEIR DURABILITY AND FIRE RETARDING PROPERTIES

Takeshi Akaki¹, Hironori Maehara² and Masayuki Tooyama³

ABSTRACT: In order to reutilize the waste wood ash emitted from woody biomass power plants, the Hydroxyapatite (HAp) solution was synthesized by reacting the wood ash with phosphate, and then the solution was impregnated into woods to make the wood-Hydroxyapatite composites. During the outdoor weathering test with 2 years exposure in full sunshine, the HAp composites show efficacy in retarding the color fading of woods. Also in fire-retarding test with cone calorimeter, the heat release rates and total heat releases of the HAp-composites were decreased with increased amount of the injected HAp solution. Although the amount of HAp injection in this trial was still not enough to keep fully effects as non-combustible wood composites, the HAp treatment have some effect for improving the fire retarding property of the treated woods.

KEYWORDS: Wood ash, Wood-mineral composites, Hydroxyapatite, Durability, Fire retarding, Cone Calorimeter

1. INTRODUCTION

For economic and environmental concerns, wood biomass has attracted a great deal of attention as a substitute of fossil fuel in the past decades. Burning of the wood fuel generates a heat and/or electrical power at biomass-combustion plants, but at the same time, enormous quantities of wood ash are excreted as waste by-products. At present, the wood ash is mostly treated as industrial waste and high disposal cost of them has pressed a management of biomass plants. The problem related to the disposal of wood ash is one of the most crucial factors that have been detrimental to the spread of wood biomass utilization; therefore the beneficial reuses of the wood ash directly lead to the zero-emission use of wood biomass fuel.

In previous studies, many developments for the wood ash recycling have been carried out. For example, wood ash is reusable for horticultural fertilizer and raw material of concrete [1]. Also new innovative attempts have been made in recent study to manufacture a composite board by combining the ash with wood particles and some adhesives [2]. And so in our study, the opportunity of another application of the wood ash was explored as a source of wood-inorganic mineral composites.

Wood ash is mainly composed of inorganic mineral components with unburned carbon. The first motivation of this study was to examine whether the wood ash could be put back and fixed into timbers again within the space of wood cells for the purpose of making the wood-inorganic mineral composites. In the previous studies, several wood-inorganic compound complexes have been developed with chemical agents in order to enhance the fire and decay resistances of the wood [3-6]. We therefore intend to produce a new composite material by combining the wood ash as mineral component within woods. However, the ash does not fit in so easily with woods because it has high alkaline and hence causes striking discoloration of the combined woods. So in our previous study [7], the ash was preliminarily converted into Hydroxyapatite (HAp; Ca₁₀(PO₄)₆(OH)₂) solution by a reaction with phosphoric acid, and then it was penetrated and fixed into woods in order to make the HAp-wood composites.

The HAp is well-known as eco-friendly biomaterials and widely applied for industrial products e.g. dental powder, bone-repairing or bone-replacing material. Since the HAp is water-insoluble material, it has a potential to be used as a fixative in wood preservative formulations. In the previous studies of wood treatment, the HAp was used as fixatives of the antibacterial metal ions within woods [8], and it was also synthesized and fixed on the surface of modified bamboo with the phosphorylation methods [9]. Depending on the other

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valuable point of the HAp implanting, it contains a phosphoric component like some kinds of fireproof agents; therefore by combining with the HAp, the treated woods would improve their fire retardance. For the purpose of evaluating these possible functionalities, the weathering resistant performance and fire retarding properties of the HAp-wood composites were examined in this study.

2 EXPERIMENTAL METHOD

2.1 WOOD ASH

Wood ash used in this experiment was obtained from a wood biomass power facility in Miyazaki, Japan. The sampling of the ash was performed on 25 August 2007. The collected sample was fly ash mainly produced by a burning of barks and mill ends of Japanese cedar (Cryptomeria japonica) in the furnace at around 600°C. It was recovered by an electrical dust collector and their grain size was around several tens of microns. According to the results in a previous study [7], the major element compositions of them were carbon (36 to 42 wt%), calcium (29 to 35 wt%), potassium (11 to 21 wt%) and sulfur (3.8 to 4.5 wt%) in estimated as oxidized compounds except for carbon based on a fluorescent X-ray spectroscopy analysis. In addition, the X-ray diffraction analysis revealed that the major inorganic mineral components of this ash sample were calcium compounds such as calcium oxide (CaO), calcium carbonate (CaCO3), calcium hydroxide (Ca(OH)2) and mineral dolomite (CaMg(CO3)2), as well as the noncalcareous salt, potassium chloride (KCl).

2.2 PREPARATION OF HAP SOLUTION

The procedures for formulating the wood ash-based HAP solution are as follows. The wood ash was dissolved with 1M acetic acid, and the charcoal particles in the solution were filtered to gain the saturated Ca2+ solution. The HAP solution was prepared via precipitation from the Ca2+ solution, 0.05M sodium dihydrate phosphate NaH2PO4 and 0.05M NaOH at pH 7 and Ca2+ : PO43- =1.67. The process was run in a glass vessel with magnetic stirring at room temperature.

After the treatment of wood specimens with that HAp suspension, several small sections of the treated specimens were prepared by cutting from them, and then the presence of the combining HAp microcrystals were observed by scanning electron microscope (Hitachi High Technology S-3000N, Japan). The observations were performed by mainly using secondary electron images under 15kV accelerating voltage.

2.3 OUTDOOR WEATHERING TEST

In order to estimate the weather resisting properties of HAp composites, the pile specimens 600×30×30mm (L, R, T) were prepared from five types of woods; sapwoods and heartwoods from Japanese cedar, Japanese cypress (Chamaecyparis obtusa), Yellow cedar (Chamaecyparis nootkatensis) and Spruce (Picea). Eight plies from each tree species were treated with HAp solution. The processing procedure was as follows. After oven-drying at 105°C for 1 week, the pile specimens were soaked into the HAp suspension and placed in a vacuum at 50 torr (6.67Pa) for 30min, subjected to pressurized air at 1.1MPa for 24hours. After impregnation, the specimens were removed from the HAp solution and dried under atmospheric conditions for 4 weeks and then oven-dried again at 105°C for 1 week until the specimens mass stayed constant. The dry HAp retentions from 5 types of wood specimens were shown in Table1.

In July 2009, these 8 HAp-impregnated piles and another 8 untreated ones from each wood species were planted to a half-length of the pile, 30cm depth at test area in Miyazaki Prefectural Wood Utilization Research Center, Miyazaki, Japan. These pile specimens have been stayed in full sunshine for the past 2 years, and their appearance and color changes were evaluated. A colorimeter (Nihon Denshoku Kogyo NF333, Japan) was used to quantify the color change at the surface of the specimens. Color was expressed using the CIE Lab (L*, a*, b*) system, and the color difference (ΔE*) between the initial and after 2 years exposure specimens was calculated according to Eq.1.

\[
\Delta E^* = \sqrt{((\Delta a*)^2 + (\Delta b*)^2 + (\Delta L*)^2)}
\]  

(1)

2.4 CONE CALORIMETER TEST

For evaluating the fire retarding properties of HAp-wood composites, test specimens 99×99×15mm (L, R, T) were prepared from sliced veneers of Japanese cedar and Japanese cypress. After oven-drying at 60°C for 72 hours, the specimens were treated with HAP solution in three ways as follows: (1) The HAP solution was simply spread over the surface of the specimens to coat them, and then the specimens were cured at room temperature for 4 weeks. (2) The specimens were soaked into the 100°C boiling-water for 2 hours, and then soaked into the HAP suspension for 24hours under atmospheric condition at 20°C. (3) The specimens were soaked into the HAP suspension and placed in a vacuum at 50 torr (6.67Pa) for 30min, subjected to pressurized air at 1.1MPa for 24hours. After the impregnation processes (2) and (3), the specimens were removed from the HAP solution and dried under atmospheric conditions for 4 weeks. After the HAP-coating or impregnating processes, the specimens were oven-dried again at 60°C for 1 week until the specimens mass stayed constant. The dry HAp retentions of the specimens were shown in Table2. These HAp-treated specimens and other untreated ones were stored in a temperature/humidity-controlled room at 20°C and RH of 65% for 4 weeks before the combustion test.

The cone calorimeter (Toyoseiki, Japan) was used for the fire-retardant experiment at Kumamoto University, Japan. Each specimen was wrapped in aluminium foil and exposed horizontally to an external heat flux of 50kW/m². A spark ignite was applied to ignite the specimens. During the test, oxygen concentration of the exhaust gas and sample weight were measured every 2s and the heat release rate (HRR) and total heat release (THR) were calculated.
3. RESULT AND DISCUSSION

3.1 WEATHRABILITY

Figure 1 shows the results of color measurements from the Japanese cedar sapwood piles. The date sets of both untreated and HAp-treated specimens are plotted on the CIE L*a*b* system. Before the outdoor exposure, the color ranges of both date sets were nearly overlapped, and then after 2 years exposure, their color changed to a greyish and the date plots in Fig.1 were shifted toward the zero point with their color fading. Compared with the untreated specimens, the HAp treated specimens were less subject to fading at after 2 years exposure. The same tendency was denoted among other 4 wood species.

Table 3 shows the results of color changes obtained from the untreated specimens. The degrees of color change were different depending on the type of wood species, the calculated ΔE* values were over a range of 19.8 to 38.5. In contrast, Table 4 shows the measurement date from the HAp-treated specimens. The color of intact wood specimens were slightly changed by the HAp impregnation, and the color difference ΔE* between those before and after the HAp treatment were over a range of 6.6 to 9.1. Based on the observation of the HAp injected woods by using a scanning electron microscope, the numerous needle-like microcrystals sized in about 50 µm length were observed on their surface with the aggregative microcrystals (Figure 2). It implies that the color change induced by the HAp impregnation was attributed primarily to the occurrence of the HAp microcrystals on the surface of the composites. Then after the outdoor exposure for 2 years, their color changes were additionally progressed. However, in comparison among the same wood species, all date sets of the HAp composites show the smaller ΔE* values (10.0 to 18.8) relative to those of the untreated wood specimens. These results indicate that the HAp treatment has some efficacy in retarding the color fading of woods.
3.2 FIRE RETARDING PROPERTY

Heat release rates (HRR) and total heat release (THR) for the HAp composite samples are shown in Figures 3-5. Figure 3 and 4 show the HRRs of HAp composites from Japanese cedar and cypress specimens, respectively. It is obvious that there are 2 peaks in the HRR curves, which is closely related to its combustion properties. Also it is clear that the peaks of HRR are much decreased with increased amount of the HAp retentions, especially for the second peak from Japanese cedar cypress. In addition, the heat release rising of second peaks tend to be delayed with HAp increasing.

Figure 3: HRRs of HAp-Japanese cedar composites

Figure 4: HRRs of HAp-Japanese cypress composites

Figure 5 shows the THR at 600sec heating. With increased amount of the injected HAp solution, the heat release rates were decreased and the declining rate of total heat release is similar between the HAp composites with Japanese cedar and cypress. These results imply the fire retarding effect of the HAp treatment. However in this trial, the amount of HAp injection was not enough to keep fully effects as non-combustible wood composites, thus the improving of their HAp solution retentions is needed in future study.

4. CONCLUSIONS

HAp wood composites were manufactured by the injection of HAp suspension, which was made from the wood ash. As a result of the 2 years outdoor exposure test, it was revealed that the color fading of woods were delayed by the HAp treatment. Also from a fire retarding test with a cone calorimeter, it was indicated that the fire retardancy of HAp composites were enhanced with increased amount of the injected HAp solution, therefore the HAp treatment has some efficacy in retarding the combustion of woods.

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REFERENCES

A NEW ERA OF COMMERCIAL STRESS-GRADING
AS/NZS 1748:2011

Geoffrey N Boughton¹

ABSTRACT: This paper presents a new Australian / New Zealand product standard for machine stress-graded structural timber. The standard aims to deliver flexibility for producers by enabling them to use a range of different grading sensors and technologies to produce similar products. At the same time it aims to ensure that all complying products will be compatible with assumptions made in the development of the timber design standards.

Prior to commercial use of the Standard, producers must undertake a qualification of the grading method, which demonstrates its ability to produce material with the required properties and performance and establishes parameters to be used in verification of the product. In commercial production, the qualified grading method must be used together with verification of structural properties of all products which are labelled as complying with this Standard.

KEYWORDS: Machine stress-grading, structural timber, verification of timber properties, grading method, qualification testing

1 INTRODUCTION

The majority of sawn structural timber in Australia and New Zealand is machine stress-graded – MGP grades in Australia, and MSG grades in New Zealand. This timber had been graded in accordance with AS/NZS 1748:2006 [1] which required that the timber be sorted primarily on the minimum local Modulus of Elasticity (MoE) of the piece. Historically, the timber was run through a mechanical stress-grader similar to that shown in Figure 1. Each piece was bent about its minor axis in order to estimate the local MoE and effect the grading.

Figure 1: Mechanical stress-grading

AS/NZS 1748:2006 [1] was written around only one type of grading technology, and that standard had been based on the results of many thousands of tests on timber prior to its first development. However, producers are keen to exploit the increasingly diverse range of grading machines and sensors now commercially available.

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AS/NZS 1748:2011 [2], [3] is a major revision of the original standard to enable flexibility in production of structural timber.

1.1 OBJECTIVES OF AS/NZS 1748:2011

The product standard defines a product that must be compatible with the requirements and assumptions of the design standards for structural timber [4], [5], [6], [7]. However, it must be couched in performance terms and allow producers to have flexibility in selecting and using grading technologies and combinations of grading technology.

2 PHILOSOPHY OF AS/NZS 1748:2011

There are two stages to the use of AS/NZS 1748:2011 [2], [3]. These two stages mirror the original development of the mechanical stress grading standard. Qualification of the grading method performs tests prior to commercial use of the grading method to validate the graded products and to establish the limits that are used in verification of properties once production has started. The daily use of AS/NZS 1748:2011 simply requires that the qualification of the grading method is current and that the properties (both structural and utility) of the complying products have been verified.

2.1 GENERAL REQUIREMENTS

These include:
- The grading method must have a current qualification certificate.
- Timber properties must be verified.
- Moisture content must be appropriate for the product – separate requirements for seasoned timber for use in Australia and for use in New Zealand.
- Utility measures, including dimensional requirements, straightness and features such as splits, want, and wane must comply.
- Products must be appropriately identified or marked.

### 2.2 Qualification of Grading Method

Part 2 of the suite of standards, AS/NZS 1748.2:2011 [3] contains the requirements for the qualification of the grading method. Qualification involves sampling, testing and analysis prior to commencement of commercial grading. Qualification applies to a specific grading method, suite of stress-grades and resource type. If there is a change in any of these, a repeat of at least part of the qualification is required.

### 2.3 Verification of Properties

AS/NZS 4490:2011 [8] is called up by the general requirements (AS/NZS 1748.1) and presents a number of options for verifying that the product has the required properties. The most commonly used of the verification techniques involves sampling from production, testing the sample and comparing the results with the target properties that are established during the qualification.

### 3 Qualification of Grading Method

The qualification report held by the manufacturer must relate to the current method of grading.
- Where the grading method changes, either by changing the measured grading parameters, or by changing the way in which they are combined, the qualification must be updated by reanalysing the original data to reflect the current method.
- Where new sensors are added, part of the testing must be repeated to assess the data from the new grading parameters.

In normal production, grade limits on all or any of the grading parameters can be changed without having to repeat any of the qualification.

### 3.1 Testing Graded Product

Phase I of qualification tests all of the properties on representative products. A representative small cross-section size and a representative large cross-section size are sampled in each stress-grade and then subjected to bending, tension, compression and shear testing.

In the Phase I analysis, the characteristic value for each property is compared with the characteristic value for design. It is a requirement that all properties meet or exceed the design value for all sizes and all grades. Phase I qualification requires the calculation of target values that have to be met in day-to-day verification of the completed products:
- Typically manufacturers test bending MoE and bending strength as part of verification, but MoE and tension strength can be chosen as an alternative. These are the indicator properties.
- Indicator Property Target Values are calculated during the Phase I analysis to ensure that meeting the acceptance criteria of verification will also indicate compliance of the untested structural properties (compression, shear, and either tension or bending strength).

### 3.2 Testing Run-of-Mill Production

Phase II of qualification establishes the link between the grading parameters and timber properties. Where timber properties are well correlated with grading parameters, the CoV of properties in each grade is within the assumptions made in the capacity factors used in design. In Phase II testing, the correlation between the indicator properties and at least one of the grading parameters is calculated to determine the coefficient of determination. A high correlation allows limits to be effectively adjusted to ensure compliance, and gives producers a higher degree of control over the properties of their products, allowing them to balance liability and profitability.

The sensitivity of the indicator properties to changes to the grading parameters may also be calculated. This measure enables producers to respond to changes indicated by verification of the properties.

Phase II sampling is drawn from the full range of material processed by the grading method and is not divided into stress-grades. During this sampling, data from all available grading parameters are collected for each piece.

### 4 Phase I Qualification Testing

#### 4.1 Characteristic Value

In the Phase I testing, characteristic values of each grade must be evaluated for all major structural properties in accordance with AS/NZS 4063.2 [9]. For generic stress grades, these evaluations include bending MoE, and bending, tension, compression and shear strength.

Each characteristic property must be greater than or equal to the design characteristic value for the grade. In this Phase, the analysis allows flexibility for grading methods under development by permitting thresholds to be adjusted if the characteristic value determined from the test result is less than the design characteristic value.

By comparing the ratio between the characteristic value from test data and the design characteristic value for each property, the limiting property (the one with the lowest ratio) can be identified. Grading limits can be adjusted to transfer enough pieces to another stress-grade to cause the limiting property to have a characteristic strength greater than the design characteristic value. Once the grading thresholds have been adjusted, this
redistribution is applied to all of the test samples of the stress grade.

4.2 Indicator Property Target Value

Qualification aims to establish the basis for demonstration of compliance of a product by verifying two properties only. The indicator properties are the two used in verification: MoE; and either bending or tension strength.

The structural properties may be grouped into two lists – List A and List B.

- List A properties – MoE, compression and shear strength – are better correlated with MoE than bending strength.
- List B properties – tension and bending strength – are better correlated with each other than any other properties.

The steps to determine the Indicator Property Target Values are:

- Find the ratio of the characteristic value from testing to the characteristic value for design for each property. The property with the lowest ratio in each list (List A and List B) will be the critical property.
- Use the upper confidence limit of the minimum ratio to establish the target value that must be achieved in verification testing.

The confidence limits of the minimum ratio in each list are given by Equations 1 and 2.

\[
\begin{align*}
\left[ r_{m,A} \left( 1 - k_s \frac{\text{CoV}}{\sqrt{R}} \right), r_{m,A} \left( 1 + k_s \frac{\text{CoV}}{\sqrt{R}} \right) \right] \\
\left[ r_{m,B} \left( 1 - k_s \frac{\text{CoV}}{\sqrt{R}} \right), r_{m,B} \left( 1 + k_s \frac{\text{CoV}}{\sqrt{R}} \right) \right]
\end{align*}
\]

for the List A minimum ratio

\[
\begin{align*}
\left[ r_{m,B} \left( 1 - k_s \frac{\text{CoV}}{\sqrt{R}} \right), r_{m,B} \left( 1 + k_s \frac{\text{CoV}}{\sqrt{R}} \right) \right]
\end{align*}
\]

for the List B minimum ratio.

Because the ratios are based on test results of a sample, the upper bound of the minimum ratio (which appears second in each of Equations 1 and 2) is used to adjust for any sampling error.

Hence for List A:

\[
\begin{align*}
\frac{R_{m,A}}{R} = r_{m,A} \times \left( 1 + 0.7 \frac{V_{k}}{V_{a}} \right)
\end{align*}
\]

with \( r_{m,A} \) as the minimum ratio amongst the List A properties.

0.7 as the \( k_s \) value used in establishing characteristic values for MoE.

\( V_{k} \) as the CoV for the MoE values found from the MoE characteristic value.

\( n \) as the number of specimens tested to give the MoE characteristic value.

And for List B:

\[
\begin{align*}
\frac{R_{m,B}}{R} = r_{m,B} \times \left( 1 + 1.17 \frac{V_{k}}{V_{a}} \right)
\end{align*}
\]

with \( r_{m,B} \) as the minimum ratio amongst the List B properties.

1.17 as the \( k_s \) value used in establishing characteristic values for strength values. (This is appropriate as the List B property will either be bending or tension strength.

\( V_{k} \) as the CoV for the strength property used as the List B verification property as determined in testing for its characteristic value.

\( n \) as the number of specimens tested to give the characteristic value from testing for the List B verification property.

The lowest value an Indicator Property Target Value can have is the relevant characteristic value for design. This will be the case if the indicator property is the critical property. However, if the indicator property is not the critical property, the Indicator Property Target Value must be higher than the characteristic value for design to ensure that even the weakest properties comply.

The characteristic value for design of the indicator property is scaled by the higher value of:

\[
\frac{n}{n-1}, \text{ the indicator property ratio divided by the upper limit of the minimum ratio, and 1.0.}
\]

The Indicator Property Target Values are used in production to verify that the product can demonstrate compliance with all required properties.

5 PHASE II QUALIFICATION TESTING

5.1 COEFFICIENT OF DETERMINATION

The coefficient of determination (\( R^2 \)) must be determined to establish the strength of the relationship between the indicator properties and the grading parameters. A better grading system will give a higher correlation between the properties and the parameters. This results in a lower CoV of properties within a grade. A lower CoV within a grade will give more reliable properties, allowing grading thresholds to be more finely calibrated to balance liability and profitability.

AS/NZS 1748.2:2011 [3] sets minimum values of coefficient of determination (\( R^2 \)) between at least one grading parameter and the indicator property that are compatible with suites of stress-grades for which the capacity factors used in design apply. These are:

- 0.6 for MoE (List A indicator property). MoE can be evaluated using a number of different non-destructive grading methods.
- 0.4 for the List B indicator property. Both bending and tension strengths can only be evaluated by destructive testing, so any non-destructive grading measurements can only predict this strength with a lower degree of accuracy.

Figure 2 shows an acceptable correlation between test MoE and one grading parameter, and Figure 3 shows an insufficient correlation. A grading method that measures both can claim qualification because \( R^2 \) for one grading parameter exceeds the acceptance criteria.
6 VERIFICATION

Verification monitors whether the product is meeting its Indicator Property Target Values which, by virtue of Phase I and II qualification testing and analysis, ensure that all properties meet their design value. AS/NZS 4490 [8] specifies the verification requirements.

6.1 VERIFICATION METHODS

Manufacturers may choose from the five available verification methods to suit their production requirements. The different methods provide the requisite confidence from a combination of test data and conservatism. Factors including the scale of production, availability of test facilities, flexibility, third party audit requirements, and the consequences of non-compliance may influence the manufacturer’s choice.

The verification methods are set out in Appendices A to E of AS/NZS 4490 [8]. Whichever method is selected, it must be checked annually using the same method detailed in AS/NZS 4490 [8]. This requires a sample drawn from the whole year’s production to check that the properties of the verified timber are as marked. If the check of verification method produces a pass result, then the verification method can continue for another year without any change. If the check of the verification method produces a fail result, then the verification method must be tightened.

6.2 VERIFICATION BY TESTING

Verification Method A (AS/NZS 4490 Appendix A) [8] is the most commonly used verification method. It involves random sampling out of each product and is suitable for use in mills in which a program of regular sampling and testing of product is part of quality control processes. It may be used either for batch monitoring or continuous monitoring.

Test data are used to estimate average MoE, 5%ile MoE, and 5%ile List B strength properties. These are then compared with Test Comparison Values (TCVs) to determine whether the timber properties comply.

TCVs are calculated from the Indicator Property Target Values determined in qualification using Equation 5.

\[
TCV = \frac{IPT}{1 + k_a V_n}
\]

with

- \( TCV \) as the Test Comparison Value
- \( IPT \) as the Indicator Property Target Value
- \( k_a \) as a confidence factor presented in [8]
- \( V \) as the coefficient of variation of the property
- \( n \) as the number of specimens in the test sample

Two different TCVs are calculated— for verified and not-verified status. The calculation of TCVs is based on the acceptable level of confidence in a particular property. If a test result is greater than or equal to the verified TCV, the product will be verified. If it is less than the not-verified TCV, it will not be verified. A value falling between the two TCVs indicates that the product is conditionally verified. Figure 4 shows the verified TCV and not-verified TCV in relation to the Indicator Property Target Value.

Figure 4 also shows an example in which the test data returned the property 20 MPa. This is higher than the verified TCV and the product is classified as verified. The probability distribution of the strength of the parent population from which the sample was drawn is shown as a continuous line. In this example, there is a very small probability that the strength of the parent population is less than the Indicator Property Target Value.

Figure 4: Test Comparison Values
Where test data is classified as conditionally verified, there is low confidence that it is above the Indicator Property Target Value. This is not reason to reject the timber if it occurs sporadically, but a number of consecutive conditionally verified results indicates a drift in properties towards non-compliance. Occasional batches that are classified as conditionally verified may be deemed to satisfy the requirements for verification. However, the third consecutive conditionally verified result is deemed to be not verified.

7 CONCLUSIONS

The new suite of product standards for solid structural timber in Australia and New Zealand focuses on product performance objectives. Flexibility in grading method has been allowed by requiring manufacturers to qualify their grading method prior to commencing production. No specification of grading method is included in the suite.

Qualification is specific to the grading method, stress grades (suite of properties), and resource type. It allows producers to optimise the grading method to the resource character and product mix prior to commencement of commercial production.

The Standard was published in 2011, and has already been implemented by a few mills. Many others are investing in more effective grading systems with a view to qualifying and commencing production in the near future.

8 REFERENCES

Changes of mechanical properties of Japanese larch square timber kiln-dried by the high-temperature setting method during various storages.

Takashi Takeda¹, Yoshihiro Hosoo², Takahisa Yoshida³

ABSTRACT: In Japan, the High-temperature setting (HTS) method, which is one of steaming kiln-drying methods, has rapidly spread on many saw mills for drying “pith-in” square timber. One of advantages of this method is short drying time and the other is reduction of occurrences of surface checks. Japanese larch timber kiln-dried by the method, however, is weaker than that by conventional drying methods. Then mechanical properties of timber were compared among various drying methods including the HTS method, and the effects of different storages on the properties were also investigated. Storage conditions might have an effect on changes of moisture content or Young’s modulus.

KEYWORDS: Kiln-drying, Air-seasoning, dynamic Young’s modulus, moisture content changes

1 INTRODUCTION

In Japan, the High-temperature setting (HTS) method [1], which is one of steaming kiln-drying methods, has rapidly spread on many saw mills for drying “pith-in” square timber. The feature of this method is that rapid drying of surface of green “pith-in” timber is occurred by high temperature and low humidity during initial drying stage. This rapid drying of surface generates tensile stress, and forms tension set at the surface. It is supposed that the tension set reduces surface checks of timber. The HTS is also called as “Surface setting method”.

By the HTS method, kiln-dried timber, however, might be weaker than those by the conventional kiln-drying method [2-3]. As an additional inconvenient situation, moisture changes through four seasons might effect on the internal stress distribution in cross section of timber constructed in wooden houses. In another words, storage method also might have an effect on mechanical properties of those materials after kiln-drying. Then, timbers dried by various drying methods were prepared, and they were under different storage methods for almost one year. We investigated on effects of storage methods on the mechanical properties.

2 EXPERIMENTS

The chief flow of the experiments is as follows:

1. Sorting green Japanese larch timber
2. Various kiln-drying, but air-seasoning for one group
3. Various storages of each material for almost one-year
4. Measurements of dynamic Young’s modulus before mouldering
5. Measurements of moisture gradients and static bending tests after moulding

2.1 MATERIALS

In the initial state, green Japanese larch square timbers were prepared. The 3m-long timbers were with the cross section of 135mm x 135mm. The total number of prepared timbers was 120, but 72 timbers were used for this experiments.

Dynamic Young’s modulus of each timber was measured by the tapping method for each timber. According to the measured values, the timber was sorted to four groups whose mean values were almost equal. Moisture contents were measured with the high-frequency-type moisture content meter (Kett Electric Laboratory, HM-520).

2.2 DRYING and STORAGE

The four groups were denoted as “HL”, “HS”, “M”, “AS”. “HL” and “HS” specimens were kiln-dried by the HTS method with dry bulb temperature of 120°C and wet bulb temperature of 90°C. The setting time was 48 hours for “HL”, and 18 hours for “HS”. “M” specimens were kiln-dried by the conventional steaming method, so-called “medium temperature drying”, whose dry bulb
temperature was under 100°C, and total drying time was 312 hours. “AS” means air-seasoning.

After kiln-drying, the timbers were under different three storage conditions: “Outdoor-exposure (Out)”, “Outdoor with roofs (R)”, and “Indoor (In)”. At the same time, “AS” specimens were also divided to the storage conditions. In the case of “Out”, six timbers were set on the supporting beams for each drying method, and the timbers were directly exposed to sun and rains. In the case of “R”, six timbers were set on the beams, and stacks were put on the timbers, and the next six specimens were in rows. On the top of the rows, corrugated tinplates were placed after four rows were up. In the case of “In”, the similar setting was done but no tinplates were placed on.

The photos of setting timber are shown in Fig. 1. In Fig. 1 (a), there were outdoor specimens in the front, and specimens with roofs were seen in the right depth. All specimens were stored at the Nagano Prefecture Research Center.

Before setting specimens, color differences were also measured with a color reader (Konica Minolta, CR-13).

2.3 MEASUREMENTS

2.3.1 Dynamic Young’s modulus
Before mouldering, dynamic Young’s modulus and moisture content were measured after almost one-year storage. At the same time, Moisture contents were measured with the MC meter. Color differences were also measured.

2.3.2 Moisture gradients
After the above measurements, the specimens were processed with a moulding machine to finish 120mm x 120mm cross section. Two blocks, whose depth is about 15mm, were cut from the timber at about 500mm apart from one end. One of them was oven-dried as the block shape, and the other was cut into seven pieces from the central strip as shown in Fig. 3. The central piece is double size of the other pieces. The small pieces were also oven-dried. Before and after oven-drying, the weight was measured for each block or pieces. In the following section, “Moisture content” denotes moisture content measured with the MC meter, and “MC” denotes moisture content by the oven-drying method.

2.3.3 Static bending tests
Four-point static bending tests were conducted according to ISO 13910:2005. The size of specimen for bending is 120mm x 120mm x 2400mm. The test span is 2160mm, and loading speed is 10mm/min. MOE, MOR, deflection at maximum load (Ym), and bending rupture work (W) were measured. The test equipment was shown in Fig. 4. The measured values were not adjusted by moisture contents for practical viewpoint since equilibrium moisture contents of timber kiln-dried by the HTS were lower than air-seasoned cases.

3 RESULTS AND DISCUSSION

3.1 MOISTURE CONTENT CHANGES
The results of moisture content were shown in Fig. 5. In the figure, “Initial” denotes moisture contents measured before drying, and “Before” and “After” denote “before or after storage”, respectively. The conditions were expressed the combination of drying method and storage condition.
Before storage, mean moisture content of “M” specimens was lowest among the drying conditions, but the differences decreased after storage. On the contrary, mean MC of “AS” was still higher than other specimens.

The MC changes during storage were shown in Fig. 6 except “AS”. In the figure, the MC changes between initial and final stage were presented for “AS” specimens. The increment of MC for outdoor condition seemed bigger than that for indoor condition. The brief explanations for the phenomenon should need further investigation.

3.2 DYNAMIC YOUNG’S MODULUS

The results of dynamic Young’s modulus (E) were shown in Fig. 7. For “Out” conditions, the E-values slightly increased during storage, but the values for “In” conditions slightly decreased except “M” specimens. There were no remarkable changes of E-values during storage, but there were slight differences among storages.

3.3 MOISTURE GRADIENTS

Final moisture contents (MC) measured by the oven-drying method were shown in Fig. 8. After moulding, the MCs were measured. For each drying condition, MC for “Out” was bigger than the other two storage methods. Compared among drying conditions, “AS” tended biggest among drying conditions.

The differences among storage methods for MC measured by the oven-drying method were more conspicuous compared with moisture contents measured with the MC meter as shown in Fig. 5. The MC meter is chiefly measuring moisture distribution near surfaces.
and moisture of the whole portion of cross section was detected by the oven-drying method. Then, moisture state of portion near surfaces for each specimen was almost equal, but moisture gradients of core portion should be different among storage methods. It should be noted that drying condition effects MC after one-year storage while relative humidity changes during storage. Moisture gradients for each storage methods were shown as Fig. 9 (a)-(c).

In Fig. 9(c), MCs were stable for “AS”, and there was no moisture gradient. For the other drying conditions, MCs at face and back were lower than MCs at other portions. The moisture gradients near surfaces remained after one-year storage. Among the drying conditions, “HS” was biggest, and “M” was smallest. For rafter storage in Fig. 9(b), the tendencies of moisture gradients seemed similar to “In” condition, but the values were bigger than MCs for “In” condition.

3.4 MECHANICAL PROPERTIES

The results of MOE were shown in Fig. 10. The differences among storage methods were small, and there were little differences among drying conditions.

Figure 10: Mechanical properties: MOE

Figure 11 shows MOR. There seemed little differences among storage methods, but “AS” was biggest among various drying conditions.

Figure 11: Mechanical properties: MOR

The deflections at maximum load (Ym) were shown in Fig. 12. The value for “AS-Out” was biggest around them, but variations of the values were not small. Ym for “HL” was smallest among them.

Figure 12: Mechanical properties: Ym
Similarly, bending rupture works (W) were shown in Fig. 13. The tendency in case of “W” was similar to “Ym” as shown in Fig. 12.

\[ W \text{ (N.m)} \]

Figure 13: Mechanical properties: W

The MOE values were almost equal among drying conditions or storage methods. For the other properties (MOR, Ym, W), there were small differences among storage methods, but the values were remarkably affected by drying conditions.

3.5 COLORS

Colors were measured by the L*\(a^*b^*\) color mixing system before and after storage. The results of L*, a*, and b* were shown in Fig. 14, 15, and 16, respectively. The colors in “Out” exposure went darker after storage for all drying conditions, but the changes of colors were very small for L*. The colors for “AS” were brighter than for “HL”.

\[ \text{Color differences (L*)} \]

Figure 14: Color differences before and after storage: L*

Similar tendency were observed for a* and b*. In Fig. 15, red components increased after storage for “AS”, the increment was remarkable for “In” storage.

\[ \text{Color differences (a*)} \]

Figure 15: Color differences before and after storage: a*

As a practical view points, final colors were one of the most interesting issues after surfacing process. Figure 17 showed the final colors after moulding. After moulding, the differences of colors among storage methods diminished for all drying conditions. For “AS” conditions, the colors seemed brighter, redder, and less yellow among drying conditions. The “HL” was vice versa. But the differences between them were small.
4 CONCLUSIONS

The results were listed as follows:

- The differences of moisture contents measured with the moisture content meter were small among storage methods, but moisture content measurements by the oven-drying method revealed the differences of moisture contents and moisture gradients among storage method.
- Dynamic Young’s modulus of the tested timber changed after storage, but the increment or decrement was small.
- MOE values among drying conditions were small, but there were differences of MOR, Ym, and W. The effects of storage on them were little, but HTS drying method might weaken the timber strength.
- Colors changed during storage period, but there were little differences of final colors after moulding.

ACKNOWLEDGEMENT

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REFERENCES


STRENGTH GRADING OF SLOVENIAN STRUCTURAL TIMBER

Tomaž Pazlar¹, Jelena Srpčič², Mitja Plos³, Goran Turk⁴

ABSTRACT: This paper presents some results of national research project titled “Strength grading of structural timber”. Project is dealing with introduction of grading methods for softwood species (spruce, fir) into Slovenian sawmills and with creation of extensive database containing the mechanical characteristics of sawn timber. The main aim of the project was a practical implementation of European standards into production of sawn timber finding the correlations between the indicative properties of Slovenian timber and grading characteristics. Relations of national research project with the European research project “GRADEWOOD” (WP3 – Experimental research), are also presented in the paper.

KEYWORDS: structural sawn timber, timber grading, non-destructive testing, characteristic bending strength, strength classes, EN 14081-1

1 INTRODUCTION

Round 60% of Slovenia is covered by forests and more than 4 m³ of wood per each resident per year is grown in Slovenian forests. However the exploitation of Slovenian timber is far beyond the potential: only half of year growth (app. 60000 m³) is cut and furthermore processed [1]. Regardless to the growing stock and increment the timber was not very popular construction material in Slovenia for many years. Its use – in terms of massive timber elements – was mostly limited on roof and floor elements. There were also few minor producers of glulam which – similar as all sawmills – mostly used their internal specification for timber grading. Those rules were in general combination of strength grading criteria based on old Yugoslav standards (JUS) and visual appearance grading [2]. On contrary with trends in Europe, the percentage of sawmills in Slovenia with capacity over 25000 m³ per year has in past decade decreased form 60 to 40 %. However the percentage of small local market oriented sawmills with capacity cca 10000 m³ per year has increased from 6 to 25 % [3]. This information is relevant when discussing the implementation of timber grading methods in practice. It is not reasonable to expect that such fragmented industry would be self-capable to update and accommodate the grading criteria to the one defined in the regulation and which are valid in whole European Union. Introduction of European standards should overcome the barriers between different internal grading criteria. However the introduction arises two important issues: what are the actual characteristics of Slovenian structural timber (spruce, fir) and are the characteristics of Slovenian timber comparable with characteristics of timber originated in “Central and Eastern Europe” growth area (as defined in European standards).

To clarify stated questions and to inform fragmented Slovenian sawmill industry about the principles of grading of structural timber, three Slovenian research organizations: Faculty for Civil and Geodetic Engineering, Department for Wood and Wood Technology of Biotechnical Faculty (both from University of Ljubljana), and Slovenian National Building and Civil Engineering Institute (ZAG, Ljubljana) initiated the project titled “Strength grading of Slovenian structural timber.” Our goal was to include in the industry oriented research project as much Slovenian sawmills as possible. Unfortunately the agreement with the Chamber of Commerce and Industry of Slovenia was not possible and the project was initiated with four biggest Slovenian sawmills.

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2 DETERMINATION OF PROPERTIES

2.1 INTRODUCTION
The project “Strength grading of Slovenian structural timber” started in spring 2009 and will be finished in spring 2012. Project is partly industry oriented: several workshops for industry partners concerning relevant topics were organized. All relevant standards (EN 14081-1, DIN 4074-1) were introduced, emphasizing visual and machine grading, factory production control, certification principles, etc. Visual grading principles were the main point of interest in workshops due to the fragmented sawmill industry in Slovenia and due to the fact that average share of production of structural timber is very low. The main part of the project was research oriented. Mechanical tests of timber elements were performed in order to determine the mechanical properties of Slovenian timber and furthermore to establish indicating properties for machine based grading. The cooperation with the Work Package 3 of European project GRADEWOOD offered us a unique opportunity to perform non-destructive tests on more than 1000 specimens.

2.2 SPECIMENS
The cross sections of specimens, sawn from previously selected logs from all parts of Slovenia (Figure 1), were adjusted to the proposed cross sections of elements in the GRADEWOOD project (Table 1). Additional rectangular specimens with dimensions 140 x 140 mm were included on proposal of one of industry partners who is focused mainly in production of solid timber elements.

Specimens were sawn from 4 m long spruce/fir logs with the mid diameter form 20 to 49 cm. Approximately 2/3 of logs were classified as class B and 1/3 as class C according to EN 1927:2008. Only one specimen was sawn from one log. Specimens were sawn at four sawmills – each industry partner took care for primary processing.

2.3 TESTS
Specimens were collected on one location: dimensions, mass, moisture content (by electric resistance), TKAR ratio and dynamic modulus of elasticity were measured. As result of cooperation with the WP3 of GRADEWOOD project specimens were then transported across Europe and tested with the following equipment: Brookhuis Timber Grader MTG, CBS TRIOMATIC, LuxScan E-Scan and CombiScan, MiCROTEC GoldenEye-706 and Viscan and Rosen Rosgrade.

<table>
<thead>
<tr>
<th>Cross sec.[mm]</th>
<th>Num.</th>
<th>Total num.</th>
<th>With pith [%]</th>
<th>Origin</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 x 100</td>
<td>119</td>
<td>70</td>
<td>500</td>
<td>Inner C. Carinthia Central S.</td>
</tr>
<tr>
<td>50 x 150</td>
<td>123</td>
<td>126</td>
<td>251</td>
<td>Inner C. Carinthia Central S.</td>
</tr>
<tr>
<td>44 x 210</td>
<td>61</td>
<td>249</td>
<td>21</td>
<td>Carinthia Central S.</td>
</tr>
<tr>
<td>140 x 140</td>
<td>74</td>
<td>74</td>
<td>89</td>
<td>Carinthia</td>
</tr>
</tbody>
</table>

Figure 1: Logs originated from Inner Carniola (A), Carinthia (B) and Central Slovenia (central part of Upper and Lower Carniola) (C and D) [4].

Figure 2: Bending test according to EN 408
2.4 TEST RESULTS

2.4.1 Density
The mean value and 5\textsuperscript{th} percentile value of wood density and clear wood density are presented in Table 2. Additionally – as required in standard – clear wood density was adjusted to 12 \% moisture content. The moisture content, estimated by the electrical resistance method was in the range from 10 to 16 \%, while the moisture content, measured by the oven-dry method was in the range from 9 \% to 14 \% (average value was 12 \%).

<table>
<thead>
<tr>
<th></th>
<th>$\rho_{\text{wood}}$</th>
<th>$\rho_{\text{clear}}$</th>
<th>$\rho_{\text{clear, 12% MC}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean value [kg/m\textsuperscript{3}]</td>
<td>458.6</td>
<td>443.7</td>
<td>444.1</td>
</tr>
<tr>
<td>5\textsuperscript{th} percentile value [kg/m\textsuperscript{3}]</td>
<td>390.8</td>
<td>373.4</td>
<td>373.8</td>
</tr>
</tbody>
</table>

Table 2: Wood density and clear wood density

Distribution of clear wood density for all specimens is presented in Figure 3.

2.4.2 Modulus of elasticity (MOE)
The mean value and 5\textsuperscript{th} percentile value of MOE were calculated from non adjusted measured values.
As presented in Figure 4 the distribution of the smallest group of samples (height 140 mm) is quite irregular due to the small number of samples. However the distributions all other groups of samples proved good matching with the expected distribution of MOE.

**Table 3: Local and global MOE**

<table>
<thead>
<tr>
<th></th>
<th>Local MOE (static bending)</th>
<th>Global MOE (static bending)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean value [N/mm²]</td>
<td>12173</td>
<td>11158</td>
</tr>
<tr>
<td>5th percentile value [N/mm²]</td>
<td>7532</td>
<td>7618</td>
</tr>
</tbody>
</table>

### 3 RESULTS ANALYSIS

Analysis of results of destructive and non destructive testing is presented in Table 5. High correlation values (R) indicate good correlation between quantities.

**Table 5: Correlation matrix between indicative properties**

<table>
<thead>
<tr>
<th></th>
<th>MOE\textsubscript{dyn,f}</th>
<th>MOE\textsubscript{dyn,us}</th>
<th>MOE\textsubscript{stat,gl}</th>
<th>ρ\textsubscript{clear w.}</th>
<th>MOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>MOE\textsubscript{dyn,f}</td>
<td>1</td>
<td>0.963</td>
<td>0.904</td>
<td>0.806</td>
<td>0.719</td>
</tr>
<tr>
<td>MOE\textsubscript{dyn,us}</td>
<td>0.963</td>
<td>1</td>
<td>0.868</td>
<td>0.834</td>
<td>0.642</td>
</tr>
<tr>
<td>MOE\textsubscript{stat,gl}</td>
<td>0.904</td>
<td>0.868</td>
<td>1</td>
<td>0.739</td>
<td>0.816</td>
</tr>
<tr>
<td>ρ\textsubscript{clear w.}</td>
<td>0.806</td>
<td>0.834</td>
<td>0.739</td>
<td>1</td>
<td>0.553</td>
</tr>
<tr>
<td>MOR</td>
<td>0.719</td>
<td>0.642</td>
<td>0.816</td>
<td>0.553</td>
<td>1</td>
</tr>
</tbody>
</table>

More information on test results can be found in [5], [6] and [8].

The correlation (R) between MOE calculated from simple non-destructive methods and with exact measurements according to EN 408 are also relatively high – above 0.8 – especially if taking into consideration the simplicity of presented non-destructive methods.

### 2.4.3 Bending strength (MOR)

The bending strength was – as required in EN 384 – adjusted to 150 mm height. The average adjusted value of all four groups of specimens is given in Table 4. Similar comment as given in previous chapter can also be given on distribution of MOR.

**Table 4: Adjusted bending strength**

<table>
<thead>
<tr>
<th></th>
<th>Adjusted bending strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean value [N/mm²]</td>
<td>43.28</td>
</tr>
<tr>
<td>5th percentile value [N/mm²]</td>
<td>23.74</td>
</tr>
</tbody>
</table>

**Figure 5: Relation between frequency dynamic MOE and adjusted MOR**
Correlation (R) of MOE obtained with different methods and MOR is different: it is high for MOEstat,gl (0.82) and lower for MOEdyn,us (0.64). The lowest correlation is obtained for density (0.55). This confirms that there is not a single indicating property which could be used for grading purposes.

As expected, density is not the most appropriate indicating property for predicting the MOR, but it is considered one of the properties which define the strength class of timber specimen.

Correlation (R) of MOE obtained with different methods and MOR is different: it is high for MOEstat,gl (0.82) and lower for MOEdyn,us (0.64). The lowest correlation is obtained for density (0.55). This confirms that there is not a single indicating property which could be used for grading purposes.

As expected, density is not the most appropriate indicating property for predicting the MOR, but it is considered one of the properties which define the strength class of timber specimen.

4 GRADING

Tested timber specimens were furthermore graded according to EN 338, taking into consideration three criteria: MOE, density (adjusted to 12% MC) and MOR (adjusted to the nominal height 150 mm). According to the EN 338 the MOE for the required class was reduced to 95% of origin value.

Optimum grading was performed as defined in EN 14081-2:2008. The optimum assignment of strength class to the tested boards is not trivial task because all three essential properties should be optimized simultaneously [6].

Different combination of strength classes can be used in optimum grading (Table 6). The most common combination originated from the visual grading and also from the old Yugoslav standard is combination C18-C24-C30. Within this combination 86% of specimens can be marked as C30.

If we take into consideration the visual grading standard, used in Slovenia (SIST DIN 4074-1), more than 4/5 can be sorted in the highest sorting grade S13. If we choose one most common strength grades, than 98 % of specimens can be sorted as C24. Even if we choose higher strength classes combination which can be identified only with machine grading (C35-C24-C30 and C40-C24-C18), the percentage of specimens in highest class (C35 or C40) is very high (71 % and 52%).

Table 6: Optimum grading

<table>
<thead>
<tr>
<th>%</th>
<th>C24</th>
<th>C24</th>
<th>C24</th>
<th>C24</th>
<th>C40</th>
<th>C40</th>
</tr>
</thead>
<tbody>
<tr>
<td>C40</td>
<td>52</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C35</td>
<td></td>
<td>71</td>
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</tr>
<tr>
<td>C30</td>
<td>86</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C24</td>
<td>98</td>
<td>3</td>
<td>14</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C18</td>
<td></td>
<td>7</td>
<td>9</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>rejected</td>
<td>2</td>
<td>4</td>
<td>6</td>
<td>8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The surprisingly good characteristics of Slovenian timber are somehow confirmed by parallel tension tests performed in the GRADEWOOD project. The characteristics strength and density values of Slovenian spruce are the highest one compared with results form Switzerland, Poland, Ukraine, Finland, Russia, Sweden, Romania, Slovakia and France. The details of this research can be found in [8].
5 CONCLUSIONS

Successful cooperation between industry and research organizations results in many benefits for all parties involved: After implementation European harmonized standard for sawn timber, some sawmills and producers of glulam can properly grade timber and mark their product with CE marking. As result of cooperation with GRADEWOOD project the machine settings for the machine grading of Slovenian spruce/fir are also available.

Presented research results and their analysis also confirmed that simple non destructive testing methods (especially acoustic emission) can be – in combination with measuring additional parameters/quantities (like density, knots, growth rings, etc) – efficiently used for grading structural timber.

The research results also confirmed that the strength, density and modulus of elasticity of Slovenian spruce/fir, compared to the countries of the “Central and Eastern Europe” growth area, are high enough to grade our softwood species according to European standard into comparable and – as indicated with presented results – even in higher strength classes.

ACKNOWLEDGEMENT

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REFERENCES

ANALYSIS OF DIFFERENT WEB-TO-FLANGE JOINTS OF WOOD I-JOISTS COMPOSITE

Wilson Fabiano Ribeiro¹, Maiko Cristian Sedoski², Jorge Luís Nunes de Góes³

ABSTRACT: The industrial process adds to products derived from wood features absent in solid wood, with more homogeneous structure and reduced defects, improved physical and mechanical properties, bio-deterioration resistance and better dimensional stability, improving quality and increasing the reliability of structures wood. The wood I-joists are manufactured by the industrial profiles bonding flange and web, formed of pieces of solid wood flanges and connected to the web consisting of OSB (Oriented Strand Board). A critical part of I-joist is the web-to-flange joint, the stiffness and strength is directly related to the material used and an efficient connection. The objective of this study is to evaluate experimentally four different geometries of web-to-flange joint with the aim of analyzing the most efficient connection. First was conducted a literature review, where then established four different geometries of web-to-flange joint: Rectangular section; wedge-shaped "V"; two parallel grooves; two grooves with a 4.6 degrees of tilt, and produced six specimens for each geometry. The adhesive used was the structural resin CASCOPHEN RS 216M. The rectangular and wedge-shaped "V" geometry had satisfactory results demonstrating higher strength and stiffness. The wedge-shaped "V" geometry had also a more practical assembly, facilitating the production.

KEYWORDS: Wood I-joists, Web-to-flange joints, Glued joints

1 INTRODUCTION

Production of sustainable construction forms a scenario where wood I-joist beam occupy a prominence place, because of the possibility of utilization of wood that comes from planted forests. Engineered wood products (EWPs) are industrial products which have the main features of sustainability, quality control, safety, efficiency structures, aesthetics, and various uses. One of engineered wood product that stood out are the wood I-joist, which can be produced from the junction of more than one wood composite, using the best properties of each one. Represent a major technological advances of EWPs and its main features, light weight, high stiffness and strength, reliability, low power consumption for its production and cost-effective when compared to solid wood beams.

2 BRIEF LITERATURE REVIEW

The wood has been widely used due to its high structural capacity, aesthetic value, great strength, low energy consumption for its production, an easily obtained materials and renewable source. The regeneration cycle, can easily exceed the volume that is being used. Regarding the reforestation trees, with increasing demand, lower value trees could be fully utilized if targeted for composite products.

2.1 WOOD I-JOISTS

One of engineered wood product that stood out are the wood I-joist, which can be produced from the junction of more than one wood composite, using the best properties of each one. The I-beams are composite structural members that are manufactured using sawn solid wood or structural composite lumber flanges and structural panel webs. The web-to-flange joint are bonded together with adhesives, forming the cross-sectional shape of an “I”.

Composite wood are becoming more prevalent in structural system, and they are expected to become even more important. Three hundred thirty million meters of I-beams are made annually [1]. In Brazil, the technology of the I-beams has been explored in the academic area with several studies; in the construction industry is still underutilized.

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³ Jorge Luís Nunes de Góes, Federal University of Technology – Paraná (UTFPR), Br 369, km 0,5, 271 Campo Mourão - Paraná. Email: jgoes@utfpr.edu.br
2.2 WEB-TO-FLANGE JOINT

A critical part of I-joist is the web-to-flange joint. Many different concepts have been introduced to ensure good shear performance, easy fabrication and gluing, improve the ability to transmit concentrated loads without crushing, guarantee of stiffness and strength, avoid splitting the flange stock when either the flange or web changes dimensions, and enhance product uniqueness. The connection by metal pins is not rigid, causing a reduction of the strength and stiffness, [2].

Due to the large development of new materials, building systems and structural systems, the studies of connections between structural elements have been important to the improvement in sizing, considering that the transmission of stresses and deformations in the structure are largely dependent upon the behaviour of these connections, [3].

The CANADIAN WOOD COUNCIL (CWC), [4] provides some models of web-to-flange joint and use of OSB or plywood for the web. Figure 1 details the geometry of connections.

3 MATERIALS AND METHODS

Through the literature reviewed was identified materials commonly used in I-beams, Pinus for flanges and OSB for the web.

3.1 USED MATERIALS

Was used Pinus Taeda in the flanges, as they are from reforested areas, low density, low cost, wide availability in this region and potential production in the country. For the web was used structural OSB Home Plus 9.5 mm thick manufactured by LP Brasil in southern Brazil. The adhesive used was the synthetic resin structural, based on Resorcinol-Formaldehyde CASCOPHEN RS 216M and hardener FM-60-M.

3.2 PUSH-OUT SHEAR TESTS

The objective of this study was to analyze, through literature review and laboratory tests, four different geometries of web-to-flange joint in order to evaluate the most effective and easy to manufacture. Was analyzed four different geometries: Rectangular section; wedge-shaped "V" with 8.6 degrees of tilt; with two parallel grooves of two mm each; two grooves with a 4.6 degrees of tilt with two millimeters thick, all with 15 mm depth. Figure 3 shows the analyzed geometries.
To production all specimens, a mortising machine was used to notch the flange and connect to the web. Figure 5 shows the mortising machine during production of the rectangular groove.

**Figure 5: Mortising machine producing the rectangular groove**

The flanges solid wood was benefited in laboratory, then was made grooves for each geometries. Figure 6 shows the four geometries produced in laboratory.

**Figure 6: Connection geometries**

A series of tests was performed and produced six specimens for each geometry. The flanges were selected by obtaining a material with a minimum of imperfection and tested on the flexural modulus, density and humidity control between 12 to 14%, like Brazilian Code “NBR 7190 – Design of Timber Structures” [6]. To ensure a perfect union in web-to-flange joint, the specimens were pressed, as shown in Figure 7.

**Figure 7: specimens on pressing apparatus**

To prevent OSB crushing, the web was reinforced on both sides with pieces of solid wood of Pinus Taeda glued onto the OSB with the same resin used in connection. Figure 8 shows the web-to-flange joint specimen with reinforcement.

**Figure 8: Web-to-flange joint specimen with reinforcement**

Tests were conducted until OSB web failure, and the failure mode was recorded. The objective of this test was to analyze if the web could slip in the flange joint when loaded or if the OSB web tore in shear.

**Table 1: Specimens average load and rupture mode**

<table>
<thead>
<tr>
<th>Specimens</th>
<th>average load (kN)</th>
<th>standard deviation (kN)</th>
<th>rupture mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>31,15</td>
<td>1,82</td>
<td>Tore in shear</td>
</tr>
<tr>
<td>Wedge-shaped &quot;V&quot;</td>
<td>28,47</td>
<td>1,52</td>
<td>Tore in shear</td>
</tr>
<tr>
<td>Parallel grooves</td>
<td>27,51</td>
<td>1,71</td>
<td>Tore near connection</td>
</tr>
<tr>
<td>Two inclined grooves</td>
<td>26,46</td>
<td>1,62</td>
<td>Tore near connection</td>
</tr>
</tbody>
</table>

Figure 9 shows the average load graphic.

**Figure 9: Average load**

The web-to-flange joint did not slip during the 24 tests performed. For the parallel and inclined grooves, the OSB tore in shear along the joint, shown in Figure 10.
TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

4 CONCLUSIONS

The rectangular and wedge-shaped "V" geometry had satisfactory results demonstrating higher strength and stiffness. The OSB web tore in shear and the web-to-flange joint did not slip during the 24 tests performed. The geometries with parallel and inclined grooves tore near connection due to reduced shear area. The rectangular and wedge-shaped "V" geometry had a higher performance, with 15.1% difference of the grooved geometry, and also a more practical assembly, facilitating the production.

ACKNOWLEDGEMENT

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THE FUNDAMENTAL STUDY OF THE PROPOSAL OF SHEAR FAILURE STANDARD TEST METHOD AND ESTIMATION OF PURE SHEAR STRENGTH OF JAPANESE CEDAR

Tomohiro Chida¹, Humihiko Gotou², Hidefumi Yamauchi³, Takanobu Sasaki⁴, Yasuo Okazaki⁵, Yasuo Kawai⁶ and Yasuo Iijima⁷

ABSTRACT: The final objectives of this study are the proposal of the new shear failure test method used bending method and the estimation of pure shear strength of Japanese cedar. Since Japanese cedar has very low bending strength than any kind of timber, therefore that shear failure occur rarely in advance of bending failure by using bending method. Thus, when shear failure tests of Japanese cedar are conducted by using bending method, it is necessary to stiffen and to reinforce with any materials which have Young’s modulus and high bending strength such as steel plate. However, if steel plates were bonded to timber with epoxy resin such as steel-plates-inserted glulam-beams, shear stress concentrations occur at the interface between steel plates and timber, and the specimen fractured at there. In this case, the obtained values can be far lower than pure shear strength. Thus, in this study, by using FEA we tried to design the specimen without stress concentrations at the interface and the shear failure occur ahead of bending failure. In addition, the specimens designed by FEA were actually made, and measured the shear strength.

KEYWORDS: Shear failure standard test, Pure shear strength, Japanese cedar, FEA

1 INTRODUCTION

In Akita Prefecture, Japan, various civil engineering structures used Japanese cedar (Cryptomeria Japonica) produced in Akita Prefecture, which is called “Akita sugi”, have been constructed, timber bridges are one of these. When these bridges were designed, 0.9MPa as allowable shear strength of Japanese cedar has been used.

Because the allowable shear strength and some standards for wooden constructions were not shown in the specification for highway bridges of Japan, 0.9MPa was referred to the value of Standard for Structural Design of Timber Structures[1], the standard of other country, etc. However, since this value was too small for timber bridge, beam section become large, and the working efficiency and the costs have deteriorated. In addition, although in timber bridge shear stress only occur subjected to bending, 0.9MPa is not the value obtained from bending failure test. Therefore we consider that 0.9MPa is not adequate value for timber bridge and it is necessary to measure the shear strength by bending method.

The shear tests using bending method, there are three-point bending test and five-point bending test compliant with ISO[2] and three-point bending test and five-point bending test compliant with ASTM[3], etc, however, each test have not become standard test method. In case of these tests, bending fracture can occur prior to shear fracture. Especially in case of Japanese cedar, shear fracture is approximately 10%. On the other hand, we have conducted many shear failure tests for steel plate-inserted Japanese cedar glulam-beams[4], all specimens were destroyed by shear fracture and obtained more than 4.5MPa. However, this is the value of the specimen fractured by shear stress concentration at the interface.
between steel plates and glulam, instead of pure shear strength.

In this study, by using FEA we tried to design the specimen without shear stress concentrations at the interface and the shear failure occur ahead of bending failure. In addition, the specimens were actually made and conducted asymmetric four-point bending tests, by measuring the shear strengths. These values were compared with the values that obtained from FEA for confirmation of whether obtain pure shear strength.

2 THE DESIGN OF SPECIMEN

2.1 FINITE ELEMENT MODEL

Before designing of the specimen by using FEA, it decided that the shear failure test was conducted by asymmetric four-point bending test as shown in Fig.1[4] and the Japanese cedar was stiffened by the sandwich type as shown in Fig.2, because the bending strength of Japanese cedar is too small. Since then, each position is called on the basis of direction of Fig. 1, for example, the loading position of 2/3P in Fig.1 is called left loading position. Finite element model is shown in Fig.3. For this model, we conducted FEA by using 4 and 8-node isoparametric brick element. The used FEA tool was ANSYS ver.13. We denoted the axes of the width, depth and axial directions of the beam by \( x, y \), and \( z \), respectively. In this model, support plates and loading plates were modelled, because the distances between left support plate and left loading plates (equally, right support plate and right loading plate) are short, and the each plate size influence on shear stress distribution as shown in Fig.4[5]. Although the positive shear stress in Fig.4 is smaller than beam theory, if the support plates

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**Figure 1:** Bending moment diagram and shear force diagram under asymmetric four-point-bending test.

Legend : \( P \) : Total load, \( l \) : Effective length, \( A \) : the length between the left support and left loading position, \( B \) : the length between the left loading position and the right support, \( C \) : the length between the right support and the right loading position, \( CL \) : Center line.

**Figure 2:** Beam-height ratio of stiffener and Japanese cedar, and height-width ratio.

Legend : \( b \) : width of beam, \( h \) : height of beam.

**Figure 3:** Finite element model with mesh divisions

Legend : \( O \) : The coordinate origin, \( x, y, z \) : the axes of the width, depth, axial directions of the beam, respectively.

**Figure 4:** Theoretical and presumed experimental shear stress diagram on neutral axis

Legend : \( A \) : the area of cross section.

**Figure 5:** (a): Restraint position of support plates.

Note : Displacements of circled nodes were restrained.

(b) The loading position of loading plates.
and loading plates are larger size of the specimen, the negative shear stress can be smaller than beam theory as with positive shear stress. Also in FEA, boundary conditions are given by restriction of nodal displacements and loading condition is directly given to the nodes, the stress $\tau_{\text{max}}$ concentration occur at its nodes themselves and wide range of nodes around them. So shoe and loading plates are only given boundary and loading condition as shown Fig. 5, and wood specimen itself is not given boundary and loading condition. In this case, the nodes of interface between wood specimen and shoe and loading plate are not shared, the analysis is conducted as contact problem using friction (RBM: restraint buffer method[6]). Thus, after deciding each plate size to $150 \times 90 \times 10$ mm, cross sectional sizes were assumed that height $h$ was 100mm (Japanese cedar: $h/2=50$mm, each stiffener: $h/4=25$mm) and width $b$ was $h/2=50$mm by referring the literature[4], the beam length $l$ was decided 800mm as the size that each plates cannot influence the negative shear stress by conducting preliminary analyses. The properties of Japanese cedar are as follows [1, 4]; the Young’s moduli perpendicular to axis $E_z = E_{y} = E_{x}$; the axial Young’s modulus $E/25$; the shear moduli $G_{xy} = G_{yz} = G_{xz} = E/15$; Poisson’s ratios $\nu_{xy} = \nu_{xz} = \nu_{yz} = 0.166$; $\nu_{yx} = \nu_{zx} = 0.4$ so that the strain-stress matrix becomes symmetric. The axial Young’s modulus $E_i$ is assumed 7.5GPa in the design of this section. In comparison with FEA and experiments, $E_i$ are used actual measured values. The properties of steels(loading plates and shoes) are as follows; The Young’s modulus $E = 200$GPa; Poisson’s ratio $\nu = 0.3$. Element sizes of Japanese cedar and each stiffener are 2.5mm and those of each plate are 15mm.

2.2 THE DESIGN METHOD OF FEA

In beam theory, shear stress $\tau_{\text{max}}$ on neutral axis of sandwich type is given by the following equation (1).

$$\tau_{\text{max}} = \frac{-E_s S(z)}{b(E^I_{zz} + E^I_{ll})} \frac{mb}{2} \left\{ y^2 - \frac{h^2}{4} - \frac{b^2}{2} \right\}$$  (1)

where $E_s$ is the axial Young’s modulus of Japanese cedar; $I_z$ is the moment of inertia of Japanese cedar; $E_i$ is the axial Young’s modulus of stiffener; $I_l$ is the moment of inertia; $S(z)$ is shear force; $y_z$ is the distance from neutral axis to stiffener, respectively. In case of using steel plates as a stiffener, shear failure can occur at the interface between Japanese cedar and steel due to shear stress concentration[4, 6]. Since this shear stress concentration can not be considered by equation (1) and also can not measure in experiment by using the strain gauges, it is necessary to design the cross section without occurring shear stress concentration at the interface. Thus by FEA can make? the shear stress concentration at the interface that we investigated the adequate stiffener without occurring shear stress concentration. FEA was conducted as Young’s moduli of stiffener were changed. Also, all FEA in this section were conducted under 60kN. Fig. 6 shows shear stress distributions derived from Young’s modular ratio (axial Young’s modulus of maximum shear stress occur stiffener/axial Young’s modulus of Japanese cedar). When Young’s modular ratio was less than 5, the at neutral axis; when Young’s modular ratio was 7, the maximum shear stress occur almost the same at neutral axis and the interface; and when Young’s modular ratio was more than 10, the maximum shear stress occur at the interface. Accordingly, it is desirable to utilize the stiffener that Young’s modular ratio is less than 5. So, different woodmaterial, whose Young’s modulus and bending strength were about two times great than Japanese cedar, become one of the candidates of stiffener. Therefore, we conducted the analysis using wood lamina L140 specified Japanese Agricultural Standards (JAS)[7] as stiffener. The axial Young’s modulus of L140 used lower limit 11.5GPa(Young’s modular ratio=2.1). In addition, we also carried out the analysis that axial Young’s modulus of stiffener was 16.5GPa(Young’s modular ratio=3.0) as the candidate for comparison. Fig. 7 shows the shear stress distributions of the beam used timber as the stiffener.
than the value of FEA. However, in the actual experiment, these error between FEA and equation (1) can not clearly occur. The lower limit of bending strength of L140 is 40.5MPa, in case of the specimen of in this section, it is possible to load 100kN. In this case, measurable shear stress becomes 9MPa (9.13MPa in equation (1)) from back calculation of FEA as Young’s modular ratio 2.1. Since lower limit of shear strength of Japanese cedar is 6MPa[1], even if wood was utilized as stiffener, it was shown that shear failure can occur prior to bending failure.

3 THE TEST SPECIMEN

The test specimen is shown in Fig.8. The basic sizes of the specimen followed previous section were $l=800$mm, $b=50$mm, $h=100$mm (Japanese cedar; 50mm, stiffener; 25mm). In this study, Larix gmelinii was used as stiffener. Also, to compare with the results of previous test[4] not Japanese cedar but Japanese cedar glulam was used to the specimen for once. To avoid bearing at support and loading position, secondary parts which were spruce glulam etc. were bonded with epoxy resin at support and loading position (Fig.8). The size of secondary parts were $50 \times 100 \times 100$mm. The installation positions of secondary parts are shown in Fig.9 and Fig.10. To confirm the effects of secondary parts, we made two kinds of specimens which have four or eight secondary parts. A total of five specimens were made; three of them were the specimens with eight secondary parts and two of them were the specimens with four secondary parts. Young’s moduli of main members are shown in Table 1, Young’s moduli of secondary parts are shown in Table 2, moisture contents of main members are shown in Table 3 and yield load of main members are shown in Table 4. Since A2 specimen broke in elastic range, the yield load of A2 is not shown in Table 4. The measurements of shear strain were utilized rosette gauges attached to both sides of the beam. The installation positions of rosette gauges are shown in Fig.9 and Fig.10.

4 THE EXPERIMENTAL RESULTS

4.1 COMPARISON WITH EXPERIMENT, FEA AND BEAM THEORY IN ELASTIC REGION

In around $P=20$kN corresponding to elastic region, the values of shear stress $\tau_{yz}$ at the installation position of rosette gauges (see Fig.9 and Fig.10) of each specimen are shown in Table 5, and the shear stress distributions of the specimens with four secondary parts and with eight secondary parts are shown in Fig.11. Experimental values in Table 5 are obtained by averaging the value of both sides, and the values of the positive shear stress are obtained by averaging right and left value. With regards to the negative shear stress, the values obtained from experiment, FEA and equation(1) corresponded well, its difference is less than 5% even at the maximum. On the other hand, With regards to the

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Table 1. Young’s moduli of main members.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Upper lamina(GPa)</th>
<th>Japanese Cedar glulam(GPa)</th>
<th>Lower lamina(GPa)</th>
<th>Secondary parts</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>20.59</td>
<td>8.32</td>
<td>20.15</td>
<td>S1</td>
</tr>
<tr>
<td>A2</td>
<td>18.64</td>
<td>7.60</td>
<td>18.66</td>
<td>S2</td>
</tr>
<tr>
<td>A3</td>
<td>20.59</td>
<td>8.05</td>
<td>20.15</td>
<td>S3</td>
</tr>
<tr>
<td>B1</td>
<td>20.59</td>
<td>7.97</td>
<td>20.15</td>
<td>S2</td>
</tr>
<tr>
<td>B2</td>
<td>18.64</td>
<td>7.50</td>
<td>18.66</td>
<td>S3</td>
</tr>
</tbody>
</table>

Legend : A1-A3 are specimens with eight secondary parts, B1 and B2 are specimens with four secondary parts. S1-S3 are secondary parts (see Table 2).

Table 2. Young’s moduli of secondary parts.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tree species</th>
<th>$E_\text{c}(GPa)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Spruce glulam</td>
<td>13.02</td>
</tr>
<tr>
<td>S2</td>
<td>Japanese Cedar glulam</td>
<td>7.89</td>
</tr>
</tbody>
</table>

Note : S1 specimen was produced as well as A1-A3 and B1-B2.

Table 3. Moisture contents (%).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>A1</th>
<th>A2</th>
<th>A3</th>
<th>B1</th>
<th>B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture contents</td>
<td>14.1</td>
<td>12.6</td>
<td>13.6</td>
<td>12.8</td>
<td>13.6</td>
</tr>
</tbody>
</table>

Legend : See Table 1.

Table 4. Yield load (kN).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>A1</th>
<th>A2</th>
<th>A3</th>
<th>B1</th>
<th>B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield load</td>
<td>41.46</td>
<td>-</td>
<td>32.54</td>
<td>33.00</td>
<td>40.96</td>
</tr>
</tbody>
</table>

Legend : See Table 1.
positive shear stress, the each value was not corresponded. In Fig.11, whereas the negative shear stress distribution is uniform, the positive shear stress distribution is very complex. There is no difference between the specimen with four secondary parts and with eight secondary parts in this behavior. Although in FEA the shear stresses are obtained in pin-point, since the size of the rosette gauge used in experiment is 20mm, it is considered that experimental values were directly affected by complex shear stress distribution.

4.2 COMPARISON WITH EXPERIMENT AND FEA AT ULTIMATE LOAD
In all specimens, shear failure occurred at adhesive layer within Japanese cedar glulam near the neutral axis. The fracture location of all specimens except B1 specimen were the center of beam, and the fracture location of B1 was the end of beam. The shear stresses at the ultimate and the ultimate load are shown in Table 6. Since A2 specimen broke in elastic range, plastic analysis for A2 specimen did not conducted. Also, in B1 and B2 specimen because the differences of the shear stress of both sides were great, the shear stresses were shown not to be averaged in Table 6. Fig.12 shows a typical example of the distribution of the yield strain obtained by FEA analysis. The parts of bright colors are the yield part in Fig.12. With regards to the specimen with eight secondary parts, the shear stress obtained FEA and experiment were in good agreement as with the elastic range. In Fig.12, there are not the yield part around the center of beam where the shear failure occur, and shear stress concentrations did not occur at there. In addition, since the bending moment is 0 at the center of beam (see Fig.1), the shear stresses obtained from A1 ~ A3 specimens can be the pure shear strength of Japanese cedar glulam used this experiment, and those values are from 4.4MPa to 5.7MPa.

On the other hand, for the specimen with four secondary parts, there are great difference between experimental value and FEA although those values corresponded well in elastic range. In the specimens with four secondary parts, since the differences of the shear stresses of both sides are great, lateral torsion could have been occurred. Due to the plastic at the parts shown in Fig.12, it is considered that the beams become unstable. Since this behaviour is not seen in the specimen with...
eight secondary parts, it is inferred that the secondary parts at left support plate and right loading plate prevented the lateral torsion. Thus the negative shear stresses in Table 6 can not be pure shear stress. However, B2 specimen broke at the end of beam, the positive shear stresses at there are 4.36 ~ 4.79MPa, and there are almost no difference with the values of the specimens with eight secondary parts.

5 CONCLUSIONS

By using FEA we tried to design the specimen without shear stress concentrations at the interface and the shear failure occur ahead of bending failure. Based on the FEA results, the Japanese cedar laminates using Larix gmelinii as stiffener were made with secondary parts at supports and loading positions. All specimens were fractured by shear failure ahead of bending failure around neutral axis at the center line. The obtained shear strengths were 4.4MPa ~ 5.7MPa which were pure shear strength of Japanese cedar glulam.

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REFERENCES

PERFORMANCE OF A MECHANICALLY LAMINATED TIMBER ARCHED SUSPENSION BRIDGE

Cecilia Alonso\(^1\), Abdy Kermani\(^2\)

**ABSTRACT:** The study aimed to investigate the potential of mechanically laminated timber structures for bridge construction and their possible application in developing structural uses for low-grade UK grown timber. A mechanically laminated bridge was designed as a half-scale of a 12m span bridge for pedestrian use and was subjected to a series of experimental and analytical investigations. A flat stress-laminated timber deck was formed by transversely post-tensioning short timber planks with steel bars. The resulting orthotropic solid plate was suspended from two screwed laminated slender arches, in which timber predominately acted in compression and end-bearing at the butt joints. The bridge behaved elastically well beyond its design load, and showed a high level of ductility with good recovery capacity. It was found that at high load levels, the out-of-plane deformation of the slender arches could lead to instability of the structure when loaded symmetrically. However, as expected, the failure resulted at load levels in excess of 6 times the design load under a quarter-point loading condition. The study also demonstrated that large span, stiff and strong structures can be constructed by means of short lengths of low-quality timber.

**KEYWORDS:** Mechanical lamination, Stress-laminated timber, Timber bridges

1 INTRODUCTION

Stress-laminated timber (SLT) is a relatively new technique in which rectangular timber planks are post-tensioned transversely with high-strength steel rods, creating a solid wood plate with orthotropic characteristics. The innovative system was first developed in Canada in 1976 for the rehabilitation of deteriorated nail laminated wood decks [1]. Since load is transmitted transversely through friction between laminates, the planks do not have to be continuous over the whole span, allowing short lengths of timber to be used in construction of the bridge deck. However, the introduction of butt joints reduces the longitudinal bending stiffness of the deck [1]. Above a minimum pre-stress level, the longitudinal stiffness is no longer influenced by the transverse pressure [2]. A reduction factor, function of the pattern of butt joints, is commonly used to calculate the effective width of the deck. The required level of pre-stress force is determined by the transverse moment to be resisted by the deck.

As in any lamination technique, wood defects are dispersed within the material, improving the load bearing capacity. SLT flat decks will not delaminate over time and provide dimensionally stable decks provided that, with regular maintenance, the stress levels in the transverse stressing bars are maintained [3]. These systems can be easily dismantled by simply loosening of the nuts, thus simplifying the replacement of damaged/deteriorated laminate(s). Additionally, SLT decks with butt joints may be cambered to offset the dead-load deflection and the additional vertical creep [2]. However, loss of stress levels in the transverse/stressing bars can result in significant loss of stiffness, in particular, in a longitudinal direction, compromising the structural integrity of the flat deck system. It has also been found that the application of flat slab-type SLT decks is limited to bridges of 6m to 9m spans [4], [5]. Composite pre-stressed T-beams and box section decks have been developed, but these systems are not suitable for the UK due to the wet weather or the requirements for glulam members [5].

To overcome these limitations, the performance of load bearing arched SLT decks was recently examined in a collaborative research project between the Centre for Timber Engineering at Edinburgh Napier University and the UK Forestry Civil Engineering, Forestry Commission. The study revealed that the utilisation of timber in an arching action where timber predominantly acts in compression and end-bearing rather than in bending, drastically improves the strength and stiffness of the bridge [6].

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The benefits of the arching action can further be enhanced with the increase in the rise to span ratio, but this would then adversely affect the ease of access over the deck, due to increase in slope, a limiting design criterion. Although SLT footbridges have demonstrated cost-competitiveness, the potential in cost savings is greater for vehicle bridges. For this purpose, a flat SLT deck supported by arches can ideally lead to a more efficient design.

2 DESIGN AND CONSTRUCTION

This research has focused on the development and examination of a flat SLT bridge deck supported from two screwed-laminated timber arches. A half-scale (half span, half width, half depth) of a 12m span bridge was designed for pedestrian use and subjected to a series of experimental investigations under static loading conditions.

The half-scale model comprised a 6m span, 540mm wide 100mm deep flat SLT deck suspended from two 110mm wide by 150mm deep screwed-laminated arches all using 22mm thick off-cuts of very low-quality Sitka spruce timber. An average MOE of 4800 N/mm² and moisture content of 15% at installation was measured for the laminates.

To form the SLT flat-deck, 1m long laminates were drilled at 250mm centres and staggered in groups of 4 (1 in 4 pattern). In practice, specialised high strength steel bars and a hydraulic jack are required for the pre-stressing system. However, due to the size of the scaled model and since no extra load had to be applied to allow for time related losses, standard mild steel M12 threaded rods and nuts were used.

The arched elements were screwed laminated, as they would not be subjected to transversal moments and in arched profiles interlaminar lateral compression/friction is not as critical as in flat arrangements [7]. Threaded fasteners benefit from the flexibility of being able to be removed and reinserted and are less sensitive to moisture fluctuations. They also provide a more positive connection in withdrawal than bolts or nails [8].

Compression forces are mainly transferred through end bearing at the butt joints in contiguous laminates. Shear and bending moments are transmitted to the adjacent laminates by means of the screws. In doing so, the fastener tends to embed itself elasto-plastically into the surrounding timber whilst it yields, developing plastic hinges at various points along its length. The design of mechanically laminated structures entails the allowance for the influence of slip between laminations, which leads to partial composite interaction.

The arch profile used for this research was a segment of a circle because it can be constructed by means of a series of equal and symmetrical laminates, details are illustrated in Figure 1.

To form one arch, five sequences of laminates were placed on top of each other. Nails were used to hold the laminates in place to prevent movement as the 5mm smooth shank screws were driven without predrilling. Nine screws were used for each laminate, alternating the entrance from one to other side of the beam.

To simulate the horizontal settlement of the supports, two M20 steel tie bars were used to prevent the extremes of the arches from spreading apart. Each rod incorporated a load cell in order to measure the horizontal reaction.

![Figure 1. General dimensions for the bridge.](image)

The extreme pre-stressing bars of the SLT deck pass through the arched beams, transmitting part of the horizontal reaction through friction to the deck. Consequently, the influence of the elasticity of the supports in the structural efficiency of the arched profile is reduced. After assembly, approximately 20kN of pre-stressing load was applied to each bar directly by tightening the nut with a calibrated manual wrench, corresponding to a compressive pressure of 0.80 N/mm². Square mild-steel bearing plates of 75mm sides were placed at each anchorage to minimise bearing failure at the outer leaves of the deck.

2.1 TEST ARRANGEMENT

The bridge was subjected to a series of loading regimes in both symmetrical and eccentric arrangements. An initial preload was applied in order to eliminate the effect of initial settlements. Displacement transducers measured the deformations at several points along the deck and the arches. Final tests to failure were performed under the worst loading conditions, aiming to illustrate the most probable failure modes.

2.2 FINITE ELEMENT ANALYSIS

A FEM analysis was performed in order to predict the behaviour of the structure. Full details of the analytical work will be discussed in future publications. However, the close agreement between analytical and experimental results demonstrated that the structural performance of mechanically laminated timber structures can be accurately predicted using linear elastic analysis.

3 RESULTS AND DISCUSSION

3.1 SYMMETRICAL LOADING

A four-point loading arrangement, as shown in Figure 2, was used to represent a uniformly distributed loading (udl) condition, which exaggerated the applied stresses and the resulting deformations by about 35%. The bridge was loaded up to 45kN, equivalent to 2.8 times the design load (5kN/m²).

The bridge behaved elastically well above its design load. At extreme loads of over 40kN, the structure began to exhibit elasto-plastic behaviour, Figure 3.
The maximum vertical deflection recorded for the deck was 17.15mm at mid-span when subjected to 45kN symmetrical loading.

At the design load of 16.2kN, 6.0mm deformation was recorded, which corresponds to 1/1000 of the span. This indicates the high stiffness properties of the structure when compared with the most restrictive limits of vertical deflection under live load of 1/400, Figure 4.

For the arches, a maximum deflection of 16mm under a 22.5kN was recorded at 1.5m from the mid-span of the bridge, under the applied load. It was concluded that this quarter-point loading condition was the most likely case to cause the collapse of the structure, since it produces a more pronounced uplift in the arches in the opposite side of the bridge.

The horizontal reaction measured for the unbalanced loading was lower than in a symmetrical arrangement symmetrical loads is governed by the out of plane buckling of the arched elements, Figure 5.

As well as the need for lateral stiffeners, the structural performance of the arches is highly dependent on the quality of workmanship. Laminates are required to be cut and assembled accurately, as geometrical imperfections are likely to promote an instability problem. Lateral buckling is intensified by the lack of composite behaviour of the laminates due to slip between laminations (if, for example, gaps are present), which results in a decreased stiffness of the arches about the weaker, transverse, axes.

After application of a 45kN symmetrical loading, the structure, which was not physically damaged, was unloaded and allowed to recover for over 24 hours.

### 3.2 ECCENTRIC LOADING

Eccentric loading was considered to be the critical loading condition for arched structures. Four positions were selected for application of eccentric line loads, at 0.5, 1.0, 1.5 and 2m from mid-span. In each case, the load was increased up to 22.5kN, equivalent to 2.8 times the design load, Figure 6.

During these tests, no significant lateral buckling of the arches was observed. The structure behaved elastically and showed no sign of any distress. The maximum measured vertical deflection in the deck was 20.95mm for 22.5kN, when the load was applied at a point 1.0m from the mid-span. At the design load, computed as the udl over half of the span of the bridge, the maximum vertical deflection was 7.24mm, corresponding to 1/830 of the span.

For the arches, a maximum deflection of 16mm under a 22.5kN was recorded at 1.5m from the mid-span of the bridge, under the applied load. It was concluded that this quarter-point loading condition was the most likely case to cause the collapse of the structure, since it produces a more pronounced uplift in the arches in the opposite side of the bridge.

The horizontal reaction measured for the unbalanced loading was lower than in a symmetrical arrangement.
and it increased as the distance between the point of application of the load and the mid-span increased.

![Graph](image1)

**Figure 6.** Vertical deformation of the bridge subjected to an eccentric line load of 22.5kN in different positions along the deck. Deflections are exaggerated by a factor of 10 for illustration.

The effectiveness of an arched profile resides in the load being transferred through compression to the abutments. However, the efficiency of the arching action is reduced when the arch is subjected to eccentric/unbalanced loading conditions, resulting in increase in bending stresses and deflections and reduction in horizontal thrust.

### 3.3 TESTS TO FAILURE

In the final stage of loading, the bridge was loaded at its quarter-point, representing the worst loading condition. Throughout the loading, the bridge exhibited an exceptionally high ductile behaviour until failure of a secondary member occurred at a load of 50kN representing 6.2 times its design load. Observation of the deformation behaviour showed that at loads above 25kN (approximately 3 times the design load) the bridge started to exhibit plastic deformation. The maximum deformation recorded was 51.3mm under 50kN load, whilst the uplift of the deck on the opposite side was 26mm.

After replacement of the damaged component, the repaired bridge was again loaded under a four-point loading arrangement up to 50kN. This last test confirmed the superior recovery capacity and outstanding post-critical load capacity of the bridge after being subjected to loads close to failure. However, the distortion of the arches was considerably greater than that in the earlier test i.e. prior to being subjected to plastic deformations.

![Graph](image2)

**Figure 7.** Horizontal reaction measured by load cells for eccentric line loads.

The study demonstrated that the mechanically laminated timber arched suspension bridge behaved elastically well beyond its design load. The structure exhibited high stiffness with deflection levels much lower than the permitted limits. At high load levels, the structural system also showed considerable ductility and the ability to redistribute the induced stresses efficiently, hence sustaining loads in excess of 6 times its design load.

### 4 CONCLUSIONS

The study demonstrated that the mechanically laminated timber arched suspension bridge behaved elastically well beyond its design load. The structure exhibited high stiffness with deflection levels much lower than the permitted limits. At high load levels, the structural system also showed considerable ductility and the ability to redistribute the induced stresses efficiently, hence sustaining loads in excess of 6 times its design load.

### 5 REFERENCES


SELECTION AND GRADING METHODS FOR MARITIME PINE UTILITY POLES

Carlos Eduardo de Jesus Martins\textsuperscript{1}, Alfredo Manuel Pereira Geraldes Dias\textsuperscript{2}

\textbf{ABSTRACT:} The objectives of the research project described in this paper are to characterize the wood product and to improve the grading methods used for Portuguese Maritime pine utility poles. The characterization comprised the experimental testing of 64 utility poles following the indications from EN 14229. Based on the results obtained, the influence of the visual characteristics and physical properties in the mechanical properties of the wood utility poles (elasticity modulus and bending strength) is analysed and discussed. The results obtained are also compared with the one available for other Maritime pine products and for utility poles from other species. The correlation coefficients between the bending strength and other mechanical properties, including dynamic elasticity modulus determined for a second sample, as well as the correlation coefficients between the bending strength and the visual characteristics and physical properties were determined and are discussed in the paper.

\textbf{KEYWORDS:} Wood utility poles, grading of wood utility poles, round wood, bending test, Maritime pine

\section{INTRODUCTION}

The Civil Engineering Department of the University of Coimbra set up a project with the purpose of characterizing Maritime pine wood utility poles and improving the grading methods used for this species. In the aim of this work experimental tests were performed in 8m utility poles following the indications from EN 14229\textsuperscript{[1]}. Based on the results obtained, the influence of the visual characteristics and physical properties in the mechanical properties of the wood utility poles (elasticity modulus and bending strength) were analysed and are discussed. Additionally to the assessment based on the visual characteristics combined with physical properties, non-destructive methods were also considered as a possibility to improve the grading methods. To this end, the dynamic elasticity modulus was used, which usually shows good correlations with the static elasticity modulus. A new sample of Maritime pine wood utility poles was selected and tested, using non-destructive methods (axial vibration) based on which the dynamic elasticity modulus was determined. The same specimens were afterwards tested to determine the static elasticity modulus. The correlation between these two mechanical properties was determined and is discussed. From the results obtained in this study it was concluded that significant improvements can be achieved in the grading methods used in the production of Maritime pine wood utility poles.

The results obtained in this study are also compared with the one obtained for other applications (structural) using Maritime pine and the same application (utility poles) using other species.

\section{TEST SPECIMENS}

In this study an initial sample of 64 Maritime poles with a nominal length equal to 8 m was tested. The selected elements had dimensions defined in accordance with the indications from EN 14229. In order to fulfil its requirements, the utility poles’ nominal length had to be between 7.92 m and 8.16 m, while its nominal diameter at the ground-line had to be between 180 mm and 220 mm. The moisture content of the poles, at the moment of the bending test, had to be equal or higher than the fibre saturation point, usually assumed as 24\% for this species. In order to fulfil this requirement the poles were selected and tested immediately after debarking, which occurred shortly after the trees were cut.

For each one of the utility poles various visual properties were determined: ovality, knots, taper, rate of growth, slope of grain and heartwood. These visual properties were assessed as listed below:

- Ovality - difference between the maximum and minimum diameters of the cross-section, expressed as a percentage of the minimum diameter;
TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

- Knots – diameter of the knot; case 1) the maximum diameter of a knot or knot cluster expressed as a factor of the circumference of the wood utility pole at the point where the knot occurs; case 2) the maximum sum of all the knot diameters in any 300-mm segment of the wood utility pole, expressed as a factor of the circumference of the wood utility pole at the mid-point of the segment;
- Taper - ratio between the difference of the nominal diameters of the ground-line section and of the load-bearing section and the distance measured between these two sections;
- Rate of growth - the number of rings in 25 mm on the largest radius of the cross-section, starting the measurement at a distance of 50 mm from the centre of the pole section;
- Slope of the grain - deviation of the fibres in a length of 100 mm extrapolated to a length of 1m;
- Heartwood – percentage of heartwood in the whole section of the pole.

The mean values of results obtained are presented in Table 1.

Table 1. Features of the tested utility poles

<table>
<thead>
<tr>
<th>Feature</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ovality, ground-line (%)</td>
<td>3.94</td>
</tr>
<tr>
<td>Ovality, load point (%)</td>
<td>4.74</td>
</tr>
<tr>
<td>Taper (mm/m)</td>
<td>6.87</td>
</tr>
<tr>
<td>Knots medium (mm)</td>
<td>23.2</td>
</tr>
<tr>
<td>Knots Maximum (mm)</td>
<td>44.2</td>
</tr>
<tr>
<td>Knots Case 1 (mm/mm)</td>
<td>0.083</td>
</tr>
<tr>
<td>Knots Case 2 (mm/mm)</td>
<td>0.321</td>
</tr>
<tr>
<td>Slope of grain (cm/m)</td>
<td>4.75</td>
</tr>
<tr>
<td>Heartwood (%)</td>
<td>16.96</td>
</tr>
<tr>
<td>Growth rate (rings/25mm)</td>
<td>12.96</td>
</tr>
</tbody>
</table>

3 TEST RESULTS AND DISCUSSION

The utility poles were tested in bending, using a cantilever test set up, in accordance with the indications from EN 14229 (see Figure 1). From these tests, the bending strength and static elasticity modulus were determined. The mean values obtained in the tests are presented in Table 2.

Figure 1. Figure of the bending tests

In addition to this, wood samples were collected from each specimen to determine the moisture content and density.

Table 2. Mean values of the density, elasticity modulus and bending strength

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>573.7</td>
</tr>
<tr>
<td>Elasticity modulus (MPa)</td>
<td>10900</td>
</tr>
<tr>
<td>Bending strength (MPa)</td>
<td>50.2</td>
</tr>
<tr>
<td>* - The density presented corresponds to moisture content of 12%</td>
<td></td>
</tr>
</tbody>
</table>

A second sample of 45 wood utility poles was used to determine the mechanical properties through a non-destructive approach. The method used was the dynamic determination of the axial elasticity modulus. For the same poles the static elasticity modulus was afterwards determined though the procedure defined in EN 14229. The mean values of the results obtained are presented in Table 3.

Table 3. Mean value of the dynamic elasticity modulus and of the static elasticity modulus of the second sample

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic elasticity modulus (MPa)</td>
<td>13714</td>
</tr>
<tr>
<td>Static elasticity modulus (MPa)</td>
<td>10839</td>
</tr>
</tbody>
</table>

The results of static elasticity modulus are quite similar to the one obtained for the first sample tested. Regarding the dynamic elasticity modulus it is important to point the results obtained are more than 20% higher than those obtained for the static elasticity modulus.

Once the final goal is the definition of improved grading rules, the determination of the correlation coefficients between the mechanical properties and other non-destructive features is a critical issue. In this work the correlations between the bending strength and the elasticity modulus (static) and the correlation coefficients between the visual/anatomical properties and the bending strength were determined. The results obtained are presented in Table 4.
Table 4. Coefficients of correlation between properties determined in destructive tests and the bending strength, first sample

<table>
<thead>
<tr>
<th>Property</th>
<th>Coef. of correl. with ( f_m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{nom,q} ) (mm)</td>
<td>-0.33</td>
</tr>
<tr>
<td>( d_{nom,g} ) (mm)</td>
<td>0.33</td>
</tr>
<tr>
<td>Ovality, ground-line (%)</td>
<td>0.13</td>
</tr>
<tr>
<td>Ovality, load point (%)</td>
<td>0.03</td>
</tr>
<tr>
<td>Knots (medium)</td>
<td>-0.44</td>
</tr>
<tr>
<td>Slope of grain</td>
<td>-0.13</td>
</tr>
<tr>
<td>Growth rate (rings/25mm)</td>
<td>0.19</td>
</tr>
<tr>
<td>Density (Kg/m(^3))</td>
<td>0.66</td>
</tr>
<tr>
<td>Static elasticity modulus (GPa)</td>
<td>0.83</td>
</tr>
</tbody>
</table>

The coefficients of correlation presented in Table 4 can be grouped in three distinct categories: good correlations, medium correlation, low or null correlations.

The static elasticity modulus showed a good correlation with the poles bending strength. There are two properties that can be considered as properties with medium correlation with bending strength: density and knots. The correlations coefficients obtained for the remaining measured properties are low.

In the second sample the correlation coefficient between static and dynamic elasticity modulus was determined, and the value obtained was 0.60. Clearly this non-destructive approach to determine the dynamic elasticity modulus of the utility poles can be considered as a good possibility for inclusion on the grading process.

It is clear from these results that the elasticity modulus is a key property in order to develop an efficient grading method, either determined in a static test (better option) or determined in a dynamic test. A good complement to this property could be the density or the knots. The problem associated with the last is the difficulty to measure it, namely because in many situations the knots are not visible in the utility pole surface. In these tests it was clear that even when not visible these knots could be the main cause of the pole failure. On the other hand, in the production process the determination of the timber density is much easier to implement. If the static elasticity modulus is combined with the other physical and visual parameters, in a multiple linear regression, the coefficient of correlation obtained in this research increased to 0.94.

Nevertheless, it is clear from these results that the measurement of many of the properties considered in this study is useless for grading purposes, once the contribution to the grading procedures accuracy would be negligible.

4 COMPARISON WITH RESULTS FROM OTHER RESEARCH PROJECTS

The values obtained are in line with the data available for Maritime pine timber in general, namely for rectangular cross sections graded according to NP 4305 [2, 3]. In terms of circular sections the results obtained in this study can be compared with the one obtained with small diameter timber, from the same species, but graded for structural applications, available from a research aiming its structural application [4, 5] (see Table 5).

Table 5. Mechanical properties for small diameter round wood timber

<table>
<thead>
<tr>
<th>Species</th>
<th>( f_m ) (MPa)</th>
<th>E (GPa)</th>
<th>( \rho ) (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poles(^a)</td>
<td>74.3</td>
<td>12.9</td>
<td>573</td>
</tr>
<tr>
<td>Small diameter</td>
<td>86.8</td>
<td>14.9</td>
<td>536</td>
</tr>
</tbody>
</table>

\(^a\) – Values adjusted to a moisture content on timber of 12% following the indication given by Hoffmeyer [6]

It is clear from these results that the wood utility poles show lower strength and stiffness than the small diameter round wood.

In this aim it is also important to compare the results obtained here for Maritime pine utility poles with those obtained for the same product, in similar tests, with other species. In Table 6 are presented values obtained for other species from various origins.

Table 6. Mechanical properties for utility poles from other softwood species

<table>
<thead>
<tr>
<th>Species</th>
<th>( f_m ) (MPa)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maritime pine – Portugal (this study)</td>
<td>50.2</td>
<td>10.9</td>
</tr>
<tr>
<td>Radiata pine – Chile - [7]</td>
<td>52.0</td>
<td>10.5</td>
</tr>
<tr>
<td>Southern pine – USA - [8]</td>
<td>58.1</td>
<td>17.3</td>
</tr>
<tr>
<td>Douglas – fir – USA- [8]</td>
<td>54.2</td>
<td>18.2</td>
</tr>
<tr>
<td>Western redcedar – USA - [8]</td>
<td>35.9</td>
<td>11.0</td>
</tr>
</tbody>
</table>

From analysis of the results presented in Table 6, it is possible to conclude that utility poles produced from Maritime pine round wood have similar mechanical properties from those produced from other species and other origins. Indeed, both the stiffness properties and the strength properties are quite similar to the one obtained for the other species.
5 CONCLUSIONS
In this research Maritime pine utility poles were tested in bending. The results obtained clearly show the good conditions of this specie for this application. The mechanical properties obtained are well in line in those obtained in other research projects for small diameter round timber and rectangular cross sections from the same species. Additionally, the comparison of the results obtained here with those obtained for other species, clearly show that the mechanical performance of Maritime pine utility poles is similar to the performance of the utility poles produced from other soft wood species. The analysis undertaken also showed that the product properties can be significantly improved through the use of improved grading procedures. In this regard the utility poles elasticity modulus is a key property in order to develop an efficient grading procedure. The density and the size of the knots can also add valuable information to the grading procedure. The consideration of the other physical/anatomical properties considered in this study would mostly add complexity and cost to the process without bringing visible improvements to the grading yields. The dynamic elasticity modulus has shown to be a reasonable alternative property, to replace the static elasticity modulus whose determination is more complex.

NOTATION
\(d_{nom,g}\) – Nominal diameter of the pole at the ground line section
\(d_{nom,q}\) - Nominal diameter of the pole at the loading section
\(f_m\) – Mean value of the bending strength
E – Mean value of the elasticity modulus
R – Mean value of the timber density

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[3]. LNEC, M2-Madeira para construção - Pinho bravo para estruturas, in Fichas técnicas do LNEC. 1997, LNEC.
“GREEN” GLUING OF TWO TROPICAL TIMBERS BY GLULAM TECHNIQUE USING A RESORCINOL-PHENOL-FORMOL (RPF) AND A ONE COMPONENT POLYURETHANE (1CPU) ADHESIVES: CASE OF TRIPLOCHITON SCLÉROXYLON (AYOUS) AND TERMINALIA SUPERBA (FRAKÉ)

Njungab Emmanuel 1; Ntede N. Hippolyte 2 Regis Pommier3;Ndikontar M. Kor1; Ayina. O2; Noah N. Joseph1;Rene oum lissouk; Mpon Richard1;

ABSTRACT
This communication presents an evaluation of gluing performance of a one-component polyurethane adhesive (1C-PUR) on Triplochiton scleroxylon (ayous) and Terminalia superba (fraké) as substrate. Traditional and established methods used in the gluing of wood joints require the wood to be dried to moisture content (MC) that is well below its fibre saturation point. However, in some circumstances, the use of raw wood material is reduced due to degradation during drying. This is particularly the case for many tropical woods which are dense. To reduce degradation due to drying and the cost of drying, various technologies for high moisture content and green/wet wood gluing have been developed, making more efficient use of valuable and renewable natural resources. There are several adhesives available for structural applications based on the different end-use requirements. Currently there are two common classes of adhesives that are used by engineered wood product (EWP) industries. These are phenolic-based and polyurethane (PU). Two popularly tropical wood ayous and fraké with various moisture contents (13 to 54 %) and respective density of 0.38 and 0.53 were glued using a resorcinol-phenol-formol (RPF) and 1C-PUR as adhesive. Shear strength tests were performed on these species and percent wood failure was measured. The result obtained indicates a decreasing performance of adhesive glue joint with the increasing moisture content of wood. Though RPF has an outstanding durability and presents good adhesion to wood, it presents disadvantages of being a water-born adhesive and a formaldehyde releaser. The polyurethane appeared to be the faster bonding adhesive due to its water-induced hardening reaction. The results obtained with RPF and 1C-PUR are promising for the optimization of the gluing process for efficient use of more tropical African timbers.

Key words: green gluing, tropical timbers, glulam, RPF adhesive, 1C-PUR adhesive, shear test

1. INTRODUCTION
Wood is one of the most abundant materials in the Congo large forest reserve (more than 80% of the country) with remarkable technological potentials. These one explains that it is use throughout various forms: solid timber products, composites or massive reconstituted wood such as glulam, laminated veneered lumber (LVL) and plywood. Included in several products, (carpentry, furniture, timber structures, decorative outdoor works and indoors furniture). In civil engineering structures, it meets an efficient and cost effective mechanical support. Composites and reconstituted solid timber (RST), generally obtained by gluing, are the most suitable products used when high and homogeneous mechanical performances are required.

Traditional gluing technologies always require preliminary seasoning of wood to water contents lower than the fibre saturation point (9-14%) [2]. However, in some circumstances, utilisation of the raw wood material is reduced due to drying defects. They usually are noted in the form of twist, bow, spring and cup and needs to be removed by machining before the wood can be successfully jointed. This is particularly the case for many tropical African woods which are massive and heavy. The alternative approach proposed in this work consists in gluing tropical wood with high moisture content (green gluing) and then seasoning it to its usual moisture content during use. Earlier work on joining timber with high moisture content was based on finger
2. WOOD SPECIES SELECTION

Wood have been selected based on resource availability, extraction risk, harvesting potential mainly the transformation ratio (very small 30%). The popularly used two tropical woods: ayous (*triplochiton scléroxylon*) and fraké (*terminalia superba*) with high moisture content. The glulam technique using a resorcinol-phénol-formol (RPF) and one component polyurethane (1C-PUR) as adhesives was adopted.

2.1. MATERIALS AND METHODS

2.1.1. Wood specimens

Wood have been selected based on resource availability, extraction risk, harvesting potential mainly the transformation ratio (very small 30%). The popularly used two tropical woods: ayous (*triplochiton scléroxylon*) and fraké (*terminalia superba*) with high moisture content. The glulam technique using a resorcinol-phénol-formol (RPF) and one component polyurethane (1C-PUR) as adhesives was adopted.

2.1.2. Moisture content of wood

The moisture content of wood specimens was computed as follows:

\[
H\% = \frac{M_w - M_o}{M_o} \times 100
\]

Where \(M_w\) is the mass of wet specimen, \(M_o\) the mass of dry specimen and \(H\%\) its moisture content.

2.2. RPF adhesive gluing

Gluing of wood specimens was made in LMM of Polytechnique High School of Yaoundé 1.

The glue is a liquid (honey colour or dark brown) with a density of 1150 kg/m³ density. It may be used for gluing even the temperature down to 10°C. The hardener is a paraformaldehyde (30-60%) supplied by the manufacturer under the trade name ”hardener 2622” and sold in the form of a brown powder with a density of 560 kg/m³ or 480 kg/m³ depending on whether it is compressed or not. The adhesive was obtained by mixing the two components and had a pH of 8. It was deposited on each face of the wood laminate using a plastic spatula to a 2×200 g/m² spreaded. In order to avoid early polymerization before pressing, gluing operations were carried out between 7 and 8 in the morning at a temperature ranging between 25 and 30 °C (~27°C) close to the adhesive storage temperature (20°C). The minimal polymerization or curing time to obtain acceptable resistances was 6 days. Assembling times ranged between 10 and 25 mn representing the life span of the mixture (adhesive/hardener) before freezing under a clamping pressure of 0,2 MPa.

2.3. 1C-PUR adhesive gluing

Beams were obtained by assembly two lamella of single wood species. For each bonding configuration (ayous, and fraké), two beams are realised. Each beam consists of two individual lamellae with the following dimensions: 20 mm thick, 100 mm wide and 400 mm long. Specific polyurethane glue for gluing substracts has been used. The wood moisture content is 48 ± 6 % (ayous) and 174 ± 9 % (fraké). The open assembly time is 2 min, and the closed assembly time 10 min. Approximately 1 hour after lamellae cutting and planning, a weighed amount of adhesive is spread on each surface at a spread rate of 400 g/m². Both adhesive-spread surfaces are immediately assembled to ensure contact between surfaces. Pressure is applied at 0.96 N/mm². Constant pressure is maintained for 20 hours. After pressing, beams were conditioned at 12°C and 65 % relative humidity for 20 days before cutting into specimens of 45±1×45±1mm² section area. Then submitted to shear stress.

2.4. Shear test [6]

A compression shear stress, parallel to the grain direction, is applied in each bond line. Eighty specimens of 45 mm thickness were cut from 1C-PUR glued beams and fifty from RPF beams. The ultimate shear resistance and wood failure ratio are measured for each broken glue line joint. The moisture content of specimens is 15±2% which represented equilibrium value in tropical climate conditions.

3. RESULTS

Results, concerning fraké and ayous are represented in table 1 and 2:

Table 1: Shear strengths and mode of rupture in accordance with moisture content of wood (ayous)

<table>
<thead>
<tr>
<th>Average humidity (±3%) at 25°C</th>
<th>13</th>
<th>20</th>
<th>25</th>
<th>54</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average load of rupture (kg/mm²)</td>
<td>0.708</td>
<td>0.640</td>
<td>0.541</td>
<td>0.273</td>
</tr>
<tr>
<td>Average shear strength (MPa)</td>
<td>7.08</td>
<td>6.4</td>
<td>5.41</td>
<td>2.73</td>
</tr>
<tr>
<td>Standard deviation (MPa)</td>
<td>0.48</td>
<td>1.27</td>
<td>0.30</td>
<td>0.58</td>
</tr>
<tr>
<td>Mode of rupture (%)</td>
<td>89</td>
<td>35.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Joint of wood</td>
<td>11</td>
<td>34.0</td>
<td>55.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Joint</td>
<td>0</td>
<td>31.0</td>
<td>44.6</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 2: Shear strengths and mode of rupture in relation with moisture content wood (fraké)

<table>
<thead>
<tr>
<th>Average humidity (±3%)</th>
<th>13</th>
<th>20</th>
<th>24</th>
<th>52</th>
</tr>
</thead>
<tbody>
<tr>
<td>at 25°C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average load of rupture (kg/mm²)</td>
<td>0.81</td>
<td>0.56</td>
<td>0.54</td>
<td>0.216</td>
</tr>
<tr>
<td>Average shear strength (MPa)</td>
<td>8.1</td>
<td>5.6</td>
<td>5.4</td>
<td>2.16</td>
</tr>
<tr>
<td>Standard deviation (MPa)</td>
<td>0.66</td>
<td>0.30</td>
<td>0.37</td>
<td>0.40</td>
</tr>
<tr>
<td>Mode of rupture (%)</td>
<td>wood</td>
<td>80</td>
<td>40</td>
<td>28.7</td>
</tr>
<tr>
<td>joint-wood</td>
<td>20</td>
<td>15</td>
<td>14.3</td>
<td>0</td>
</tr>
<tr>
<td>joint</td>
<td>0</td>
<td>45</td>
<td>56</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 3: Average shear resistance of single bond lines and glulam shear requirements

<table>
<thead>
<tr>
<th>adhesive Type</th>
<th>Bond lines</th>
<th>Wood MC at assembly time (%)</th>
<th>Average shear resistance (MPa)</th>
<th>Wood failure (%)</th>
<th>Average density at 15% moisture content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ayous</td>
<td>46±6</td>
<td>8,03±0.95</td>
<td>0.36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1C-PUR</td>
<td>Fraké</td>
<td>174±9</td>
<td>5.26±1.65</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glulam</td>
<td></td>
<td>8</td>
<td>≥ 72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Requirements</td>
<td></td>
<td>9</td>
<td>≥ 63</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(EN 386)</td>
<td></td>
<td>11</td>
<td>≥ 45</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The respect of delamination requirements allows us to perform shear tests (EN 392). Results are shown in table 8 and figures 4 and 5.

Figure 1: Mode of failure in relation with wood moisture content (Frake)

Figure 2: Mode of failure in relation with wood moisture content (Ayous)

Figure 3: Cumulative frequencies of bond lines shear resistance

Figure 4: Shear rupture in single bond line (failure occurs in wood)

Figure 5: Shear rupture in single bond line (failure occurs in adhesive for frake assembly at 174% MC)
5. DISCUSSION

The general trend observed was that wood moisture contents at the time of assembly affects the mechanical performance of the adhesive joint. This influence was most noticeable when the moisture content of wood increase from 13% to the saturation point of fibres or above for the two wood species. Thus for wood water content range between 13 and 17% comparable to air drying, a moderate influence of moisture was observed for the ayous specimen with a percentage rupture in wood of about 80%. This was not the same for fraké which was greatly influenced by wood moisture content even though failure in wood was 90%. The different in behaviour noted here can be explained by the difference in density of ayous and fraké although both of them are considered low density wood. Averages of the corresponding shearing strengths are 8.1 and 7.8 MPa for the lowest moisture content (13%).

Specimens conditioned to a moisture level exceeding fiber saturation

The same experiment have been done with 1C-PUR. In this case it is observed better results of average shear strength than those of RPF adhesive. This can be explained by the fact that 1C-PUR reacts in wet conditions.

6. CONCLUSION

On the basis of the results obtained, either with PF and 1C-PUR adhesives on frake and ayous, RPF although being structural adhesive are not appropriate in wet gluing conditions. Meanwhile 1 C-PUR seems to be more appropriate for green gluing conditions.

REFERENCES


DEVELOPMENT OF A MATERIAL-EFFICIENT FINGER-JOINT PROFILE FOR STRUCTURAL FINGER-JOINED LUMBER

Meng Gong¹, Stephen Delahunty², Shuzhao Rao³, and Y. H. Chui⁴

ABSTRACT: Finger-joined lumber is widely used for structural applications such as glued-laminated timber and I-joists. In North America, the length of a structural finger-joint is between 22 mm (7/8 in) and 29 mm (1 1/8 in), which is longer than that adopted in some countries. Adopting a shorter finger-joint length would result in less waste of good quality lumber and possibly the less use of adhesive. This study was aimed at developing a material-efficient finger-joint profile for structural finger-joined lumber with minimal influence on joint strength. A new joint profile of 12.7 mm (½ in) in length was developed. Lumber grade and cutter sharpness were considered. 2x3 (38 mm thick and 63 mm wide) lumber was used to fabricate 8 groups of 2.44 m (8') long finger-joined lumber specimens. Two groups of unjoined lumber were used as controls. The adhesive used was a two-part polyurethane. The key findings from this study were that the optimized cutters of 12.7 mm (½") in length could produce the finger-joined lumber of better-than-expected quality in terms of modulus of elasticity (MOE) and ultimate tensile strength (UTS) and the bonding quality was satisfied in terms of delamination resistance values (less than 5%). These results suggest that adoption of a short finger joint profile is feasible for Canadian engineered wood products manufacturers to produce finger-joined lumber for structural applications.

KEYWORDS: Finger-joined lumber, joint profile, knife sharpness, joint length, lumber grade, modulus of elasticity, ultimate tensile strength, delamination resistance.

1. INTRODUCTION

Finger-joined lumber is widely used for structural applications such as glued-laminated timber, I-joists, wall studs and trusses. In North America, the length of a structural finger-joint is between 22 mm (7/8 in) and 29 mm (1 1/8 in), which is longer than that adopted in some countries, for example, which uses a 9.5 mm (3/8 in) to 12.7 mm (½ in) finger length. A general misconception among Canadian engineered wood products industry is that short finger length corresponds to a lower joint strength. As a result, there has been limited work done on examining the effect of shorter finger-joint profiles on joint strength. Adopting a shorter finger-joint length in Canada would result in less waste of good quality lumber and possibly the less use of adhesive, thereby making the product more competitive. The objective of this study was to develop a material-efficient finger-joint profile for structural finger-joined lumber with minimal influence on joint strength for Canadian engineered wood products manufacturers.

2. METHODS

Two bundles of black spruce (Picea mariana) lumber were sampled at the Boise AllJoist Ltd. mill in St-Jacques, New Brunswick, Canada. One bundle comprised of 504 pieces of 2x3 black spruce 1500F5-1.4E MSR lumber, while the second bundle contained 324 pieces of 2x3 black spruce MSR 2250F5-1.9E lumber. The dimensions of each lumber piece were 38 mm x 63 mm x 2.4 m (1.5 in x 2.5 in x 8 ft). The moisture content of lumber sampled was measured, varying from 12% to 19%. All lumber was wrapped and shipped to FPInnovations – Wood Products Division East Laboratory in Quebec City, Canada, for manufacturing finger-joined lumber. The adhesive used was a two-component polyurethane adhesive provided by Ashland Ltd., ISOSET UX-160 and ISOSET WD3-A322.

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⁴ Y. H. Chui, Faculty of Forestry and Environmental Management, University of New Brunswick, P.O. Box 4400, Fredericton, NB, Canada, E3B 5A3. Email: yhc@unb.ca
A new joint profile (UNB-P2) of 12.7 mm (½ in) in length was developed by the authors [1]. A commonly used joint profile (AC16-139) of 29 mm (1-1/8 in) was used as reference [3], Table 1. A Conception RP2000 finger-joining machine was used to produce the specimens. This type of machine is common in the North American finger-joining industry [2]. All finger joints were profiled along the width of lumber to produce horizontal finger joints. Two types of cutters were freshly sharpened. Both produced a reverse finger joint having the profiles. Only three sets of cutters per head were used to continuously fabricate finger-joined lumber in the laboratory scale production line at FPInnovations. The machining parameters for manufacturing finger joints used were the feed speed of 9 m/min, rotation speed of 3500 rpm, chip load of 0.86 mm, cutting speed of 2932 m/min and finger joint knife marks of 1.15 M.P.mm. The maximum segment lumber length used was 0.76 m (30”), which produced at least 4 finger joints over a 2.44 m (8’) long section.

| Table 1: Geometric parameters of finger-joint profiles used |
|---------------------------------|----------------|----------------|
| Number of fingers               | Joint 1       | Joint 2       |
| Finger length (mm)              | 28.27         | 12.70         |
| Tip gap (mm)                    | 0.16          | 0.38          |
| Tip thickness (mm)              | 0.84          | 0.71          |
| Pitch (mm)                      | 6.69          | 3.53          |
| Slope (degrees)                 | 5.07          | 4.76          |
| Contact area (mm²)              | 19922         | 16185         |

After the machining of finger joint profiles, the adhesive was manually applied on each joint at a spread rate of 1.5 to 1.8 grams per joint, which was recommended by Ashland Inc. The two-part adhesive was mixed at the recommended ratio using a mix nozzle. The end pressure was held at 3.43 MPa (498 psi) for 5 to 8 seconds. If lumber specimens failed to be finger-joined during the production line, they were cut by removing the ends and re-joined.

Three variables were considered at two levels, lumber grade (1500Fb-1.4E and 2250Fb-1.9E), joint length (12.7 and 28.6 mm), and cutter sharpness (within 10 and after 60 minutes). Thus, there were 8 groups of 2.44 m (8’) long finger-joined lumber specimens in this study. Each group contained 40 replicates. This gave a total of 320 finger-joined test specimens. Each specimen contained 4 or 5 joints. Two groups of un-joined lumber were used as controls (grades 1500Fb-1.4E and 2250Fb-1.9E).

Lumber properties measured in this study included moisture content, density, modulus of elasticity (MOE), ultimate tensile strength (UTS), and delamination resistance. Density was measured based on oven-dried mass and oven-dried volume, referring to Method B described in ASTM D2395 [4]. The MOE of each specimen was measured using a three-point static bending method with a span of 2.34 m (92 in). Each piece of lumber specimen was tested in tension according to ASTM D198 to determine the ultimate tensile strength (UTS) [5]. Delamination resistance of a specimen is defined as the sum of all the delamination found (excluding the outermost gluelines), divided by the total length of all the gluelines in which the delamination is measured, expressed as a percentage, which is described in Appendix A of NLGA SPS4 [6] and was tested in this study.

3. RESULTS AND DISCUSSION

3.1 JOINT PROFILES

It is reasonable to say that the contact area of a finger joint determines the bonding strength of the joint. However, the authors summarised after an extensive review of technical publications that neither length nor pitch of a finger significantly affects finger joint strength, and the tip thickness and slope govern joint strength to some degree [1]. The authors therefore indicated that a high-strength finger joint can be achieved by designing and adopting relatively flat slopes and sharp tips, and a proper tip gap can improve mechanical performance of a finger joint [1]. Walford pointed out that decreasing the tip thickness in comparison to the reference finger joint requires more precise machining and a more precise alignment of the finger joints when they are pressed together in the crowder machine [7]. In comparison to joint 1, joint 2 reduced the length and the contact area by 55% and 19%, respectively, Table 1. This suggests that possible savings of lumber and adhesive could exist when using joint 2 to fabricate finger-joined lumber.

3.2 DENSITY AND MOISTURE CONTENT

The mean density values of finger-joined lumber tested were 0.470 and 0.519 g/cm³ for 1.4E and 1.9E grades, respectively. The density of 1.9E grade lumber was about 10% higher than that of 1.4E grade one. It was also found that there was no statistically significant difference in density among the groups of the same grade. It should be pointed out that all the lumber pieces sampled passed MSR and visual grading criteria. This suggests that the grouping of lumber among the same grade was quite satisfactory. The mean MC of finger-joined lumber tested was 10.8 and 12.5% for 1.4E and 1.9E grades, respectively.

3.3 MODULUS OF ELASTICITY (MOE)

Figure 1 summarizes the mean MOE values of all ten groups of specimens, indicating that the short finger profile length (1/2”) used could produce similar or even higher MOE values than the commonly adopted long profile (1-1/8”). The ANOVA tests were conducted on finger-joined lumber of two grades. A p-value of 0.1701 (larger than 0.05) from ANOVA on low grade (1.4E) finger-joined lumber indicates that there was no statistically significant difference in mean MOE among the four groups at a confidence level of 95%. A p-value of 0.0123 (less than 0.05) from ANOVA on high grade (1.9E) finger-joined lumber shows that there was some statistically significant difference in mean MOE value.
among the four groups. The test results also suggest that the joint length and cutter sharpness had some effect only on the mean MOE of high grade finger-joined lumber.

Table 3 gives the results of an ANOVA on the UTS of high grade finger-joined lumber (1.9E). The results indicate that there was no statistically significant difference (p-value > 0.05) among the four groups of finger-joined lumber tested at a confidence level of 95%. This could be explained by more uniform quality of the high grade lumber in terms of less strength reducing defects such as large knots, resulting in uniform quality of finger-joined lumber.

Table 3: ANOVA analysis on UTS of finger-joined high grade (1.9E) lumber

<table>
<thead>
<tr>
<th>Source</th>
<th>SS</th>
<th>Df</th>
<th>MS</th>
<th>F</th>
<th>p-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Treatment</td>
<td>20.8926</td>
<td>3</td>
<td>6.96419</td>
<td>0.16</td>
<td>0.9225</td>
</tr>
<tr>
<td>Error</td>
<td>5,930.5177</td>
<td>137</td>
<td>43.28845</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>5,951.4103</td>
<td>140</td>
<td></td>
<td></td>
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</tbody>
</table>

The 5th percentile UTS values were calculated using the non-parameter method. It is interesting to find that the finger-joined lumber of short joint length (1/2") had higher 5th percentile UTS values than that of long joint length (1-1/8") and un-joined control groups regardless of sharpness of cutters and lumber grade.

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<tr>
<td>Total</td>
<td>5,951.4103</td>
<td>140</td>
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</table>


4. CONCLUSIONS

Based the results and discussion, it can be concluded:
(1) The cutters of ½” long finger profile developed could produce the finger-joined lumber having higher mechanical properties (MOE and UTS) than the commonly used cutters (i.e. 1-1/8” long finger profile);
(2) The cutter sharpness tested did not have a significant impact on the MOE and UTS of finger-joined lumber; and
(3) The bonding quality was satisfied in terms of delamination resistance values (less than 5%).

In a word, switching to a ½” from 1-1/8” long finger profile is feasible for fabricating structural finger-joined lumber. There is an on-going project investigating the structural performance of I-joists with finger-joined flange stock lumber made using a short finger joint profile.

It could be a great benefit to Canadian engineered wood products industry by adopting the ½” long joint profile developed in this study. With reducing the joint length from 1-1/8” to ½”, 1.7% of good quality wood materials could be saved when the lumber segment for finger-joined lumber is 6 feet long. Meanwhile, about 20% adhesive could be saved. With decreasing the length of each lumber segment, the saving of good quality lumber will increase.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the financial support from Natural Resources Canada under its Value-to-Wood Program and New Brunswick Innovation Foundation under its Research Assistantship Initiative Program. Thanks are also due to the industry partners: Boise AllJoist Ltd. (St-Jacques, NB), FPInnovations and Ashland Inc. for their support, technical advices and supply of test materials.

REFERENCES

CLASSIFICATION AND SAMPLING OF TROPICAL WOOD SPECIES FOR STRENGTH AND DURABILITY ASSIGNMENTS

Geert Ravenshorst\(^1\), Wolfgang Gard\(^1\), Jan-Willem van de Kuilen\(^{1,2}\)

ABSTRACT: Strength class assignments and durability class assignments of wood species to be used in structures are necessary to make it possible for the engineer to design safe and durable timber structures. As a result of sustainable managed forests, more tropical wood species with relative small batch size, are coming on the market. In Europe, strength class and durability class assignments are allocated to wood species, identified by their botanical name. In practice this gives problems because the trade names may not represent the botanical wood species and the representativeness of the underlying tests is unclear. This object of this paper is to start a discussion on the classification of structural timber for strength and durability. It is proposed to make classifications on measurable characteristics of the timber, not on the tree.

KEYWORDS: tropical wood species, strength classification, durability classification.

1 INTRODUCTION

In Europe, strength class assignments for structural timber based on visual grading are listed in EN 1912 [1], for durability aspects so called ‘Durability classes’ have been established which are listed in EN 350-2[2]. In these assignment it is assumed that the sampling is representative for the timber that is brought on the market. In general, timber originates from tree species (wood species) which are recognised by unique botanical names. It seems, that in practice it is very difficult to determine the wood species. The allocation is related to the material by its trade name and botanical name, and the source area [1] of the wood species which have to be linked. Since the Plant Systematic is based on morphological features of the tree such as flowers, leaves, fruits, etc., these features cannot be used in the later production chain of timber because of their absence. From the nineteen century plant anatomy has attracted attention in relation with wood anatomical features [3]. These anatomical features has been used to identify wood species as far they are distinctive enough. When timber has to be judged at the timber trader’s storage or in a laboratory, the relation with a wood species can only be laid by comparison of wood anatomical features. In many cases trade names cover more than one botanical wood species which cannot always be distinguished by wood anatomical features. This paper which addresses firstly the dilemma of identification of wood species from industrial timber batches which arrives the European market from tropical regions and secondly the representative sampling of wood species originated from tropical forest sources. Two applications have been considered, strength grading and natural durability assignments.

2 IDENTIFICATION AND ORIGIN OF WOOD MATERIAL

Identification of wood species is necessary because wood properties are linked to them.

Lesser-known wood species
Most of the timbers from tropical forests (Asia, South America, Africa) for building applications enter the European Market as sawn timber. There are about 500 wood species which have commercially been used [4]. Almost 80% of these wood species belongs to the group ‘lesser-known’ species. That means that the identification characteristic and/or wood properties are hardly known by the market participants.

Therefor it is likely that ‘lesser-known’ wood species could be mixed with defined timber species in the same batch. Because properties are linked to the species, it would be necessary to identify the species of each piece of the timber batch.

Trade names of timber
Timber is delivered largely in batches under a ‘trade name’. Trade names in particular are assigned by

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individual traders or common names are introduced by the wood industry of single countries. After the tree is harvested, the trunk will be transported to a dispersal area or straight to a sawmill where the primary conversion takes place. Normally different wood species from different origins are processed at the same workstation. From this stage it is possible that the timber could be mixed because of the same secondary visual characteristics. The timber get a trade name which is used during the following process and transport stages until it arrives Europe.

Several attempts have been made to constitute a tabulated overview of common timber trade names in standards with the accompanying scientific name of the wood species [5]. It seems that those lists have a limited validity because of the dynamics of the trade market which changes trade names of wood species and merges wood species under one trade name. The reason for this could be for example the use of secondary identification characteristics such as colour, density and grain texture of the timber but also the decrease of availability of certain wood species and qualities.

During transportation of the timber in the country of origin and/or regions (several countries) the timber can get a different trade names. For example the trade name ‘cumaru’ is used in Brazil for an assortment of at least two wood species Dipteryx odorata (Aubl.)Willd. and Dipteryx alata Vogel, whereas in Venezuela the trade name ‘sarrapia’ is used [6]. To make sure that the assortments with these trade names consists of the same wood species, each piece/board has to be determined by wood anatomical features.

In practice in many cases this is not possible because the anatomical features are not distinctive enough or the needed skills at site are not available. To a great extent this applies to assortments which consist of ‘less known’ wood species.

Wood characteristics
In some cases wood species of the delivered timber cannot be identified at ‘species’ level but be determined at a higher hierarchical level of the plant systematic such as ‘Family’ or ‘Genus’.

For instance the trade name ‘dark red meranti’ includes at least 10 wood species which belong to the Genus Shorea Roxb. ex C.F.Gaertn.. The identifying features are based on Genus level regarding wood anatomical features such as vessels and parenchyma cell distribution and ray cell structure. Even these characteristic are not always sufficient visible to the naked eye or with a magnification of a loupe. Therefor in practice also secondary characteristics such as mentioned earlier have been used for identification of a ‘wood species’.

Figure 1 shows the transverse section of 2 meranti species which can not be distanced by anatomical features such as cell type distribution. But the features are distinctive on genus level.

Timber source area
Dark red meranti which consists of Shorea species originates from South East Asia and covers at least the Philippines Malaysia and Indonesia. In many cases it is not possible to trace the source area.

In general this raises the question how representative test results are for a certain wood species or wood assortments which are widely spread over continents. In some cases wood properties have been determined for certain species from material originating from specific forest stands. In how far these samples are representative for other forest stands for the same wood species cannot clearly be stated. This concerns especially wood species with a wide area of distribution (e.g. tropical South America or Africa).

3 STRENGTH CLASS ASSIGNMENTS
3.1 INTRODUCTION
For strength assignments the test results must be linked to a visual grade for the species. In the sampling the uncertainty in strength influencing parameters should be addressed. These can be divided in three groups: genetics, growing conditions and processing. The sampling these influences must be addressed. The question is how this can be judged when the timber will be tested in a laboratory in Europe. In the laboratory the genus and visual characteristics like knots and grain angle can be determined on the timber. Whether the influences of genetics, growing conditions and processing are addressed in the timber to be tested must be taken care of by the number and place of the source locations. This must ensure that the testing is representative.

3.2 CASE STUDY
As a case 5 samples of timber under the trade name cumaru were studied. In table 1 the locations of the samples and the 5-percentile values of the bending strength is given. This table shows huge variation within and between locations, which cover a large area. Cumaru is related to the wood species Dipteryx odorata (Aubl.) Willd.. Depending on the Plant Systematic the genus Dipteryx consists up to 23 species [7] which are not all distinguishable by wood anatomical features [8]. Therefore in practise cumaru is a mixture of Dipteryx species. For the 5 subsamples the genus could be determined by microscopic research, but not the individual species.
Table 1: 5-percentile bending strength values of 5 cumaru subsamples

<table>
<thead>
<tr>
<th>subsample</th>
<th>Location</th>
<th>5% bending strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Brazil</td>
<td>76.8</td>
</tr>
<tr>
<td>2</td>
<td>Brazil</td>
<td>58.7</td>
</tr>
<tr>
<td>3</td>
<td>Brazil</td>
<td>100.9</td>
</tr>
<tr>
<td>4</td>
<td>Peru</td>
<td>46.0</td>
</tr>
<tr>
<td>5</td>
<td>Bolivia</td>
<td>56.0</td>
</tr>
</tbody>
</table>

3.3 DISCUSSION OF SAMPLING

In Figure 2 the different levels which can influence the strength properties are shown. This figure will be discussed in this paragraph.

The genus level is the level which can be determined by an expert in Europe, based on the structural timber on the market. The next level, the species level, can not be determined in Europe by an expert, so when a genus consists of a large amount of species, this gives extra uncertainty. A source area is determined by the market, this could be a country, more than one country, or a small concession, depending on the expectations of the timber supplier. The sampling locations should be representative for the source area. Different locations can have different growing conditions, different forestry management approaches, and different ways of processing the timber (e.g. sawing patterns). Eventually structural timber from the sampling locations will graded to a visual grade and tested to determine the strength properties.

All the subsamples from table 1 have been assigned to the same visual grade for tropical hardwood according to [9] It is clear from table 1 that the visual grading parameters alone can not explain the variability in the subsamples. The variation therefore must be caused by the variation in in sampling locations and in species. Which of the two are the main cause of the shown variation, the sampling locations or the difference in species can not be distinguished. In the case of table 1, what could be determined that the timber came from the same genus, which and consisted of different species.

3.4 POSSIBLE CLASSIFICATIONS

A possible classification method for timber for strength assignments depends on the information that is available at the different strength influencing levels. When for a species the genus-species combination is very clear, for instance for azobé (Lophira alata) the variability lies within the sampling locations. However, like in the case study mentioned, only the genus can be determined and not the exact species, which is only possible by assessing the tree (see Figure 3). The strength allocation could be done on the genus level. Then in the sampling the occurrence of different species should be required.

4 DURABILITY CLASS ASSIGNMENTS

4.1 INTRODUCTION

Wood durability has been classified in so called Durability Classes (DC) [2]. Durability concerns natural durability against micro-organisms such as fungi. In general, it is assumed that natural durability is strictly connected to a tree species (wood species) and, in some cases, dependent on their origin.

Usually natural durability has been determined by field or laboratory testing against fungi which take several month or even years. Until today there is no method available where natural durability of a single piece of timber can be determined non-destructively. Therefore still the wood species of the timber has to be known to get indicate the natural durability.

For tropical timber, wood anatomical features are often not distinctive enough to identify the wood species in order to assign the wood to a durability class.

Another important aspect of durability assignment of wood species is the representativeness of the population and the reliability of the classified values. Both aspects have been intensively discussed by the members of the European Standardisation Committee CEN TC38 WG21.

Research has shown [10] that durability of a wood species can have large scatter which is not taken into
account in the classification system. These large scatter could be explained by a mix of wood species in the sample or the variation of natural durability within the same species. In order to investigate this, considerable large sample sizes are necessary to get reliable results..

Statistical sampling is concerned with selecting a subset of individuals from a defined population in order to estimate the characteristic of the whole population [11]. However this is not appropriate for wood species which are spread throughout continents.

4.2 NATURAL DURABILITY
Natural Durability of wood is defined in EN 350-1 [12] as ‘The inherent resistance of wood to attack by wood-destroying organisms’. The present classification system does not take into consideration if the wood has been tested in or without ground/soil contact. This can lead to different durability classes of the same wood sample. Furthermore the table in EN 350-2 [2] gives no information about the test method, number of samples and the source (-area) of the tested material. Many assignments of natural durability to wood species in that table relies on historical assessments based on practical experience. This has been applied mainly to tropical hardwood species. In addition the variation of test results have not been reported. Neither the reliability nor the confidence level of the natural durability of the wood species has been reported.

Also the currently accepted test methods for determine natural durability have to be reconsidered if those are still suitable to determine natural durability. One important parameter for decay under application conditions is ‘time’. Most of the test methods considers relatively short time lines in comparison of fungi activity.

4.3 DISCUSSION OF SAMPLING
Sampling for testing and statistical evaluation in order to describe the probability of the natural durability needs high attention if those values are integrated in national building legislations.

This raise an essential question with regard to tropical hardwood which reaches the market as sawn timber: Should the sawn timber be sampled or should the trees be sampled from the source area? And to what growth area must the results be restricted?

Figure 4 shows that first of all a population has to be defined. The next step is to decide either to sample a tree species or a timber species. A timber species is a timber assortment which is commercially supplied under a trade name.

From the practical point of few, for some tropical timber it would be reasonable to sample timber batches. This has to be repeated over time to ensure that the durability of this assortment has not been significantly changed.

4.4 POSSIBLE CLASSIFICATIONS
From practical point of view it is not necessary to identify the tree species if the natural durability of the timber could be determined at each piece in situ. In this case also the origin of the timber has not necessarily to be known. Extractive configurations in the wood are governing to great extend the natural durability of wood [13]. By sophisticated methods such as NIR or other analytical methods a fingerprint of the chemical formation could be taken from each piece of timber. However, these fingerprints have to be tested against fungi resistance. Finally a database would be set up in time where ‘fingerprints’ could be linked to durability classes. For example if tannins configurations have a certain resistance against fungi, then timber with this configuration could be supposed having a certain durability and accordingly to be assigned to the classification system.

5 CONCLUSIONS
This paper is a discussion paper which addresses the existing praxis of sampling wood species for strength grading and natural durability of wood and the reliability and consequences of this.

Currently for both strength grading and natural durability assignments the identification of tropical wood species is necessary. This causes uncertainty because of the wood identification methodology.

Before the sampling of structural timber of tropical hardwoods takes place the uncertainties for the structural timber coming on the market should be assessed. Firstly it should be determined if the wood species can be distinguished, or only the genus. Secondly if the strength allocation is for only a restricted known source, or that it should also be representative for future available sources. Based on the aspect above the sampling of the locations and inclusions of species can be considered. The allocation at the genus level is reasonable in
practice, provided that the different species are taken into account in the sampling. The reliability and the confidence of the assigned natural durability to (tropical-) wood species has to be introduced in the durability classification system. This would give more liberty and flexibility to deal with wood species with a wide scatter of durability. This assumes that also the related test methods have to consider this statistical approach.

To overcome the uncertainty of identifying wood species, novel approaches for both strength grading and natural durability assignments should be invented by using only features that can be measured or determined from the single piece of timber in situ. Hereby the identification of the specific wood species wouldn’t be necessary anymore. Thus the properties which have been determined belong to the population of the timber which is on the market.

REFERENCES


MECHANICAL PROPERTIES OF WOODEN I-JOISTS WITH DIAGONAL PLYWOOD WEBS

Yoshinori Ohashi¹, Susumu Kawamura ², Takuro Hirai ³

ABSTRACT: The experimental evaluation of mechanical properties and creep performance were conducted on wooden I-joists with diagonal plywood webs to improve the shear performance of the I-joists with Japanese softwood materials. Their bending and shear properties were evaluated from bending and shear failure tests. Their creep performance was evaluated from bending creep test. Failure test results showed that the shear stiffness and shear capacity of the diagonal plywood web types was higher than not only those of the perpendicular plywood web types but also those of the particle board web types. The bending creep test results showed that the relative creep of the diagonal plywood web types was smaller than those of the perpendicular plywood web types. These result indicated that the diagonal plywood web could improve the shear performance of the wooden I-joists and reduce the short-term deflection and long-term creep deflection as floor joists.

KEYWORDS: Wooden I-joist, Diagonal plywood, Shear property, Bending creep

1 INTRODUCTION

Wooden I-joists have some advantages such as their light weight and dimensional stability in joist height, which are practically effective for improving the workability and the quality of floor framing of wooden light-frame constructions. Wooden I-joist market in Japan is now potentially expanding because of these advantages, and many house-builders demand the increase in production capacity and high-performance products using Japanese wooden materials. Although only four domestic manufactures are ready to produce commercial I-joists at present.

One of the practical problems for producing the I-beams from domestic wooden materials is that the shear rigidity of Japanese softwood structural plywood expected as the web material, which are mainly made of Japanese cedar and Japanese larch, is lower than those of imported OSB, which are major web material of the I-beams currently produced in Japan [1].

Diagonal plywood is one of the solutions of improving the shear performance of plywood. Early study on wooden I-joists reported the efficiency of diagonal webs in carrying shear stresses [2]. Resent work developed an efficient production method of diagonal plywood [3].

In this study, the trial production of the I-joists with diagonal plywood webs and the experimental evaluation of their bending and shear properties and creep performance were conducted to develop the high-performance I-joists with Japanese softwood materials.

2 EXPERIMENTAL

2.1 MATERIALS

Table 1 shows the specimen number of each I-joist type examined in this study. Figure 1 illustrates the cross-section and types of the I-joists. One of two flange types was kiln-dried Japanese fir lumber finger-jointed according to the Japanese Agricultural Standard for structural lumber for wood en light frame construction. Another flange type was Japanese cedar LVL. Web materials were Japanese larch plywood, Japanese cedar plywood and Japanese particle board. The surface fiber directions of plywood were perpendicular or diagonal to beam span. A resorcinol resin adhesive was used for finger-joints in the flanges.

<table>
<thead>
<tr>
<th>Joist Type Flange-Web</th>
<th>Bending test</th>
<th>Shear test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H=235</td>
<td>H=286</td>
</tr>
<tr>
<td>F-PW(LP)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>F-DPW(LP)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>F-PB</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>C-PW(CP)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>C-DPW(CP)</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

¹ Yoshinori Ohashi, Senior Researcher, Hokkaido Forest Products Research Institute, Asahikawa 071-0198, Hokkaido, Japan. Email: ohasi-yosinori@hro.or.jp
² Susumu Kawamura, Senior Researcher, Shimane Institute for Industrial Technology, Matsue 690-0816, Shimane, Japan. Email: susumuk@fa.mbn.or.jp
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An aqueous vinyl polymer solution isocyanate adhesive was used for web-web joints and flange-web joints. All I-joists for specimens with 4m length were fabricated using a hydraulic batch press. Finger-jointed lumber and LVL for flanges were graded by their dynamic modulus of elasticity and evenly divided into each I-joist type.

2.2 TEST METHODS

Figure 2 illustrates the methods of bending and shear test. In bending test, third points loading test was carried out at 3640mm span. In shear test, midpoint loading test was carried out at 1280mm span. In the both tests, a web joint in longitudinal direction of the beam was arranged at the center between supporting points and loading points. Maximum bending moment ($M_{max}$), pure bending stiffness ($EI$), maximum shear force ($Q_{max}$) and shear stiffness ($GA/\kappa$) were calculated from their observed data.

Web material tests such as edgewise out-of-plane bending test and panel shear test were conducted for investigating their material properties of web board. Effect of diagonal plywood on bending creep of I-joist was also investigated in this study. Figure 3 illustrates the method of bending creep test. The four loading equipments with 2550mm and 3720mm test span were located in a laboratory at Shimane Institute for Industrial Technology. All creep tests were carried out under controlled temperature and uncontrolled relative humidity for 33 months from May 2009 to February 2012. The temperature and RH in the laboratory were monitored using a hygro-thermograph. The applied load to specimen was 5kN. Displacements at mid-span were monitored using digital displacement transducer at the top face of upper flange. The specimens for creep test were fabricated using Japanese Cedar LVL and Japanese Cedar Plywood (PW and DPW).

3 RESULTS AND DISCUSSIONS

3.1 BENDING AND SHEAR FAILURE TESTS

Figure 4 shows the typical failure modes in the bending and shear tests. The failure modes observed in bending tests were almost tensile failure in the bottom flange. The typical failure mode observed in shear tests were shear failure at the web joint and embedding failure of the flange at the loading point or supporting point.
Figure 5 shows the typical Load-Deflection and Strain in bending test.

Table 2 shows the results of in-plane shear tests of wood materials.

<table>
<thead>
<tr>
<th>Type</th>
<th>$f_{ps}$ (MPa)</th>
<th>$G_{ps}$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AV</td>
<td>CV</td>
</tr>
<tr>
<td>PW(LP)</td>
<td>4.87</td>
<td>0.08</td>
</tr>
<tr>
<td>DPW(LP)</td>
<td>7.12</td>
<td>0.16</td>
</tr>
<tr>
<td>PW(CP)</td>
<td>4.19</td>
<td>0.06</td>
</tr>
<tr>
<td>DPW(CP)</td>
<td>5.02</td>
<td>0.10</td>
</tr>
<tr>
<td>PB</td>
<td>7.14</td>
<td>0.04</td>
</tr>
<tr>
<td>OSB</td>
<td>6.66</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Table 2 shows the results of in-plane shear tests. These results show that the diagonal plywood has a significant effect on the in-plane shear properties and even softwood plywood has equal or higher shear properties compared to PB and OSB.

Table 3 shows the results of the mechanical properties obtained from bending and shear failure tests. In comparison of the average shear strength, $M_{max}$, and $EI$ of F-DPW and C-DPW were equal or less than those of F-PW and C-PW. In contrast, $Q_{max}$ and $G\kappa$ of F-DPW and C-DPW were significantly higher than those of F-PW and C-PW except for $Q_{max}$ of F-DPW with 286mm height because one specimen broke weakly at the web-joint. Using these $EI$ and $G\kappa$, the mid-span deflections were calculated in the practical load conditions for residential floor (Table 3). The calculated mid-span deflections ($\delta_{cal}$) of F-DPW and C-DPW were lower about 10% than those of F-PW and C-PW. And then the ratios of shear component in mid-span deflection of F-DPW and C-DPW were substantially smaller than those of F-PW and C-PW. These results show that the diagonal plywood could improve the shear properties and bending deflection of I-joists.

Table 3: Mechanical properties of I-joists obtained from bending and shear failure tests

| Type  | $W$ (N/m) | $M_{max}$ (kN·m) | $EI$ (kN·m²) | $Q_{max}$ (kN) | $G\kappa$ (kN/m²) | $\delta_{cal}$ (mm) | $R_s$ (%) | $W$ (N/m) | $M_{max}$ (kN·m) | $EI$ (kN·m²) | $Q_{max}$ (kN) | $G\kappa$ (kN/m²) | $\delta_{cal}$ (mm) | $R_s$ (%) |
|-------|-----------|------------------|--------------|---------------|-------------------|---------------------|-----------|-----------|------------------|--------------|---------------|-------------------|---------------------|-----------|-----------|
| F-PW  | AV        | 28.6             | 10.2         | 463           | 12.0              | 1347                | 6.3        | 19.9      | 31.7             | 11.8         | 693            | 15.7              | 1860                | 4.3        | 21.3      |
|       | CV        | 0.14             | 0.11         | 0.03          | 0.06              |                      |            |           | 0.20             | 0.11         | 0.01           | 0.01              | 0.05                  |           |           |
| F-DPW | AV        | 28.9             | 9.9          | 439           | 17.3              | 5197                | 5.7        | 5.8       | 30.5             | 12.3         | 671            | 16.7              | 5045                | 3.8        | 8.8       |
|       | CV        | 0.14             | 0.12         | 0.06          | 0.15              |                      |            |           | 0.16             | 0.10         | 0.27           | 0.16              |                      |           |           |
| F-PB  | AV        | 31.9             | 9.8          | 431           | 16.3              | 2672                | 6.1        | 10.5      | 36.6             | 12.4         | 704            | 18.2              | 3441                | 3.8        | 12.9      |
|       | CV        | 0.24             | 0.06         | 0.04          | 0.10              |                      |            |           | 0.19             | 0.10         | 0.01           | 0.01              |                      |           |           |
| C-PW  | AV        | 26.8             | 9.1          | 408           | 10.6              | 1064                | 7.3        | 21.8      | 30.6             | 12.3         | 660            | 13.8              | 1468                | 4.7        | 24.6      |
|       | CV        | 0.18             | 0.15         | 0.03          | 0.02              |                      |            |           | 0.12             | 0.12         | 0.04           | 0.04              |                      |           |           |
| C-DPW | AV        | 27.8             | 9.0          | 413           | 16.2              | 3803                | 6.1        | 7.3       | 29.9             | 11.6         | 623            | 16.7              | 4900                | 4.1        | 8.4       |
|       | CV        | 0.14             | 0.14         | 0.07          | 0.10              |                      |            |           | 0.07             | 0.11         | 0.05           | 0.14              |                      |           |           |

AV: Average; CV: Coefficient of variation; H: Joist Height; W: Joist Weight; $M_{max}$: Maximum bending moment in bending test; $EI$: Pure bending stiffness in bending test; $Q_{max}$: Maximum shear force in shear test; $G\kappa$: Shear stiffness in shear test; $\delta_{cal}$: Calculated deflection at mid-point of 3640mm span (Joist pitch=455mm, Uniform distributed load=2250N/m²); $R_s$: Ratio of shear component included in $\delta_{cal}$.
3.2 BENDING CREEP TESTS

In this study, creep deflections were estimated by following method of the Japanese building code [4] to predict a creep deflection after 50 years load duration.

\[
\delta_{0/50y} = 10^e t^f
\]

where \(\delta_0\) = initial deflection at the moment of loading (mm), \(\delta_{50y}\) = deflection at mid-span after t minutes from the start of loading (mm), \(t = \) elapsed time (min), e and f = constants.

These constants were calculated by linear regression analysis of \(\log_{10}(\delta_0/\delta_{50y})\) and \(\log_{10}t\).

As already reported [5], the data within 24 hours after all data, the regression analysis was applied to the partial data after 24 hours from the loading start in this study.

Figure 7 shows the relative creep of observed data (plot) and estimated value (solid line) by Eq.(1). The observed relative creep fluctuated in response to the RH. They periodically increased during dry winter and decreased during humid summer. This creep behaviour with desorption, so-called “mechano-sorptive creep”, was observed in all types. The estimated relative creep bordered on the centreline of each observed data curve.

Table 4 shows the relative creep after 50 years load duration estimated by Eq.(1).

The estimated values of \(C_{50y}\) with 2550mm span were larger than those of \(C_{50y}\) with 3720mm span. The estimated values of \(C_{50y}\) of DPW were smaller than those of PW in both span length. This result indicates that the diagonal plywood could reduce long-term bending creep.

4 CONCLUSIONS

In this study, the experimental evaluation of mechanical properties and bending creep performance were conducted on wooden I-joists with diagonal plywood webs. The following conclusions were obtained.

The results of bending and shear failure tests showed that the shear stiffness and shear capacity of the diagonal plywood web types was higher than not only those of the perpendicular plywood web types but also those of the particle board web types.

The bending creep test results showed that the relative creep of the diagonal plywood web types was smaller than those of the perpendicular plywood web types.

These result indicated that the diagonal plywood web could improve the shear performance of the wooden I-joists and reduce the short-term deflection and long-term creep deflection as floor joists.

This study indicated the improvement of performance of wooden I-joists without detriment to lightness and dimensional stability, and also the utility of plantation timber materials in Japan.

REFERENCES

THE POTENTIAL OF ASPEN TIMBER IN NORDIC WOODEN BUILDINGS

Harri Metsälä

ABSTRACT: This study presents the changes in the use of aspen (*Populus tremula* L.) in wooden buildings during the last century. By analyzing the recorded change and the species-based characteristics of the European aspen the potential expansion of the rather limited present use is detected. The traditional use of aspen timber in building in Finland has been based on the weather resistance, pleasant surface feel and appealing appearance of the tree species. Aspen has been utilized in log-walls, claddings, sauna interiors and in some special uses in ground constructions e.g. The aim of this article is to present the unutilized potential of aspen timber in contemporary building. The survey results show that aspen is sold almost with no exception to sauna interiors as wall panels, lists and seating planks. The majority of the products are imported ready-made from the Baltic region. Half of the sales consist of untreated aspen, half of thermo-treated aspen.

KEYWORDS: European aspen, wood in architecture.

1 INTRODUCTION

The European aspen (*Populus tremula* L.) is the sole native member of the poplar family in Finland. On the other hand, aspen is one of the most widely ranging tree species on the face of the Earth, its habitat covering most of Eurasia. The aspen is clearly outnumbered in the Finnish forests by the three main species, the Scots pine, the Norway spruce and the birch(es). Aspen reserves form a mere percentage of 1,5 % of the total timber volume. However, the annual exploitation of the aspen resources is below the annual growth and the average exploitation grade of birch.

The European aspen has been attacked by bacterial canker in the Central Europe. It has also been regarded as a less valuable wood for building. From the poplar family they have found other members, such as the white poplar and the black poplar instead. In the Nordic countries aspen does not suffer from bacterial canker and has a better overall quality than the trees further south.

Of all the deciduous tree species in Finland aspen has been the most widely used one in traditional vernacular building. Despite this fact, the use of aspen must be regarded as marginal. This is due to the unquestionable status of the Scots pine as the number one source of building timber in our country throughout history. The pine is followed by Norway spruce, both in the size of the timber stands in the forests and in its use as a building material.

There are no difficulties in marketing top quality knotless aspen panelling for interiors, or bulk class timber for pallets or chipboards. The value-added development of aspen products, however, requires a better understanding of the properties and possibilities of the species.

Picture 1. The European aspen.

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1 Harri Metsälä, Wood Architecture, Aalto University, Kauppilantie 33, 36430 Sahalahti, Finland. Email: harri.metsala@gmail.com
2 CHANGES IN THE USE OF ASPEN

2.1 TRADITIONAL VERNACULAR USE OF ASPEN TIMBER

In the vernacular Finnish architecture aspen has been the most widely used hardwood species. Stems meant for building have been cut in late summer – in contrast to the conifers, that were felled as a rule in midwinter. The branches were left unlopped until the leaves got brown and dry. The logs were sawn or hewn for timber during the winter, or better still, the winter thereafter. Aspen is quite suitable for log buildings. For one reason or another, it has not been preferred for residential buildings, but has had a minor role as the raw material for cold outbuildings, such as far-away hay-barns, and lofts and saunas within the courtyard.

![Picture 2. Aspen log sauna.](image)

Although the use of aspen in building is known throughout its distribution in Finland, it has not been the number one choice for logwood anywhere – probably just for being so clearly outnumbered by the conifers in our forests.

As planks and boards aspen has been used even more rarely. In the outdoor use it has been appreciated for its durability. The main applications have been the underlay planks for roof shingles, wall claddings for barns, fences etc. Indoors has aspen served mainly as floors and especially as panellings and fittings in saunas. Floors made of aspen have been praised for their warmth and good holding of paint.[1].

One special target for aspen planks has been the stable floor. It has held very well under the wear of horse hoofs and in the moist of the animal shed. This particular use for aspen has been reported also in Norway [2].

The best known use of aspen even nowadays is as roof shingles, both the hewn large church shingles and the thin, machine-made ones. Aspen shingles work considerably with the changes in air humidity, but despite this aesthetic deficiency they have proven to last long. Outside the very building aspen has been given tasks as steps and stairs, jetties and bridge poles and decks as well as water pipes.

2.2 CONTEMPORARY USE OF ASPEN

The traditional fields of application for aspen were mainly based on its recorded weather resistance, resulting thus as a rule in outdoor use. With the industrialization of the building branch these uses became forgotten as the conifers gained the monopoly as construction timber. The use of aspen was reduced for decades to sauna seating planks only. The situation changed during the 1990es, mainly with new trends in interior decorating.

The new product oriented thinking in the timberyards and timber-selling hardware business has produced a variety of ready-made interior fittings and concepts for the consumers. Today aspen is sold almost with no exception to sauna interiors as wall panels, lists and seating planks. The majority of the products are imported ready-made from the Baltic region. Half of the sales consist of untreated aspen, half of thermo-treated aspen. The vendors trust for a slight increase in the market share of aspen. [5].

![Picture 3. Sauna interior with aspen fittings.](image)
modern churches and other public buildings in Finland have been faced with aspen shingles. The obvious intention of the material choice is to remind of the tradition of wooden churches with their dark tarred roofs, and to create a distinctive contrast to the surroundings which express the architecture of hard business and industrial potential.

*Picture 4. Aspen shingles cover the facades of Viikki church in Helsinki built 2004. They are made in the traditional splitting technique to gain the best weather resistance.*

3 PROPERTIES OF ASPEN TIMBER

Similarly with all other white poplars, the wood of aspen is remarkably white in colour, with no visible difference between the sapwood and the heart. The wood is light weight, tough, nails without splitting, glues and embosses well, wears smooth and machines easily. The wood is very absorbent when dry. Fresh aspen wood has a characteristic smell, but becomes odourless after drying. Aspen wood is remarkably homogeneous visually, structurally and technically.[1,3,4]

As weaknesses of aspen there should be mentioned the prevalent stem rot and the common occurrence of rotten knots. The drying of aspen has also shown to be a challenge as the wood is not even in its moisture content and it often has inner tensions that cause splitting and deformations. This has come out particularly in connection with the thermowood. If the tools that aspen is treated with are not sharp enough, the appearance will also be coarse and fluffy. [3,4]

**Table 1:** Some technical properties of aspen and spruce. [3].

<table>
<thead>
<tr>
<th>Property</th>
<th><em>Populus tremula</em></th>
<th><em>Picea abies</em></th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity (ε), %</td>
<td>70</td>
<td>71</td>
</tr>
<tr>
<td>Density (r₀), kg/m³</td>
<td>396</td>
<td>378</td>
</tr>
<tr>
<td>Shrinkage %, longitudinal</td>
<td>0,3</td>
<td>0,3</td>
</tr>
<tr>
<td>radial</td>
<td>3,5</td>
<td>3,5 ... 3,7</td>
</tr>
<tr>
<td>tangential</td>
<td>6,7 ... 8,5</td>
<td>7,8 ... 8,0</td>
</tr>
<tr>
<td>volume</td>
<td>11,0 ... 12,8</td>
<td>11,6 ... 12,0</td>
</tr>
<tr>
<td>Static strength, Mpa,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- bending (s_{Bb})</td>
<td>82</td>
<td>84</td>
</tr>
<tr>
<td>- compression (s_{Dc})</td>
<td>42</td>
<td>44</td>
</tr>
<tr>
<td>- shear (s_{LII})</td>
<td>75</td>
<td>21 ... 90 ... 245</td>
</tr>
<tr>
<td>- tension (s_{Am})</td>
<td>5,7</td>
<td>6,8</td>
</tr>
<tr>
<td>Elasticity, (E-JI), GPa</td>
<td>13,2</td>
<td>13,4</td>
</tr>
<tr>
<td>Brinell hardness, MPa</td>
<td>11</td>
<td>12</td>
</tr>
</tbody>
</table>

There was a significant increase in the interest towards aspen and its hybrids in the turn of the millennium caused by new technologies in paper making based on aspen fibre. The opening of Paper Mill 3 in Kirkniemi made the demand of aspen timber rise above the domestic supply and lead to remarkable import from Russia and the Baltic region. Simultaneously there rose a variety of research programs both in the fibre studies and in genetic and plant studies. As markets for pulp and paper got into difficulties after 2005, little interest has been showed to aspen in resent years. The mechanical properties of aspen for building were neglected in the mentioned research programs. The surface properties of aspen were preliminarily studied by the author (reported in WCTE 2004 proceedings [6]), but further research is needed in order to exploit the full potential of the species.

*Picture 5. Sawn surface of aspen in SEM-view. [6]*
4 CONCLUSION: THE UNUSED POTENTIAL OF ASPEN

The traditional use of aspen timber in vernacular architecture was based mainly on the weather resistance of the raw material in normal circumstances. It is to believe, that the benefits of aspen were not regarded as great enough to resist the raising monopoly of the conifer-originated timber coming from large sawmills. The tradition of the outdoor use of aspen has been restricted to museological restorations of roof shingles on old churches.

The recent trend of using aspen, thermally treated aspen and alder in sauna interiors has brought up new markets for aspen timber. This phenomenon has nothing to do with the traditional use, though. It is a separate event in the interior decorating branch, and may as well turn out to be a temporary one.

Table 2. SWOT-analysis based on architects’ views according to a survey made by the author.

<table>
<thead>
<tr>
<th>STRENGTHS:</th>
<th>WEAKNESSES:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Domestic</td>
<td>Difficult to machine</td>
</tr>
<tr>
<td>Vernacular</td>
<td>Rot defects</td>
</tr>
<tr>
<td>Weather resistant</td>
<td>Wringing</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>OPPORTUNITIES:</th>
<th>THREATS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Profitable for planting</td>
<td>Problems in availability</td>
</tr>
<tr>
<td>Interesting</td>
<td>Lack of R&amp;D</td>
</tr>
<tr>
<td>Has potential!</td>
<td>Lack of know-how</td>
</tr>
</tbody>
</table>

Together with the few examples of the adaptation of aspen shingles to modern facades, the interior panelling business can be regarded as a true possibility to develop a serious industry based on aspen timber. This requires R&D –actions on the branch to create products for utilizing all the timber of the aspen logs. In this development the vernacular traditional know-how can be exploited.

In the surveys made with retail dealers of timber and architects respectively, it came out that the main properties and traditional uses of aspen were reasonably well known even today. The respondents felt however that they lacked information and know-how in order to be able or confident to use aspen besides the main stream applications in interior panels and sauna fittings. The attitudes towards aspen were on the other hand very positive and there was a clear willingness to widen the range of aspen use. These surveys back up strongly a need for enhanced academic and field studies on aspen as a building material.

REFERENCES

SPECIFICATION OF FEM MODELS OF GLUED LAMINATED TIMBER WITH VARIABLE LOCAL MODULUS OF ELASTICITY

Lenka Melzerová¹, Petr Kuklík², Michal Šejnoha³

ABSTRACT: The present paper is concerned with the formulation of advanced FEM based models of beams made from glued timber segments. These models account for variable elastic moduli in individual segments and their analysis is based on the application of LHS method. All results from probabilistic calculations are compared with experimental measurements conducted on twenty beams as well as with the FEM results derived for the same beams assuming deterministic analysis with piecewise constant moduli in individual segments. The main contribution of enhanced probabilistic models is seen primarily in the ability to provide cost effective designs of long-span glued timber structures.

KEYWORDS: Displacement, Glued laminated timber, Local modulus of elasticity

1 INTRODUCTION

The present contribution builds upon an extensive experimental program examining the behavior of glued laminated timber beams. Twenty beams were tested at the Department of steel and timber structures of the Faculty of Civil Engineering in Prague. Two types of experiments were conducted. First, non-destructive measurements were performed to measure the elastic moduli of timber in the fiber direction at 1448 locations while monitoring the current state of moisture. The second type of experiments, performed on twenty beams, corresponds to destructive four-point bending tests with the option to measure various parameters with principal attention accorded to deflection at the center of beams. This parameter is not only decisive from the engineering practice point of view, since the limit deflection is typically reached prior to exceeding the bearing capacity, but it also serves as the most objective measure of the behavior of strongly heterogeneous materials such as the analyzed glued laminated timber. Both types of experiments will be reviewed in the first part of this text. The second part is then concerned with the finite element (FE) simulation of these experiments including the introduction of material uncertainty through variable Young’s modulus. The first series of calculations assumes constant moduli assigned to individual segments as averages of values measured for a given segment. The numerical results show a relatively good agreement of this deterministic approach with experiments. The next part of the paper then deals with probabilistic simulations of the same beams assigning to each segment of the beam Young’s modulus with a given probability of distribution. Individual samples (realizations), eventually providing the probability density function or the distribution function of the maximal deflection, were generated using the Latin Hypercube Sampling (LHS) method. The results from the three approaches – experiment, deterministic and probabilistic numerical analyses – are compared next for two levels of the applied load. The first reference load level was accommodated by all twenty experimentally examined beams with no failure. For this load level the deterministic analysis shows a very good agreement with experiments. Probabilistic simulations with variable moduli provide even better predictions. The second load level corresponds to the maximal load which when exceeded leads to failure destruction of the beam. The results show a similar trend with improved predictive power of probabilistic simulations. Nevertheless, the agreement with experimental results is less satisfactory, which can be attributed to the initiation of cracks prior to the

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catastrophic failure, which was not be reflected by purely elastic FEM simulations. The paper closes by presenting a general methodology for the preparation of the FEM models of the glued timber beams with variable moduli of elasticity and application of the methodology in practice for exceptional long-span structures.

2 EXPERIMENTS

As already mentioned in the introductory part the experimental program assumed two types of experiments to examine the behavior of glued laminated timber. An illustrative example of a glued laminated timber beam appears in Figure 1. In general, such a beam consists of an arbitrary number of segments glued together over their entire area to form a layered structure. The present example assumed eight layers (lamellas). Our previous work was also concerned with the FEM modeling of the gluing effect by introducing an additional layer of glue along the longitudinal joints of individual segments thus allowing for the evolution of progressive delamination. However, based on experimental observations, suggesting no damage within these joints due to delamination when exceeding the overall bearing capacity, this approach was not pursued any further and a perfect bonding was assumed in the present study. The onset of cracking was detected in the locations of various flaws such as knots and not within the glued joints. No interface slip was also confirmed by the resulting deflections. Therefore, our attention was shifted to the examination of random nature of such structures as described in the next sections [2].

\[ E = -564.1 \times t_p + 19367 \]  

(1)

The experimentally derived values were checked first approximately (globally) by comparing an expected maximal deflection with that provided by an independent bending test and second more rigorously by measuring local strains with the help of strain gauges applied at several locations of high stresses developed during the bending test. Given an approximate value of the local stress calculated for a homogeneous body the modulus of elasticity can then be determined from Hook’s law and compared with the corresponding measured value. The four point bending test is schematically shown in Figure 3 and documented by photographs in Figure 4. The center beam deflection was measured by a

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displacement sensor corrected for support compression by installing two other sensors at their vicinity. The loading was supplied by two forces at one third of the beam span through two cylinders of a loading press.

Figure 3: Sketch of four-point bending test

A 40 cm long steel plate was placed below each force to allow for the introduction of a distributed loading as also assumed in the numerical tests. Both forces were gradually increased up to complete failure with a loading step equal to 4 kN. Each load step followed by a hold period to collect the data from all measuring sensors. The failure load for individual beams experienced a considerable scatter from 30 to 60 kN.

The measured deflections, adopted in this study for the sake of comparison, were statistically evaluated for two different load levels. The results for the first load level of 24 kN for each force corresponding to 60kN/m uniformly distributed load are plotted in Figures 5 and 6. Figure 5 provides the Gaussian probability density function estimated from 20 measurements. Figure 6 then shows the corresponding distribution function[1].

3 DETERMINISTIC MODELING USING FEM

The resulting elastic moduli delivered by experimental measurements were already evaluated in the previous section assuming all 1448 measured values.

Figure 7: Comparison of probability density functions of Young’s modulus E derived from all 1448 measured data and from values averaged over individual segments

Figure 8: Comparison of distribution functions of Young’s modulus E derived from all 1448 measured data and from values averaged over individual segments

Deterministic FEM based modeling discussed in this section requires, however, averaging these values over individual segments yielding a new set of data, which can also be statistically evaluated.
The results appear in Figures 7 and 8 (solid line in red color corresponds to all 1448 measurements) suggesting higher uncertainty when taking into account all measurements and possible misrepresentation of input data associated with the used local averages of Young’s moduli.

![Figure 9: Illustrative example of FEM model](image)

The results presented in this and the subsequent sections are derived on the basis of several simplifying assumptions including two-dimensional (2D) plane stress analysis and piecewise isotropic material. It has been found that errors associated with these simplifications when compared to the predictions provided by a three-dimensional analysis while also accounting for a material orthotropy are rather negligible and are of the order of magnitude smaller in comparison with measurement errors arising in experiments.

![Figure 10: Distribution of averages of Young’s moduli E within the beam with average deflection of 18.88 mm](image)

The adopted computational model is plotted in Figure 9 also showing relatively fine finite element mesh. To further appreciate a structural heterogeneity due to different elastic moduli in individual segments as well as structural variability of individual beams we also show a concrete pattern of segments for three selected beams. Figure 10 corresponds to an average beam rendering the maximal deflection close to the average value calculated from all twenty specimens.

![Figure 11: Distribution of averages of Young’s moduli E within the beam with large deflection of 21.7 mm](image)

Figure 11 illustrates an example of a beam having a large number of segments with low moduli of elasticity which are not even arranged in favor of the static response thus yielding a large deflection.

![Figure 12: Distribution of averages of Young’s moduli E within the beam with small deflection of 16.87 mm](image)

Contrary to that, Figure 12 displays an example of a majority of segments having the above average value of Young’s modulus, which are randomly arranged such as to provide one of the stiffest response. Notice that segments with lower moduli of elasticity are found in the vicinity of neutral axes, whilst segments with higher moduli are located in outer lamellas. Also recall that the presented deflections correspond to a uniform load of 60 kN/m.

4 PROBABILISTIC MODELING USING FEM

The advanced FEM models employ probabilistic simulations performed in the framework of LHS method.

![Figure 13: Illustration of the input data used in the LHS method](image)

In the light of this, each segment is assigned Young’s modulus with a corresponding probability density function. In all cases the Gaussian distribution with the given mean and standard deviation is assumed as seen in Figure 13.

![Figure 14: Principle of selecting the k-th sample in the LHS method](image)

The associated distribution function is then utilized to generate individual samples. In the present study the distribution function was split into 100 intervals to randomly select a single value $kE$ as schematically shown in Figure 14.

![Figure 15: Resulting maximal deflections for a single beam from one hundred realizations](image)

This result is in accord with the LHS method based on 100 strata [1,2]. The resulting map of realizations, see
Table 1, is constructed such as to comply with a statistical independence of elastic moduli from segment to segment. Note that selecting lamellas to form a beam is conducted in a totally random manner. Figure 15 shows a variation of maximal deflections from 100 samples derived for a single beam with a given pattern of segments.

These results can be statistically evaluated and fitted to the selected probability density function as illustrated in Figure 16 for the Gaussian distribution.

Table 1: Example of creating individual realizations using the LHS method for a beam with 18 segments and 100 strata

| Segment 1 | $E_1$ | $E_1$ | $E_2$ | $E_2$ | ... | $E_1$ | $E_2$ |
|-----------|-------|-------|-------|-------|     |       |       |
| Segment 2 | $E_2$ | $E_1$ | $E_2$ | $E_2$ | ... | $E_2$ | $E_1$ |
|           |       |       |       |       |     |       |       |
| Segment 18| $E_1$ | $E_1$ | $E_1$ | $E_1$ | ... | $E_1$ | $E_1$ |
| Beam for run 1 | Beam for run 2 | Beam for run 100 |

5 COMPARING OBTAINED RESULTS FROM FEM SIMULATIONS AND EXPERIMENTS AND THEIR EVALUATIONS

This section compares the results provided by individual methods.

The results derived from the deterministic FEM modeling for the uniformly distributed load level equal to 60 kN/m are compared with the corresponding experimental values in Figure 17 for all twenty specimens.

Figure 16: Example of the Gaussian probability density function of deflection for the selected beam

These results can be statistically evaluated and fitted to the selected probability density function as illustrated in Figure 16 for the Gaussian distribution.

Table 1: Example of creating individual realizations using the LHS method for a beam with 18 segments and 100 strata

Figure 17: Comparison of measured and numerically derived deflection using deterministic FEM model applied to 20 beams each loaded by 60 (kN/m)

It is evident that the resulting differences experience both positive and negative sign not showing a unique pattern.

Figure 18: Comparison of measured and numerically derived deflection using deterministic FEM model applied to 20 beams loaded by their maximal loading

This is in contradiction to the plot in Figure 18 presented for a maximal loading pertinent to each beam where the FEM results are consistently below the measured values.

Figure 19: Comparison of Gaussian probability density functions of both measured and calculated deflections from the ensemble provided by all 20 beams

This is what one would expect, since in reality the beams may witness evolution of local damage even prior to reaching an ultimate load not addressed by the elastic FEM analysis. From the absolute value point of view the differences are, however, in the same range as in the case
of elastic loading (Figure 17), note two different scales of vertical abscissa in Figures 17 and 18. Henceforth, attention will be dedicated to the results provided by probabilistic simulations.

![Figure 21: Comparison of measured and calculated deflections (the circles show averages from 100 realizations obtained for individual beams)](image)

To compare individual approaches (experiment, deterministic and probabilistic modeling) a single value given by the averages obtained from 100 samples, see also Figure 16, will be adopted. This appears in Table 2 suggesting in such a case no need for more advanced and computationally exhausted probabilistic simulations. It might be, however, expected that a better agreement with experimental results will be obtained with improved probabilistic data of input parameters conditioned by considerably more measurements in individual segments (recall that only four measurements are presently available for each segment). Probability of not exceeding a certain limit deflection is even more important than a simple mean, provided by probabilistic simulations.

### Table 2: Comparison of measured and numerically derived deflections for the selected beam

<table>
<thead>
<tr>
<th>Beam</th>
<th>w (mm)</th>
<th>Percent of measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured</td>
<td>19.15</td>
<td>100</td>
</tr>
<tr>
<td>Discrete</td>
<td>18.8</td>
<td>98.17</td>
</tr>
<tr>
<td>FEM</td>
<td>18.83</td>
<td>98.34</td>
</tr>
</tbody>
</table>

Figures 19 and 20 then compare the probability density functions and the distribution functions, respectively, estimated from the results provided by all 20 specimens (in case of probabilistic modeling we assumed the ensemble of 20 averages of maximal deflections). The variations of maximal deflections in Figure 17 are finally re-plotted in Figure 21 showing also the comparison with the averages delivered by the probabilistic analysis. Clearly, when comparing only averages the difference between deterministic and probabilistic modeling is almost negligible. Recall, however, that above each mean value one should image a particular distribution, \( f_{W,n}(w) \), as also schematically shown in Figure 21. This allows us to estimate the probability of exceeding a certain level of the assumed allowable deflection of the beam \( w \) as

\[
P(w > \bar{w}) = 1 - \frac{1}{N} \sum_{n=1}^{N} F_{W,n}(w),
\]

where \( N \) is the number of beams and \( F_{W,n} \) is the corresponding distribution function of the deflection of the \( n \)-th beam.

6 CONCLUSIONS

The presented results demonstrated that a certain improvement in the prediction of the response of glued timber beams can be achieved by extending the deterministic modeling to allow for a variability of input parameters in the framework of probabilistic simulation. However, the degree of improvement strongly depends on the quality of input parameters being in turn dependent on the number of available laboratory measurements. The actual computational methodology is nevertheless independent of such data. Also, it is not surprising that the results from the two approaches are rather similar since compared on the basis of averages only. Information provided by the stochastic analysis is, however, significantly broader, recall Equation (2).

The proposed technology of determining deflection and stress or strain distributions assumes synergy of experimental measurements of local moduli and FEM based analysis of a given structure with random material data. Although verified on a relatively small set of twenty beams its practical applicability is expected in the design of unique nonstandard glued laminated timber structures.

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REFERENCES


Mode I fracture toughness for crack initiation by CT tests and three-pointed-bending tests with structural glulam

Wataru Kambe¹, Naoyuki Itagaki², Yasuo Iijima³ and Tadao Nakagomi⁴

ABSTRACT: Mode I fracture criteria loaded perpendicular to the grain would be obtained from 3-pointed bending test in this paper. These specimens are modelled by FEM and we can estimate the fracture criteria with fem analysis and numerical method. These values are almost the same with the value from CT tests. As a result, we can obtain Mode I fracture criteria by these two methods.

KEYWORDS: Fracture Toughness, Crack Initiation, CT Tests, Three-pointed-bending Tests, Structural Glulam

1 INTRODUCTION

For timber structure external-loaded, we would see its connections are failed. At that time in those connections, fasteners would be failed by bending, shear or embedding stress, additionally the wood around the fasteners are fractured. In the standard for structural design of timber structures in Japan [1] or in some standards, the equation based on European Yield Theory [2] or the equation based on the study by Put and Leijiten for fracture loaded perpendicular to the grain [3] are indicated. As known in the world, the former equation can estimate its yield strength loaded parallel to the grain or perpendicular to the grain considering bending or embedding deformation of fasteners. The latter equation is useful to evaluate the ultimate strength of bolted connections loaded perpendicular to the grain [3]. As point to notice, we can see some fracture of wood in connection at the earlier stage than ultimate condition in the laboratory tests. However we couldn’t evaluate the strength of crack-initiation in joints. Then for analyzing that fracture and improving the precision of design equation, a new fracture criterion or standard test methods would be needed. And we should propose a new calculation method of connection-strength with above criterion.

In the past study, we observed crack initiation loaded perpendicular to the grain in CT tests and proposed determination method of fracture toughness for crack initiation [4]. Additionally by finite element methods, that criterion was applied to estimate the crack-initiation-strength of bolted connection loaded perpendicular to the grain [5]. When the thickness of the specimens would get large, the condition of a specimen would get unstable, so there is a problem of scale limitation, it is pointed in some article. In another word, there are some problems about fracture of wood in joint of timber engineering.

In this study, we conducted three-pointed-bending tests (hereafter “3PB tests”) referred to the Nordtest methods [6] and “Standard method of test or elastic-plastic fracture toughness $J_{IC}$” normalized by Japan Society of Mechanical Engineering (hereafter “JSME”) [7]. According to the latter standard, we could get the fracture toughness of a metal specimen from CT and 3PB tests. However in wood materials, such standard or study haven’t ever seen. Then with those tests we would observe the fracture behaviour and from the test results with the equation in that standard we got the fracture toughness. Additionally we compared two types of $J$-integral from 3PB and CT tests.

2 TEST METHODS

For the test methods we adopted 3PB tests with three species of wood, Scots Pine (Pinus sylvestris Linn), Japanese Larch (Larix kaempferi Carriere) and Japanese Cedar (Cryptomeria Japonica D.Don). At that time, we had following three steps in this study,

1) 3PB tests with Japanese Cedar and analysis with FEM
   For judgement of the adequacy of the equation JSME equation compared with results of FEM analysis
2) 3PB tests with Scots Pine and Japanese Larch
   From the same methods, we would got the fracture toughness from JSME equation
3) The comparison with the result from 3PB and CT tests

2.1 MATERIALS

The specimens are cut from the glulam which are composed with laminae graded in Japanese Agricultural Standard (JAS) by L125, L100 and L80 of Scots Pine and Japanese Larch and from the glulam with laminae graded in JAS by L80, L60 and L50 of Japanese Cedar.
The cutting condition of part (1) is indicated in Fig.1. We would fabricate the initial crack with cutter blade and a paper cutter shown at the upper of Fig.2, that wood-processing is the same method in our previous article [8]. The dimensionless-crack-length “a/W” for these tests is 0.5.

The specimens of 3PB test are composed with part (1) and (2) as shown Fig.2. The part (2) would be prepared for securing enough length of bending tests. The part (2) is cut from the same graded glulam. The part (1) and (2) are combined together with two-liquid type epoxy-glue

### 2.2 TEST PROCEDURE

The 3PB tests are conducted under monotonic loading and displacement-control (0.5mm/min.). We observed the crack behaviour with the commercially available video cameras on those surfaces. The test conditions are shown Picture 1.

#### 3 TEST RESULTS AND FEM ANALYSIS

#### 3.1 FRACTURE BEHAVIOUR IN 3PB TESTS

An example of relationship with load and displacement is shown in Fig.3, in that diagram the crack-starting points on both surfaces (Picture 2) are shown with circle or multiple mark. We could affirm that those cracks started around the maximum load, and this tendency is the same with CT tests in our past study [4]. The macroscopic fracture behaviour is not in brittle, it is similar to ductile fracture determined in our past study [4]. The propagation speed was not constant and we couldn’t measure its crack length continuously in this study. Then the point at the maximum load is determined as the crack initiation point.

#### 3.2 FEM MODELING

We analyzed with finite element methods with the 3PB specimens, in those models we estimated crack initiating point only, then that propagation behaviour can’t be modelled. The analysis code is ANSYS. A half-part model is used in this study shown in Fig.4 and material properties for fem analysis are denoted in table 1. A comparison with test and analysis results of macroscopic behaviours are indicated in Fig. 5, we could confirm that our modelling is valid. With these results, we could calculate fracture toughness until its maximum loading, J-integral, and compared with other methods (in next chapter).
The calculation results with equation 1), 2) and 3) are compared in Fig.7. As shown that diagram, the behaviours of fem analysis and simple method with equation 2) are similar, so we would confirm equation 2) is available to get fracture toughness from experimental result simplify.

In our past study, we would analyse the same type study with CT tests, then the equation based on JSME is useful for three wood species (Scots Pine, Japanese Larch and Japanese Cedar). So we would get the same tendency with fem analysis and simple method in 3PB tests.

The test results with Scots Pine and Japanese Larch are shown in Fig.8. Those fracture patterns are almost the same with Japanese Cedar specimens.

4 CALCULATION METHODS OF FRACTURE CRITERIA

In our past study with CT tests\cite{8}, we would calculate fracture toughness for crack initiation, $J_{i}$, based on the JSME equation. According to that result, we could get the same values on J-integral from the same type equation. So with three methods, we would fracture toughness and compared with the follows.

The first method is classic theoretical method by Rice, and in fem analysis with equation 1) we could calculate J-integral. Its calculation pass of J-integral calculation is shown Fig.6. For the crack initiation point, we would determine the maximum load in our tests.

The second method is based on the JSME with equation 2), its theoretical method is proposed by Landes and Morten (1981).

And the last one is based on Nordtest methods\cite{6}. The fracture energy would be calculated by equation 3).

$$J_{i} = 2 \* \frac{A}{(B \* b_{0})} \hspace{1cm} \cdots \text{Eq.2}$$

where,

$A = \int_{0}^{\delta} P d\delta$

$A$: external work, $B$: the width, $b_{0}$: the ligament length

$$G_{f} = \frac{W}{(B \* b_{0})} \hspace{1cm} \cdots \text{Eq.3}$$

where,

$W = \int_{0}^{\delta} P d\delta$

$W$: total external work
Table 2 comparison with CT tests and 3PB tests

<table>
<thead>
<tr>
<th>Species</th>
<th>Grading</th>
<th>Part</th>
<th>J-integral[N/m]</th>
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<td></td>
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<td>3PB tests</td>
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6 CONCLUSIONS

In this paper, we conducted 3PB test with three pieces of wood for structural glue-laminated timber and analyzed with fem methods, and calculated J-integral by some methods. And J-integral from 3PB tests would be compared with that from CT tests in our past article. The knowledge in this article is followings;
1) These specimens which its dimensionless-crack-length, $a_0/W$, is 0.5 are failed in ductile (not in brittle), and we could confirm crack initiation on its surface before perfectly rupture. And crack initiation point is almost at its maximum load.
2) The behaviours of fracture toughness from fem modelling with equation 1) and simple calculation with equation 2) are similar, so we get J-integral with that simple equation.
3) We could get J-integral at the same level from 3PB and CT tests. So both methods are useful to evaluate fracture toughness of wood.

Reference
OPTICAL EXTENSOMETRY BY GEOMETRIC MOIRÉ DIGITAL ANALYSIS OF STRESS IN THE PLANE – AN APPLICATION IN TIMBER

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ABSTRACT: The study of the behaviour of building structures pass through the determination of the strain and stresses to evaluate the level of local and global security against the active actions. In the last years in Brazil, the extensometers have been commonly used to measure the strain at a specific single point. The optical extensometry presents itself as an alternative to this task by providing results for all points of measurement area (full field). The moiré techniques are the simplest of the optical extensometry and have the smaller indices relating to noise. In this work, the sensitivity of mechanics (or geometric) interference of moiré technique reached levels that previously could only be obtained using interferometers (moiré interferometry). It is worth noting that interferometers are appliances that cost thousands of dollars. This work was made possible through the multiplication of fringes obtained through digital technology with application present in photography and digital image processing and proved able to measure deformations for micrometric from very low frequency cross gratings. This work is presented as digital geometric moiré technique (TMGD) because everything from image capture of specimen gratings until the generation of fringes was made by digital means and therefore does not fall within the classification set out in chapter which deals with the classification of moiré techniques. The result was something still unheard in civil engineering in Brazil, a low-cost technique, insensitive to vibrations and of the environment, moreover able to make measurements in plane surfaces without major apparatus. Findings show the applicability of the proposed method to measure strain and to analyse stresses in models and civil engineering structures, among others. In this work, the sensitivity of mechanics (or geometric) interference of moiré technique reached levels that previously could only be obtained using interferometers (moiré interferometry). Its application is showed in timber structures.

KEYWORDS: optical metrology, full field extensometry, moiré techniques, non-destructive testing, behavioural analysis of structures, wooden structural connections.

1. INTRODUCTION

The analysis of the behavior of deformable bodies is a fundamental need for science and technology. While the applicability of the content of this work is vast, it has its main focus addressed at the problems of civil structural engineering, where the study of the behavior of building structures, dams, tunnels, highways, railways, ports, airports, bridges and furthermore pass through the determination of the stresses and strains to evaluate the level of local and global security against the active actions. In the last years, the electrical strain gages have been commonly used to measure the strain at a specific single point. The optical extensometry presents itself as an alternative to this task by providing results for all points of measurement area (full field). The moiré techniques are the simplest of the optical extensometry and have the smaller indices relating to noise. In this work, the sensitivity of mechanics (or geometric) interference of moiré technique reached levels that previously could only be obtained using interferometers (moiré interferometry). It is worth noting that interferometers are appliances that cost thousands of dollars.

2. INTERFEROMETRY TECHNIQUES

2.1 HISTORY

A group of techniques based on interferometry has emerged from the 1960s, starting with the holographic interferometry (HI), followed by electronic speckle...
pattern interferometry (ESPI) and then by the moiré interferometry technique (IMT) at the end the 1970s. All these groups of displacement techniques provide information on the surface of an opaque sample, from which the field of specific deformation can be calculated by numerical differentiation.

The ESPI technique has emerged as a preferred option in many cases due to their ability to display the fringes shift in real time.

The technique of HI is refreshed with the use of sensors modern high resolution image.

TM1 has the highest spatial resolution and signal to noise ratio between the three, however, the need to prepare and restrictions on the sample size has limited its field.

Two other interferometry techniques are closely related and deserve examination. The technique of scanning interferometry of white light (SWLI) and optical coherence tomography (OCT) are both based on the principle of low coherence interferometry. In this case, light with a broad spectral content is used such that the fringes are seen only in those regions corresponding to the minimum optical path differences between the wave fronts which interfere.

The surface profiles of opaque materials can be determined with high precision in the case of SWLI, or also the internal structure of three-dimensional image material with a low dispersion in the case of OCT.

The extreme sensitivity of all interferometry techniques achieve a range of displacement that ranges from sub-micron to tens of micrometers. This sometimes ends up being more of a disadvantage than an advantage, particularly when it is necessary to make measurements in noisy environments.

As a result, other non-interferometry techniques such as reticule or the method of the correlation digital image (DIC) have been widely used in recent years.

The main advantages include immunity from nuisance; experimental simplicity, the easy determination of scale, and sensitivity to shifts adapted to the range typically required for structural testing of large and medium scale.

The technique consists of the reticule image analysis reticules, which are transferred, stored or pasted on the surface of the model in order to track the movement of sample points when it is deformed.

The technique of digital image correlation (DIC) is a direct descendant of the technique known as speckle photography, and is also closely related to particle image velocimetry, which has widespread use in experimental fluid mechanics.

The basic concept of DIC considers that a random pattern is applied to the sample, correlating sub-pictures of a standard reference image with an image of the sample deformed and can be derived from a vector offset for each pair of sub-related images.

A final important class of techniques in the arsenal not interferometry photomechanical is called the infrared thermography (IR), which involves the use of an infrared camera to detect small changes in temperature associated with changes in load applied to the model. These variations can be analyzed in the context of Thermo elasticity (thermo elastic stress analysis).

Have recently been processed fields of temperature to recover the heat sources that are directly related to some irreversible phenomena occurring in materials engineering, such as damage or plasticity.

Feng [1] introduced a new optical fiber sensor for monitoring the integrity of structural bridges.

The choice of an optical method for a given application is not a trivial task. However, many criteria may be considered in this choice, such as: the cost; simplicity of implementation; performance (resolution, spatial resolution, etc.); to measure the magnitude (displacement, strain, etc.); the expected range of variation of measurement (large or small deformations); sensitivity to vibration; size of the region of interest (macro, micro or nano).

2.2 THE MOIRÉ FENOMENON

According to several authors, the word moiré is French and refers to the brightness of wet tissue, where alternating bright and dark bands.

Moiré patterns appear when two uniform lines or dots of the same repeat spacing, or almost identical, are superimposed. It is common to see this effect when two parallel garden fences are seen by one another, or if two parts of windows with the same repeating patterns.

Figure 1 shows fringes obtained from reticule formed by a family of curves of constant period.

For TM Shadow and Projection, the measurement sensitivity varies with the illumination and observation angles to the normal to the plane of the moiré reticule.

By increasing these angles, or only one, also increases the sensitivity.

2.3 ANALYSIS FOR SMALL DEFORMATIONS AND ROTATIONS FROM FIELD DISPLACEMENTS IN DIGITAL IMAGES

According to Lino [2], the sensitivity of the TM may be increased by various methods, such as interpolation of fringes and fringe multiplication techniques reticule mismatched (mismatch techniques). We included the two most important methods, the first being used in the experiments to be presented in this paper.

According to Spots [3], case of the displacement field are provided for digital images, and equations (1, 2, 3) are used to calculate deformations in the plane:
The geometric moire technique (TMG) needs to be applied in the reticule model amplitude with frequency of 5 to 100 lines/mm. This application reticles can be made by a wide variety of processes. The more interferometric technique (MIT) needs reticles of phase with frequency much higher than the TMG.

According to Cloud [4], reticles designed to make analysis of deformation are not easy to obtain, manufacture, or handle. They must have high contrast, have well-defined borders, should be light and somewhat rigid enough not to affect the mechanical behavior of the model on which they apply. It is a good technique to use reticles of circular points, separated by free spaces. This prevents interference with the movement of each point with its neighbors and cause less disturbance in deformations field. Most commercial reticle has a line width of approximately 50% of the period to produce maximum contrast of the fringes.

Once in possession of a grid master, he has to be applied or reproduced on the sample surface (Kafri and Glatt [5]).

The photograph can be used to duplicate reticulosis in special cases to vary the line width of the reticule, by 40 lines/mm of parallel lines and crossing lines on clear float glass, 25 cm in size and metal matrix of circular points with a period of 20 lines/mm.

### Table 1: Reticules patterns used in the model associated with different materials. Source: (Kafri and Glatt [5]).

<table>
<thead>
<tr>
<th>Metals</th>
<th>Porous Materials</th>
<th>Plastic, Composites</th>
<th>Others</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-ferrous metals</td>
<td>Concrete</td>
<td>PCV</td>
<td>Textiles</td>
</tr>
<tr>
<td>Steel (1-4)</td>
<td>(1-3 and 6)</td>
<td>(4)</td>
<td>(5)</td>
</tr>
<tr>
<td>Brick (1, 2 and 6)</td>
<td>Timber (1, 2)</td>
<td>Rubber (4, 6)</td>
<td></td>
</tr>
<tr>
<td>Clay (4)</td>
<td></td>
<td>Skin (4)</td>
<td>Card (5)</td>
</tr>
<tr>
<td>Concrete</td>
<td>Brick (1, 2 and 6)</td>
<td>Rubber (4, 6)</td>
<td></td>
</tr>
<tr>
<td>Clay (4)</td>
<td></td>
<td>Skin (4)</td>
<td>Card (5)</td>
</tr>
</tbody>
</table>

({1}Pattern printed on paper; {2}Stripping film; {3}Direct photo printing; {4}Stencil Patterns; {5}Not covered in this text; {6}Other: Textile, random patterns with tuned optical imaging (speckle photography).

#### 2.4.1 MULTIFRAN routine for the multiplication of fringes by 8, 16, 32, 64 and 128 times

The application of TMGD for multiplication of fringes by 8, 16, 32, 64 and 128 times, is made by the routine IMAGE J macro program created by the authors, named MULTIFRAN.

Para alimentação dos dados de entrada, são necessárias as duas imagens Iop.bmp e Iop_scaled.bmp em 8 bits e as seguintes informações:

To supply the input data, you need both Iop.bmp Iop_scaled.bmp 8-bit images and the following information:

- identification of the specimen;
- loading or deformation data;
- step reticule in pixels;
- the field direction of displacement (x, y);
- relative rotation angle θ between the two input images.

The output of routine Multifran provides the fringes multiplied by the factor β, rotated in θ, in the chosen direction, and also the four phase shifted images corresponding to the penultimate step.

It is especially important to use these four images of the penultimate step to doing data entry in the program MAS (Moiré Analysis Software), to use the filtering capabilities advanced to the final analysis.

#### 2.4.2 Filtering Advanced Program Moiré Analysis Software (MAS)

The MAS program has several tools including two powerful advanced filters: Self Adaptive Filter and Phase-Shift Filtering (Figure 2). Self-Adaptive Filter works by interacting as a function of the density and direction of the fringes, and Phase-Shift Filtering interacts with the four phase shifted images. The effect of these advanced filters on the waves of intensities and proves to be an excellent resource (Figure 3).
2.5 **VIRTUAL TESTING WITH DIGITAL GEOMETRIC MOIRÉ TECHNIQUE (TMGD)**

2.5.1 **Objetives**

Apply, by software, a strain known in the image reticule, and compare its value with that obtained by applying the digital geometric moiré technique (TMGD) for the purpose of verifying the effectiveness of this technique.

2.5.2 **Equipment and arrangements utilized**

- Notebook connected to the camera with the Digital Photo Professional software installed.
- Camera CANON OS Rebel 2Ti + MACRO EF-S 60 mm 1:2, 8 USM lens.
- Bubble level 2 axes in the berm of camera’s flash.
- Rail with milimetric focus.
- Manfrotto Tripod, with horizontal axis.
- Illumination:
  - Arrangement 1: two compact fluorescent lamps of 15w;
  - Arrangement 2: a halogen circular lamp 18 cm in diameter.

In one arrangement (Figure 4), objects with the graved reticules were positioned on a horizontal surface and were illuminated with two fluorescent lamps.

In the second arrangement (Figure 5), objects with the graved reticules already written to model of ductile steel were placed in an upright position and were illuminated with circular halogen lamp. It was provided with a black background around the model to avoid maldistribution of indirect illumination due to reflections of light. Recall that the mylar plastic surface is specular, and the model surface is diffuse, and that these two types of surface has very different behaviour.

It is worth noting that the setup configured for picture resolution of 8000 dpi was to reconcile it with the resolution of photoplotter who printed the reticulum.

2.6 **TESTS WITH WOODEN MODEL**

2.6.1 **Test made with a tensile wood model**

The following test was made with a tensile wood model, shown in Figure 6 (Afonso, Del Fabbro and Demarzo [6]). Was utilized wood specie ipe (*Tecoma ssp*) for the specimen. The static loads were applied within the limit of proportionality of the material.

This model was coated with epoxy white level of approximately 0.1 mm thick. Were drawn on the white epoxy two orthogonal axes, pad, passing through the center of the model. The mylar plastic containing the
grid, at a rate of 7.87 lines/mm was pasted on the transparent araldite epoxy coating, having the axes already designed by reference.

The latter technique showed a regular formation of fringes, which allowed the extraction of numerical results. The markings on the template and the center axes crossed drawn above were used to impose the coincidence of the two images and thus, eliminate / minimize the displacements and rotations of the plane of the rigid body model. It was finally solved the problem of rotations and rigid body displacement in the plane of the model, which had been hampering the interpretation of the formation of fringes.

As this experiment, strain gages are not pasted, TMGD results were compared with theoretical values calculated for the elasticity modulus of the ipe extracted tables. The numerical results also coincided occasionally, but not kept pace with the loading.

Since in this experiment the strain-gauges were not glued, TMGD results were compared with theoretical values calculated for the elasticity modulus of the ipe extracted tables. The numerical results also coincided punctually, but not kept pace with the evolution of loading.

### 2.6.2 Experimental application

The follows experimental applications were made: uniaxial tensile tests on steel and wood (Figures 7 and 8).

**Figure 7:** Preparation of specimen timber for exposure to UV light; IFGW Optics Laboratory-UNICAMP.

**Figure 8:** LMS-Laboratory of Soil Mechanics, Polytechnic School of University of São Paulo.

### 2.6.3 Results

The results of test in tensile wood model are showed in Table 2.

### 3. CONCLUSIONS

This research has produced, in an innovative way, the moiré technique geometric digital (TMGD), which allows increasing the factor $\beta$ from 10 to 128, to the 8-bit system, using the digital multiplication of fringes DFM.

### Table 2 Summary of results of displacement $\Delta_x$ ($\mu$m) and specific deformation $\varepsilon$ ($\mu$) to wood specimens.

<table>
<thead>
<tr>
<th>Test</th>
<th>Image</th>
<th>Arrangement</th>
<th>Surface</th>
<th>Factor Multiplication ($\beta$)</th>
<th>Analytical Calculation</th>
<th>TMGD</th>
<th>Deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>128</td>
<td>1</td>
<td>especular</td>
<td>1</td>
<td>513 10258</td>
<td>511 10200</td>
<td>0.39 0.60</td>
</tr>
<tr>
<td>02</td>
<td>128</td>
<td>1</td>
<td>especular</td>
<td>2</td>
<td>513 10258</td>
<td>515 10225</td>
<td>0.39 0.32</td>
</tr>
<tr>
<td>03</td>
<td>140</td>
<td>1</td>
<td>diffuse</td>
<td>4</td>
<td>127 1511.78</td>
<td>135 1610</td>
<td>6.29 6.50</td>
</tr>
<tr>
<td>04</td>
<td>150</td>
<td>2</td>
<td>diffuse</td>
<td>4</td>
<td>319 6569</td>
<td>313 6435</td>
<td>1.88 2.0</td>
</tr>
</tbody>
</table>
The application of TMGD with multiplication of fringes by DFM, and the technique developed here to apply the reticule in the model, extended the original proposal, made only for wood, for almost all structural materials of construction.

This task was made possible through the multiplication of fringes obtained through digital technology with application present in photography and digital image processing and proved able to measure deformations for micrometric from very low frequency cross gratings. This work is presented as digital geometric moiré technique (TMGD) because everything from image capture of specimen gratings until the generation of fringes was made by digital means and therefore does not fall within the classification set out in chapter which deals with the classification of moiré techniques.

The hybrid TMGD, calibrated with strain-gages, is a proposition quite reliable, because it allows to adjust optical and physical parameters and align the confiability of strain-gages for measurement in the entire field by TMGD.

The development of this work showed advances in photonics and computing in this area:

1) Improvement of the quantitative results by the inclusion of instruments for measuring brightness and laboratory instruments to control the stability of the model and camera.

2) Improvement of filtering and computational algorithms associated with existing.

3) Exploration of the replication technique of high resolution reticule teachers with higher density, using microphotolithography with dry film thickness not less than 15µ.

4) Exploring new models of cameras and new software features and hardware.

5) The application of TMGD for large objects such as bridges and buildings.

6) The application of TMGD with metal reticule at high temperatures.

Quantitative validation of results was made by comparing the results obtained in these tests and those of strain gages bonded to specimen.

A particularity of this method is the possibility of extending its application to large elements such as buildings, bridges.

The result was something still unheard in civil engineering in Brazil, a low-cost technique, insensitive to vibrations and of the environment, moreover able to make measurements in plane surfaces without major apparatus. Findings show the applicability of the proposed method to measure strain and to analyze stresses in models and civil engineering structures, among others.

REFERENCES


EMBEDDING BEHAVIOUR OF DOUGLAS FIR

Steffen Franke¹, Pierre Quenneville²

ABSTRACT: Connections with mechanical fasteners are important for all structures. For the prediction of ductile failure, the design concept in the current New Zealand timber standard for bolted connections is not based on the well accepted European Yield Model (EYM) and depends mainly on the diameter and the timber thickness. For using the EYM, the embedding strength of the timber to be used is one of the important parameters to be known. These strength values are not available for Douglas Fir, even it is being used in constructions. To achieve the missing information and to implement the EYM into the NZ standard, embedding tests with different dowel diameters in Douglas Fir were conducted. The results are compared with former results of New Zealand Radiata Pine and LVL as well as the corresponding results using the Eurocode 5 formulas. It shows that adjusted formulas of the Eurocode 5 can be used.

KEYWORDS: Timber, material properties, Douglas Fir, embedding strength, 5 %-offset method, EYM

1 INTRODUCTION

For all connections it is important to predict the failure strength as accurately as possible. This includes both the ductile and in some cases especially in timber construction, the brittle failure as well. For the calculation of the ductile failure strength, the European Yield Model (EYM) is used in many standards and accepted as a very accurate model. It forms the basis of the European timber standard EN 1995-1-1:2004 (Eurocode 5), [1]. The development of this standard is based on a multitude of embedding and joint tests with different European and North American wood species by many researchers. Furthermore, a continuous adaptation and improvement is reported overseas, [2]. In the current New Zealand timber standard NZS 3603:1993 [3], the design concept for bolted connections is not based on the EYM and depends only on the diameter of the fastener, the timber thickness and the species of wood. It does not predict the different types of failure and overestimates the joint strength partially.

The most important parameters for the EYM are the fastener yield moment and the timber embedding strength, which are known for most of the softwoods and tropical hardwoods. Although, values for New Zealand Radiata Pine and LVL are available, [4], [5] and [6], embedding strength values for Douglas Fir are not available. But to implement the EYM design concept for mechanical connections for Douglas Fir in the current New Zealand design standard, it is essential to investigate the material behaviour and to determine the embedding values.

To achieve the missing information, embedding tests parallel, perpendicular and under various load-to-grain angles with different dowel diameters with Douglas Fir were conducted and compiled to build a database of embedding strength values. The paper also compares the results with the values already available for New Zealand Radiata Pine and LVL and with the predicted embedding failure results calculated using the formulas of Eurocode 5. Moreover, the paper presents a comparison of the different available test standards used to determine the embedding strength.

2 TEST SERIES AND METHODS

2.1 TEST STANDARDS

The embedding strength was evaluated using the 5 %-offset method according to the ASTM D 5764-97a [7], the extended proportional limit load following the DIN 52192:1979 [8] and the maximum load, which is either the ultimate load or the load at 5 mm displacement, according to EN 383:1993 [9] and ISO/DIS 10984-2 [10] respectively. A summary and comparison of the specific procedures in the different test standards are given below.
2.1.1 ASTM D 5764-97a
The ASTM standard “Standard Test Method for Evaluating Dowel-Bearing Strength of Wood and Wood-Base Products” provides a full-hole and a half-hole testing setup, as shown in Figure 1 and Figure 2. The minimum specimen dimensions are 38 mm or 2d in thickness and a maximum of 50 mm or 4d for the width and length, independent of the load-to-grain angle, where d is the dowel diameter.

The test is conducted as to reach the maximum load in 1 to 10 min, using a constant rate of testing of 1.0 mm/min. There is no further information about the loading procedure. The results are given as the yield load, determined using the 5 %-offset method, the proportional limit load and the ultimate load. The embedding strength, calculated from the yield load, is given by:

\[ f_h = \frac{F_{\text{yield}}}{d \cdot t} \]  

(1)

2.1.2 ISO/DIS 10984-2
The tests according to the international standard “Timber structures - Dowel-type fasteners - Part 2: Determination of embedding strength and foundation values” shall be carried out using a full-hole test shown in Figure 3a, but it is a requirement of the test to avoid bending of the fastener under test. Thus it also allows using a half-hole test shown in Figure 3b. The minimum specimen dimensions for tests parallel and perpendicular to grain can be found in Figure 4.

The loading procedure to be used consists of one preload cycle between 0.1·\(F_{\text{max, est}}\) and 0.4·\(F_{\text{max, est}}\) and the force is to be increased or decreased at a constant rate. The maximum load is to be reached within 300 ± 120 s. The standard includes formulas to calculate the embedding strength, where \(F_{\text{max}}\) is either the ultimate load or the load at 5 mm displacement, and the foundation modulus as below, where \(w\) is the displacement:

\[ f_h = \frac{F_{\text{max}}}{d \cdot t} \]  

(2)

2.1.3 EN 383:1993 or DIN EN 383:2007
The European testing standard “Timber structures – Test methods, Determination of embedding strength and foundation values for dowel type fasteners” is equal to the ISO/DIS 10984-2 [10], except that it does not provide the half-hole test alternative.

2.2 TEST SERIES AND SPECIMEN
The embedding tests series conducted include a total of 185 tests with Douglas Fir and a dowel diameter \(d\) of 6, 8, 12, 16, 20, 25 and 30 mm. They also comprise load-to-grain angles \(\alpha\) of 0°, 45° and 90°, as shown in Figure 7. The labelling of each test is based on the following definition:

DF-E0-1x12-01
- Specimen number
- Bolt diameter: Ø 6, 8, 12, 16, 20, 25, 30 mm
- Number of bolts: 1, 2
- Load to grain angle: \(\alpha\) = 0°, 45°, 90°
- Kind of test: Embedement
- Wood product: Douglas Fir

![Figure 1: Test configuration full-hole test, ASTM D5764-97a [7]](image1)

![Figure 2: Test configuration half-hole test, ASTM D5764-97a [7]](image2)

![Figure 3: Test configuration, ISO/DIS 10984-2 [10]](image3)

![Figure 4: Sizes of test specimen, ISO/DIS 10984-2 [10]](image4)
The tests were conducted according to the ASTM D5764-97a [7] as a half-hole test (Figure 2 and Figure 5) which involves pushing a bolt so that no bending effects are observed. Due to the different requirements for the size in the different standards, a general height of 70 mm, except for the tests with dowels bigger than 20 mm in perpendicular and parallel direction, was used. The width of the specimens depends on the dowel diameter and the load-to-grain angle and was 10d for the parallel direction. Perpendicular to the grain, a constant width of 120 mm was used. All specimens were cut from boards with a constant thickness of 40 mm. The density covers a range between 467 kg/m³ and 565 kg/m³ with mean of 513 kg/m³ and a coefficient of variation less than 6 % as shown in Figure 8. The specifications, number, sizes and densities of all groups are summarized in Table 2. The specimens were conditioned to 20 °C and 65 % relative humidity until mass consistency was reached. The moisture content was measured to 10.5 % in average.

3 RESULTS AND DISCUSSION

3.1 EVALUATION METHODS OF THE EMBEDDING STRENGTH

Typical load-displacement curves for the series with one 16 mm dowel are shown in Figure 9. The curves represent the average curve of each group, depending on the load-to-grain angle. For all dowel diameters, the curves show a linear increase of the load up to the proportional limit. After the yielding point, curves are almost constant for $\alpha = 0^\circ$ or increasing slightly for $\alpha = 45^\circ$ but increase significantly for $\alpha = 90^\circ$.

<table>
<thead>
<tr>
<th>Group name</th>
<th>Diameter $d$ [mm]</th>
<th>No. of specimens</th>
<th>Width/Height/Thickness $w/h/t$ [mm]</th>
<th>Density $\rho$ [kg/m³]</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF-E0-1x6</td>
<td>6</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>525</td>
<td>4.9%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x6</td>
<td>8</td>
<td>10</td>
<td>90 / 70 / 40</td>
<td>473</td>
<td>6.4%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x6</td>
<td>10</td>
<td>10</td>
<td>70 / 70 / 40</td>
<td>565</td>
<td>2.3%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x8</td>
<td>6</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>467</td>
<td>4.3%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x8</td>
<td>8</td>
<td>8</td>
<td>100 / 70 / 40</td>
<td>482</td>
<td>3.7%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x8</td>
<td>10</td>
<td>10</td>
<td>80 / 70 / 40</td>
<td>471</td>
<td>12%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x12</td>
<td>12</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>548</td>
<td>7.5%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x12</td>
<td>7</td>
<td>7</td>
<td>120 / 70 / 40</td>
<td>545</td>
<td>9.4%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x12</td>
<td>10</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>512</td>
<td>5.9%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x16</td>
<td>16</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>480</td>
<td>6.2%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x16</td>
<td>16</td>
<td>5</td>
<td>140 / 70 / 40</td>
<td>532</td>
<td>6.6%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x16</td>
<td>10</td>
<td>10</td>
<td>160 / 70 / 40</td>
<td>503</td>
<td>14%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x20</td>
<td>20</td>
<td>10</td>
<td>120 / 70 / 40</td>
<td>515</td>
<td>2.9%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x20</td>
<td>20</td>
<td>5</td>
<td>160 / 70 / 40</td>
<td>528</td>
<td>2.5%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x20</td>
<td>10</td>
<td>10</td>
<td>200 / 70 / 40</td>
<td>500</td>
<td>4.8%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x25</td>
<td>25</td>
<td>10</td>
<td>120 / 80 / 40</td>
<td>511</td>
<td>12%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x25</td>
<td>25</td>
<td>5</td>
<td>140 / 70 / 40</td>
<td>530</td>
<td>6.2%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x25</td>
<td>10</td>
<td>10</td>
<td>250 / 80 / 40</td>
<td>502</td>
<td>3.6%</td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x30</td>
<td>30</td>
<td>10</td>
<td>120 / 90 / 40</td>
<td>519</td>
<td>2.4%</td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x30</td>
<td>30</td>
<td>5</td>
<td>140 / 70 / 40</td>
<td>565</td>
<td>1.0%</td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x30</td>
<td>10</td>
<td>10</td>
<td>300 / 90 / 40</td>
<td>500</td>
<td>5.9%</td>
<td></td>
</tr>
<tr>
<td>Total or average:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>185</td>
<td>513</td>
</tr>
</tbody>
</table>
For each test, the following characteristics, as shown in Figure 10, were evaluated: the proportional limit load $F_{\text{prop}}$, the yield load $F_{5\%}$, and the maximum load $F_{\text{max}}$, either as the ultimate load (mostly for $\alpha = 0^\circ$) or the load at 5 mm displacement (mostly for $\alpha \geq 45^\circ$). The proportional limit load is defined as the contact point of the test data and a line with a slope of 2/3 of the initial stiffness $K$ according to DIN 52192:1979 [8]. The 5 %-offset method, according to EN 383:1993 [9] and ISO 10984-2 [10] respectively, was adopted to evaluate the yield load. Based on the relation given at equation (2), the embedding strength is calculated to the 5 % yield embedding strength $f_{h,5\%}\alpha$ and the maximum embedding strength $f_{h,\alpha,\text{max}}$ in this paper respectively, where $\alpha$ is the load-to-grain angle.

3.2 EMBEDDING STRENGTH OF DOUGLAS FIR

Table 2 shows the mean values and the coefficient of variation of the yield and maximum embedding strength for each test series depending on the dowel diameter, the number of dowels and the load-to-grain angle. The last row includes the ratio between the yield and maximum embedding strengths.

The mean values of the yield embedding strength $f_{h,5\%}\alpha$ are compared as a function of the dowel diameter in Figure 11. They show a small reduction in the strength values with an increase of the dowel diameter. The

![Figure 9: Typical load vs. displacement curves for Douglas Fir](image)

![Figure 10: Evaluating methods for embedding strength](image)

![Figure 11: Mean yield embedding strength vs. dowel diameter for Douglas Fir](image)

![Figure 12: Embedding strength vs. density for $d = 16$ mm for all loading angles in Douglas Fir](image)

<table>
<thead>
<tr>
<th>Group name</th>
<th>Yield strength $f_{h,5%}\alpha$</th>
<th>Max. strength $f_{h,\alpha,\text{max}}$</th>
<th>$f_{h,\alpha,\text{max}}/f_{h,5%}\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF-E0-1x6</td>
<td>43.2 12.3 % 45.7 9.4 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DF-E45-1x6</td>
<td>23.6 16.6 % 27.6 35.9 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DF-E90-1x6</td>
<td>25.8 10.1 % 35.4 12.6 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DF-E0-1x8</td>
<td>35.6 14.2 % 37.1 12.7 %</td>
<td></td>
<td>1.04</td>
</tr>
<tr>
<td>DF-E45-1x8</td>
<td>21.7 15.3 % 26.0 13.0 %</td>
<td></td>
<td>1.19</td>
</tr>
<tr>
<td>DF-E90-1x8</td>
<td>21.0 14.7 % 26.9 20.5 %</td>
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<td>1.28</td>
</tr>
<tr>
<td>DF-E0-1x12</td>
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<tr>
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<td>27.0 12.0 % 28.7 12.1 %</td>
<td></td>
<td>1.06</td>
</tr>
<tr>
<td>DF-E90-1x12</td>
<td>20.0 12.6 % 23.1 10.6 %</td>
<td></td>
<td>1.16</td>
</tr>
<tr>
<td>DF-E0-1x16</td>
<td>38.2 6.5 % 38.4 5.9 %</td>
<td></td>
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</tr>
<tr>
<td>DF-E45-1x16</td>
<td>23.6 23.8 % 26.1 20.8 %</td>
<td></td>
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</tr>
<tr>
<td>DF-E90-1x16</td>
<td>16.8 13.8 % 19.5 22.2 %</td>
<td></td>
<td>1.16</td>
</tr>
<tr>
<td>DF-E0-1x20</td>
<td>35.1 13.5 % 35.1 13.5 %</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>DF-E45-1x20</td>
<td>19.4 10.1 % 21.8 12.0 %</td>
<td></td>
<td>1.12</td>
</tr>
<tr>
<td>DF-E90-1x20</td>
<td>17.2 16.9 % 16.9 10.8 %</td>
<td></td>
<td>0.98</td>
</tr>
<tr>
<td>DF-E0-1x25</td>
<td>40.6 20.3 % 40.6 20.3 %</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>DF-E45-1x25</td>
<td>20.5 9.8 % 21.6 11.0 %</td>
<td></td>
<td>1.05</td>
</tr>
<tr>
<td>DF-E90-1x25</td>
<td>15.4 19.1 % 17.4 13.0 %</td>
<td></td>
<td>1.13</td>
</tr>
<tr>
<td>DF-E0-1x30</td>
<td>32.8 6.8 % 32.8 6.8 %</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>DF-E45-1x30</td>
<td>20.1 8.0 % 22.0 9.7 %</td>
<td></td>
<td>1.10</td>
</tr>
<tr>
<td>DF-E90-1x30</td>
<td>15.5 12.3 % 16.2 10.7 %</td>
<td></td>
<td>1.05</td>
</tr>
</tbody>
</table>
dependency of the embedding strength on the load-to-grain angles was more significant for the yield embedding strength. For dowel diameters \( d = 6 \) and \( 8 \) mm, we observed a sensitive influence on the strength values for \( \alpha = 0^\circ \) and \( 45^\circ \). This was also observed within former investigations on the embedding strength of nails and other wood species. All embedding strength values vs. density for a dowel diameter of \( 16 \) mm are exemplary shown in Figure 12. The strength values have a visible positive correlation with the density for \( \alpha = 0^\circ \) and \( 90^\circ \). It also shows an even more positive correlation for \( \alpha = 45^\circ \), but for other diameters the trend is different and even in opposite direction (decreasing of the embedding strength).

4 EMBEDDING STRENGTH VS. EUROCODE 5

The comparison of the mean test values with the corresponding embedding strengths according to the Eurocode 5 formulas are shown in Figure 13. The left graph shows the embedding results for \( \alpha = 0^\circ \) and \( 90^\circ \) in comparison with the embedding strength by EC 5, whereas the right graph shows the embedding strength \( f_{h,0} \) together with the reduction factor \( k_{90} \) as well as their linear regression. The embedding strength according to the Eurocode 5 was calculated using:

\[
f_{h,0} = 0.082(1 - 0.01d)\rho \quad (3)
\]

\[
f_{h,a} = \frac{f_{h,0}}{k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha} \quad (4)
\]

where

\[
k_{90} = \begin{cases} 
1.35 + 0.015d & \text{for softwood} \\
1.30 + 0.015d & \text{for LVL} \\
0.90 + 0.015d & \text{for hardwood}
\end{cases} \quad (5)
\]

and \( d \) the dowel diameter, \( \rho \) the mean density of all tests of \( 513 \) kg/m\(^3\) and \( \alpha \) the load-to-grain angle.

The results in the left diagram of Figure 13 show a good agreement for the embedding strength \( f_{h,0} \) for smaller dowels, where for bigger dowels higher values could be observed. The results according to the EC 5 for the embedding strength \( f_{h,0} \) are generally about xx % higher than the experimental results. The direct comparison of the reduction factor \( k_{90} \) in the right diagram shows higher experimental values with a slightly higher trend. Both trends, for the embedding strength and the reduction factor, would be even better in trend with the EC 5 when reducing the embedding strength value for the \( 25 \) mm dowel. But this influence and reason for the higher value should be validated with more tests.

5 DESIGN PROPOSAL AND COMPARISON

Based on the current results and the relation given in the Eurocode 5, the following formula for the prediction of the embedding strength of Douglas Fir could be derived. A summary is given with the Equations (6, 7 and 8).

\[
f_{h,0,k} = 0.079(1 - 0.0046d)\rho_k \quad (6)
\]

\[
f_{h,a,k} = \frac{f_{h,0,k}}{k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha} \quad (7)
\]

where

\[
k_{90} = 1.56 - 0.0029d \quad (8)
\]

and

\[
f_{h,0} = 40.654 - 0.1862d \quad (9)
\]

\[
f_{h,a} = 0.079\rho_k \quad (10)
\]

\[
k_{90} = 1.559 - 0.0029d \quad (11)
\]

\[
f_{h,0,k} = 40.654 - 0.1862d \quad (12)
\]

\[
f_{h,a,k} = 0.079\rho_k \quad (13)
\]

\[
k_{90} = 1.559 - 0.0029d \quad (14)
\]

\[
f_{h,0} = 40.654 - 0.1862d \quad (15)
\]

\[
f_{h,a} = 0.079\rho_k \quad (16)
\]

\[
k_{90} = 1.559 - 0.0029d \quad (17)
\]
\( \rho_k \) is the characteristic timber density in kg/m³
\( d \) is the bolt or dowel diameter in mm
\( \alpha \) is the load-to-grain angle.

The comparison in Figure 13 with embedding strength results according to the formulas for Radiata Pine and LVL determined by Franke & Quenneville [6] shows similarities in the behaviour of these three wood species respectively wood product. This comparison is based on the use of a constant density of 500 kg/m³ for each species. In a next step, it could be tried to combine these formulas to one with respecting the differences by e.g. a factor for each species or product for easier handling.

6 CONCLUSIONS AND OUTLOOK

Based on the current results from Douglas Fir, the method of the Eurocode 5 with adjusted formulas for estimating the embedding strength \( f_{h0} \) can be used, as shown in Equation (6). Also for the reduction factor \( k_{90} \), an adjusted formula can be used, cp. Figure 13.

With the presented results and formulas developed, it is, in addition to the results which are already available for Radiata Pine and LVL, [6], easily possible to implement the European Yield model into a revised New Zealand design standard for the prediction of the ductile failure mode of bolted and doweled connections.

For a comprehensive use of the EYM model, there is also a need to evaluate the embedding strength values for nails and screws. Furthermore, for an easier handling, it could be tried to combine the formulas for the different species to one formula with respecting the differences by e.g. a factor for each species or product. Another focus for further research is the other modes of failure, the brittle failures, for which many research activity is documented around the world.

ACKNOWLEDGEMENT

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REFERENCES

INVESTIGATION OF THE SPLITTING BEHAVIOUR OF RADIATE PINE AND LVL UNDER TENSION AND SHEAR

Bettina Franke¹, Pierre Quenneville²

ABSTRACT: The fracture mechanics is accepted and used for the description of the brittle failure behaviour of wood and wood products. The paper presents the investigation of the failure mechanism of the fracture modes I and II and the mixed mode of New Zealand Radiata Pine timber (RP) and of Radiata Pine Laminated Veneer Lumber (LVL). Different test setups were compared and used for the determination of the material parameters fracture toughness and fracture energy. The research results show that Radiata Pine LVL is more ductile than Radiata Pine timber. It also leads to higher values of the fracture energy for mode I for Radiata Pine LVL than for timber.

KEYWORDS: Fracture mechanics, Mode I, Mode II, Solid wood, LVL

1 INTRODUCTION

Typical timber constructions consist of members with joints, notches or holes which can lead to high stress singularities in the structure. Fracture mechanics gives the possibility to characterize situations with stress singularities including crack initiation and crack propagations which can lead to failure. For the prediction of failures or load capacities, the fracture mechanics methods are used in new design proposals. However, important material parameters such as the fracture energy or the fracture toughness have to be known. International research results are published for solid wood e.g. in Radiata Pine [1], in European Spruce [2], [3], [4], [5] or in Canadian Spruce [6], [7]. For engineered wood products, only a portion of the values are published, e.g. [8], [9], [10]. The trend to large scale and multi-storey timber constructions requires high performing wood products with increased strengths and stiffnesses. In order to allow the analysis of failures in structures made of Radiata Pine timber or LVL, a study to determine the fracture energy and fracture toughness was initiated. The research results presented include the investigation of the failure behaviour of New Zealand Radiata Pine LVL compared to timber.

The fracture mechanic method distinguishes three different failure modes, as shown in Figure 1. They are: Failure mode I – cracking under transverse tension stress in relation to the crack plane, mode II – cracking under shear stress in plane and mode III – the failure under shear stress out of plane. The so-called mixed mode failure is a combination of the mode I and mode II. The fracture modes I and II and for some cases, the mixed mode failure were investigated in experimental test series.

None of the test setups available for the different failure modes are approved or standardized for wood. Therefore different methods were used and compared for the same stress situation.

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Figure 1: Different test setups for the fracture modes I, II and III and the mixed mode
2 MATERIAL AND METHODS

2.1 MATERIAL AND SPECIMEN

New Zealand Radiata Pine timber and LVL were used for the analysis of the failure behaviour of solid wood and wood product respectively. New Zealand Radiata Pine is a relatively fast growing plantation pine species which is fully grown after approximately 20 to 30 years. The timber was conditioned in an environment with a temperature of 20°C Celsius and a relative humidity of 65 %. At testing, the average moisture content from the test specimens was 11.4 % and the mean density was 486 kg/m³.

LVL is a wood product manufactured from rotary peeled veneer layers glued together to form panels. The veneers are 3 - 4 mm thick and in general laid up with parallel to grain direction. The hyspan product from the Carter Holt Harvey Company from the North Island of New Zealand was used. LVL specimens were also stored in the conditioned environment chamber. The average moisture content of the LVL test specimens was 8.9 % and the mean density was 600 kg/m³.

The test series are classified relative to the loading situation in the fracture mode I and mode II and in the crack systems RL, TL and NL. The letters indicate the three natural material axes, radial, tangential and longitudinal. The letter N defines an angle to the grain of about 45° between the tangential and radial orientation. The first letters specify the direction normal to the crack plane and the second letter the crack growth direction. Cracking in the longitudinal direction were tested in all cases. The crack systems used are shown in Figure 2.

The different crack systems were replicated in the mixed mode test series as well. Table 1 gives an overview of the test groups, the cracking systems as well as the number of each test configuration. The sizes of the specimen are defined by the depth of the member h and the thickness d at the crack plane.

2.2 TEST SETUPS

For the investigation of the failure behaviour of wood under tension perpendicular to grain as well as shear and for the determination of fracture parameters, different test setups are available and accepted as shown in e.g. [3], [4], [11], [12], [13]. The following three test setups were chosen for the investigation of Radiata Pine timber and LVL.

For the fracture mode I, the single end notched beam (SENB) specimen was used according to the draft standard from [5]. The specimen tested was glued to two outer wooden beams, as shown in Figure 3. The grain direction of the specimen is perpendicular to the span of the beam. The beam is simply supported and loaded in a three point bending test. The displacement is controlled and the load measured.

For the pure fracture mode II, a variation of the compact shear specimen (CSS), [14], [15], was used. The specimens were pre-cut with two notches symmetrically to the center, as shown in Figure 4. The specimen was loaded in a displacement controlled regime and the load was applied on top of the specimen over the full thickness. The loading and support situation of the specimen leads to shear of the middle part.

The compact tension shear test specimen (CTS) was introduced by Richard and Benitz [16] as a simple method for the determination of the fracture toughness under pure failure mode I and mode II and also for different mixed mode failures. The test setup developed can be used to investigate superimposed normal and shear stress situations in planar specimens. While rotating the specimen with regards to the load direction, different mode I to mode II ratios can be investigated, as shown in Figure 5. A load angle \( \alpha \) of 90° leads to pure mode I failure where \( \alpha \) of 0° leads to pure mode II.

<table>
<thead>
<tr>
<th>Table 1: Test program characteristics</th>
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</thead>
<tbody>
<tr>
<td>Series</td>
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<tr>
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</tr>
<tr>
<td>1</td>
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<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
</tbody>
</table>

Figure 2: Specimens crack systems and orientations
To be able to compare results obtained from the different test setups used, all tests were done with the same displacement controlled rate of 0.5 mm per second.

3 THEORY AND CALCULATIONS

For the characterization of the failure process, two main concepts were used in the fracture mechanics. First, the stress intensity criterion where the surrounding stress field at the crack tip, is described by a stress intensity factor (SIF) $K$. The crack grows unstable if the critical value of the material, the fracture toughness $K_c$, is reached. The second one is the energy criterion. With the use of the energy balance of the complete system, the energy needed for the crack process can be determined. When the critical fracture energy $G_c$ of the material is reached, the crack grows in an unstable manner and the system fails.

The fracture energy release rate can be determined from the load displacement curve observed during the complete separation of the test specimen under a displacement controlled loading. The integration of the load displacement curve divided by the cracking area gives the fracture energy release rate $G$, and is given in Eq. (1).

$$G = \frac{1}{ad} \int_0^a F \cdot du$$

Where $a$ is the crack length and $d$ the specimen thickness. For the analysis and the determination of the fracture energy for the mixed mode test series, the load was split into the two components, i.e. the transverse tension and shear forces. These components for the pure fracture mode I and mode II are calculated with these values respectively.

The SIF $K$ is defined by the stress $\sigma$, the crack length $a$ and the form function of the geometry $Y$, as given in Eq. (2).

$$K = \sigma \sqrt{\pi a} Y = \frac{F}{d \sqrt{h}} f\left(\frac{a}{h}\right)$$

The fracture toughness $K_c$ is then the corresponding SIF at failure of the specimen with the maximum load $F_c$ and the corresponding crack length $a_c$.

3.1 OBSERVED BEHAVIOUR AT MACRO SCALE LEVEL

In general, during the tests, all mode II specimens failed in a sudden brittle manner, whereas for the fracture mode I specimens, the crack grew in a stable manner. The visible difference after the failure between Radiata Pine timber and LVL are small, since both materials show even crack surfaces in the RL-crack system and uneven or stepwise crack surfaces in the TL-crack system, as shown in Figure 6 and Figure 7.

For the failure mode I specimens, with a load angle $\alpha$ of 90° degrees, small fiber bridges could be observed in the TL-crack system and more often for LVL than in timber. Furthermore, for the RL-crack system, the crack surface lies between the annual growth rings for timber or within one layer of LVL. By increasing the load angle $\alpha$ or for the fracture mode II, the crack surfaces increase due to uneven cracks for both materials. In addition, for LVL, the crack jumps between the different veneers, which leads to an interlocked crack surface.
3.2 MECHANICAL FAILURE BEHAVIOUR

In general, the load capacities increase with decreasing loading angle \( \alpha \), which confirms the known differences between the strength for tension perpendicular to the grain and shear. Furthermore, the crack propagation during the failure changes from relatively stable to a more unstable crack growth, as observed from the comparison of the displacement \( u \) needed for the complete separation of the crack area. Figure 8 shows for example the load displacement curves for all loading cases and for the TL-crack systems for Radiata Pine timber and LVL. The load capacities are on average in the same range for both materials but there are differences in the stiffness between Radiata Pine timber and LVL.

3.3 FRACTURE PARAMETERS

The mechanical failure behaviour described above is also reflected in the fracture parameter values. The fracture energy and fracture toughness are calculated for the complete test series. The values are summarized in Table 2. The number of tests, used to calculate the results shown in Table 2, is sometimes less the total number of specimen tested as given in Table 1, because in some cases other failure occurred and these test results were not used. The first value is for the fracture mode I and the second for the fracture mode II. For each of the fracture parameters, the coefficient of variation (COV) is also given.

Larsen and Gustafsson [5] obtained a linear relationship between the fracture energy of wood in tension perpendicular to grain and density for all test results observed from a joint testing project. The fracture energies for mode I for Radiata Pine timber show no clear dependency on the density, as shown in Figure 9. The fracture energies of the RL- and TL-crack systems are not statistically different. For the NL-crack system, the density and fracture energies are higher compared to the other ones, but this is also influenced by the change of the crack system itself.

The approach by Larsen and Gustafsson [5] for mode I for the LVL specimen is in agreement with the lower limit of the test results. For the test series in LVL, the density is in a close range and the values show only differences between the two crack systems RL and TL, as shown in Figure 10 and Figure 11. For the RL-crack system, the average fracture energy for mode I and mode II is lower than the one of the TL-crack system.

4 DISCUSSION

The failure behaviour of Radiata Pine LVL and timber can be summarized using the fracture toughness observed within the common failure criteria. The failure criterion describes the relation between the failure modes I and II and the mixed mode failure. Figure 12 shows the values for Radiata Pine LVL and timber. The parameters observed are not enough to classify the failure behaviour of Radiata Pine LVL. The concentration of the results to the pure fracture mode I confirms only the mechanical behaviour observed and the known difference between the tension strength perpendicular to grain and the shear strength.
The failure behaviour of Radiata Pine LVL and timber shows a high crack resistance in both fracture modes I and II in comparison to the one for European wood species. For example, the fracture energy for Radiata Pine timber is approximately double the one for mode I or five times for mode II compared to the average values of the fracture energy of European spruce or pine or Canadian spruce found e.g. [2], [3], [6], [7], [17], [18], [19], [20]. This high crack resistance could be partly transferred to LVL from Radiata Pine timber. The fracture energies observed for LVL in the fracture mode I are higher for both crack systems RL and TL. However, the differences of the fracture energies for the fracture mode II are in opposite direction. Radiata Pine timber clearly shows a higher crack resistance than LVL.

When compared to the results by King et. al [1], the observed fracture toughness for mode I and II of Radiata Pine timber shows similar differences between the crack systems RL and TL but the values obtained are lower. LVL results show a lower crack resistance when compared to other wood products such as particle board (FPY), medium density fiberboard (MDF) or oriented strand board (OSB), [9], [10].
5 CONCLUSIONS

A comprehensive test program to determine the fracture parameters of Radiata Pine LVL and timber was initiated. The characterization of the different test setups shows no differences in the fracture parameters determined. Only the preparation of the test specimens and the handling during the experiment is different.

The failure behaviour of Radiata Pine LVL shows a more ductile behaviour in all fracture modes considered in comparison to timber. This leads to higher values of the fracture energy for mode I for LVL than for timber. On the other hand, the energy which was needed for the creation of the crack surface for mode II is smaller for LVL than for timber. The comparison of the mechanical behaviour also shows influences of the orientation and the production method for LVL on the crack resistance. Radiata Pine timber shows on average a stiffer material behaviour, but therefore leads almost to a quite sudden brittle failure.

In structural details such as connections loaded perpendicular to grain or in complex constructions, the different failure behaviour of Radiata Pine LVL and timber should be analyzed carefully as well as the orientation of the layers for LVL has to be considered for the design.

ACKNOWLEDGEMENT

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REFERENCES

PRE-GRADING OF DOUGLAS-FIR LOGS FOR GLULAM LAMELLA PRODUCTION

Andreas Rais¹, Jan-Willem G. van de Kuilen²

ABSTRACT: The assessment of timber quality at any time in the production chain promises to be a big advantage for the forest and timber industry. This paper presents results for pre-grading analysis, which were performed on the complete processing chain – starting with standing trees through to sawn timber. The quality of wood (timber) was detected by means of the dynamic modulus of elasticity. The measurements were conducted on 108 trees of Douglas-fir growing at different forest experimental stands in Germany. Long logs of about 13 m were bucked into short logs of 4.1 m. The dataset contains 630 boards (50 x 160 mm²) cut from 247 short logs. Dependencies were studied by simple linear regression analysis. The investigation shows high correlation between different stages in production chain of timber. The strongest relationship exists between wet and dry boards ($r^2 = 0.97$, $n = 630$). But also the relationship between long logs and short logs cut of them is very good ($r^2 = 0.94$, $n = 180$). Finally, the effects of pre-grading long logs on the yield of sawn timber are shown. A possible threshold value of 11000 for long logs leads to an increase in board yield of up to 30 percentage points for C 24. The boards were strength graded by ViSCAN-COMPACT in C-classes according EN 338. The increase in yield depends on the raw material quality and the strength class combination.

KEYWORDS: Douglas-fir, non-destructive testing, quality, pre-grading, round wood, sawn timber, dynamic MOE

1 INTRODUCTION

Quality of sawn timber varies to a large extent. This is due to the sawing pattern, the log size or the distance to pith. But also site conditions, silvicultural treatment, kind of species or the genes play an important role. Most of timber is used for structural applications. In terms of quality of structural timber, properties like strength and modulus of elasticity are decisive. For these reasons sawn timber is strength graded in order to guarantee characteristic strength values. In recent years there are tendencies to pre-grade timber at the stage of logs, not even when the logs are cut into boards. The assessment of log quality can help sort the wood resource prior to processing. The information provides a more effective usage of wood or timber. Timber can be strength graded visually or by a machine. Detecting the dynamic modulus of elasticity of timber is the most common way of machine strength grading. The European market is dominated by machine grading techniques based on the relationship between a measured dynamic MOE and strength. The method is characterized by easy handling, high infeed speed and high correlation to real strength. Hence, this method makes it also interesting for grading logs.

The goal of this paper is to show the relations of stages in the wood production chain. Finally, the increase in sawn timber yield can be seen when long logs are pre-graded.

2 MATERIAL AND METHOD

2.1 Material

The investigation deals with 108 trees of Douglas-fir (*pseudotsuga menziesii*). The age of all trees is about 40 years. The trees stem from two experimental plots in Bavaria. The plots are part of the Bavarian network of long-term research plots. One plot is located in the North of Bavaria next to the city of Aschaffenburg in the low mountain range Spessart. The other plot is located about 15 km south of the city Ansbach. The experimental plots were founded in the beginning of the 1970s in order to comprehend the dependencies of growth on different silvicultural treatments. The diameter of breast height (dbh) of the 108 trees lies between 24.5 cm and 52.9 cm, the mean is 30.4 cm and the standard deviation (sd) is 4.7 cm.

108 trees were crosscut to long logs of about 13 m. Then long logs were crosscut to 324 short logs of 4.1 m.
a band saw, short logs were cut into 630 boards of 50 mm x 160 mm. It was tried to get as many as possible boards of the cross section 50 mm x 160 mm. Table 1 gives an overview of logs and boards obtained from logs. Mean value, standard deviation, minimum and maximum value are given.

**Table 1: Details of the sample collected – logs and boards**

<table>
<thead>
<tr>
<th></th>
<th>mean</th>
<th>sd</th>
<th>min</th>
<th>max</th>
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<tr>
<td><strong>long logs</strong></td>
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</tr>
<tr>
<td>n = 108</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>0.56</td>
<td>0.16</td>
<td>0.33</td>
<td>0.97</td>
</tr>
<tr>
<td>density ( \rho ) [kg/m²]</td>
<td>693</td>
<td>57</td>
<td>544</td>
<td>847</td>
</tr>
<tr>
<td>( \text{MOE}_u ) [N/mm²]</td>
<td>11200</td>
<td>1500</td>
<td>7600</td>
<td>15800</td>
</tr>
<tr>
<td><strong>short logs</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>n = 324</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>density ( \rho ) [kg/m²]</td>
<td>660</td>
<td>50</td>
<td>542</td>
<td>798</td>
</tr>
<tr>
<td>( \text{MOE}_u ) [N/mm²]</td>
<td>10800</td>
<td>1500</td>
<td>6500</td>
<td>15000</td>
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<tr>
<td><strong>wet boards</strong></td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>n = 630</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>moisture content u</td>
<td>0.38</td>
<td>0.06</td>
<td>0.24</td>
<td>0.75</td>
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<tr>
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<tr>
<td>( \text{MOE}_u ) [N/mm²]</td>
<td>9500</td>
<td>1900</td>
<td>5500</td>
<td>16300</td>
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<td><strong>dry boards</strong></td>
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<td>n = 630</td>
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<td></td>
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<tr>
<td>moisture content u</td>
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<td>388</td>
<td>643</td>
</tr>
<tr>
<td>( \text{MOE}_u ) [N/mm²]</td>
<td>11150</td>
<td>2100</td>
<td>6600</td>
<td>19200</td>
</tr>
</tbody>
</table>

2.2 Method

Throughout different stages of the production chain (see Figure 1) timber quality was tried to assess by non-destructive method. The dynamic properties at the standing tree were detected by the Hitman ST 300 from fibre-gen. This device is able to quantify the average velocity of a stress wave. For different other stages eigenfrequency, weight, dimensions, and the moisture content were determined: long logs, short logs, wet boards, and dry boards. For measuring the eigenfrequency a hammer was used to excite the resonance vibration of the log or lumber in the longitudinal direction. The fundamental resonance frequency and its overtones were measured.

Therefore, it was possible to calculate the dynamic modulus of elasticity according to equation (1):

\[
\text{MOE}_u = (2f_u)^2 \rho_u
\]

where \( \text{MOE} \) = dynamic modulus of elasticity, \( l \) = length, \( f \) = eigenfrequency, \( \rho \) = density and \( u \) = moisture content.

Finally, long logs were graded by different possible settings of MOE. The effect of pre-grading logs on yield of sawn timber is investigated.

3 RESULTS

3.1 Dependency - standing tree vs. long log

Figure 2 shows the relation between the dynamic modulus of elasticity of long logs and the squared average velocity measured by the Hitman ST 300. The coefficient of determination is 0.29, the number of logs (trees) is 108. For standing trees no density was measured. The length of long logs is about 13 meters.
3.2 Dependency - long log vs. short log
In a next step the long logs of about 13 m were cut cross up to 3 logs of 4.1 m, respectively. Based on the MOE-values of the three short logs a simple arithmetic mean of the dynamic modulus of elasticity was calculated and correlated to the dynamic modulus of elasticity of the long log. Figure 3 illustrates the close relationship between short and long logs.

Figure 3: Dynamic measurements of long logs vs. short logs

3.3 Dependency - short log vs. board

Figure 4: Relation between short log and average wet board

Figure 4 and Figure 5 indicate the dependencies between logs and boards cut out of the logs. The coefficient of determination is 0.7 and 0.55, respectively.

3.4 Dependency – wet boards vs. dry boards

Figure 6 shows the dependency between the MOE of wet boards and dry boards. The moisture content of wet boards is ranged above the fibre saturation point. If the fibre is full of water, the dynamic MOE calculated by natural eigenfrequency remains constant [1]. Each MOE of the dry boards is standardized to a moisture content of 12 %. The conversion factor for the MOE of dry boards is one percent per one percent change of moisture content. It is observed that MOEs of dry boards are higher than MOEs of wet boards; even all points are located above the line through the origin (dashed line, Figure 6). The adjusted r-squared value of the linear regression analysis is 0.97, the dataset contains 630 boards of cross-section (50 x 160) mm².

Figure 5: Relation between short logs and individual boards cut of them

Figure 6: Relation between the MOE of dry and wet boards
Figure 7: Ratio between MOE of dry and wet boards

The ratio between the modulus of elasticity of dry boards (moisture content of 12 %) and of wet boards (moisture content above fibre saturation) was calculated. The distribution of the ratio is illustrated by Figure 7. The mean is 1.16, i.e., in average the dynamic modulus of elasticity for dry boards is higher by 16 percent than the modulus of elasticity for wet boards.

3.5 Pre-grading

Figure 8: Increase of sawn timber yield in case of pre-grading logs

In a last step the effects of pre-grading logs were investigated. All boards were machine graded and classified in strength classes (C-classes) of EN 338. Grading model and settings of the strength grading machine ViSCAN-COMPACT from the company MiCROTEC have been applied. The machine is able to detect eigenfrequency and density (based on weight and dimensions). Different strength class combinations were graded: C 18 / reject, C 24 / C 16 / reject, C 24 / C 18 / reject, C 30 / C 16 / reject, C 30 / C 24 / C 18 / reject, and C 35 / C 24 / C 16 / reject. For each combination only the highest strength class assignments were analysed. The order of the legend in Figure 8 corresponds to the order of the given strength class combinations.

A high percentage increase of yield can be observed for strength class C 24 in the combination C 24 / C 16 / reject as well as in the combination C 24 / C 18 / reject, when long logs are (pre-) graded. An example is the combination C 24 / C 18 / reject: A threshold value for long logs of more than 11000 leads to an increase in yield of about 20 percentage points. The number of long logs is reduced from 108 to 58. In other words, without pre-grading 141 of 630 boards are classified to C 24; whereas with pre-grading the long logs using a setting of 11000, 113 of 289 boards are classified to C 24.

4 DISCUSSION

Previous investigations have shown the relation between different stages in the wood (timber) production chain [2],[3],[4],[5],[6]. None of them measured all the variables for calculating the dynamic MOE based on longitudinal vibration for each level of the processing chain. It allows to represent a complete overview from the tree to the logs and finally to the boards.

The relation between the average velocity of standing trees and long logs is comparable to [3], although [3] estimated the density of logs. For this investigation weight and volume were measured and consequently density was calculated. Further the average velocity is compared to long logs of about 13 m in this paper, not to short logs. Other investigations have been carried out to identify the quality of timber before harvest by means of acoustic velocities (time of flight). The results here are more in line with results of [7], where 698 trees of Douglas-fir were investigated. They found a coefficient of determination of 0.25 between sound velocities measured by the time of flight method and the corresponding eigenfrequency of the butt logs. The relation between dynamic measurements at standing trees and logs differs. This is due to different species and diameters, but also the stand age (of the trees) influences the relation [8],[9].

The relation between short and long logs is very strong; the $r^2$-value is 0.94. Similar results have been published by [10]. They researched the acoustic velocity between long and short logs of Douglas-fir ($n = 85, r = 0.9$).

The dynamic MOE of short logs correlates well with the average MOE of wet boards obtained from the short log (see Figure 4). The coefficient of determination found is 0.7 and higher than the $r^2$-value for Douglas-fir ($r^2 = 0.5$) in [11]. A good relationship exists between the dynamic MOE of individual wet boards obtained from the log and the dynamic MOE of the log (see Figure 5). The relationship given in [11] is also weaker ($r^2 = 0.3$) than in this investigation ($r^2 = 0.55$).

The relationship between dynamic MOEs measured in wet (green) and dried conditions is very strong [1],[12].
The important point is to ensure that the moisture content of boards in green condition is above fibre saturation point. In machine strength grading this knowledge is already successfully applied. The correlation of determination in a linear regression analysis of about 0.95 displayed the strong correlation. Dynamic MOE of the dry board is higher than dynamic MOE of the wet board. The ratio (see Figure 7) between MOE of wet conditions and MOE of moisture content of 12 % is in line with [1]. This makes it possible to calculate dynamic MOE of dry conditions – based on wet conditions – without knowledge of the actual timber moisture content.

The input material of the timber manufacturing process is characterized by quality shifts [12]. Timber quality shifts are identifiable due to silviculture treatment or to a site index [13]. It is clearly shown that the correlations of the dynamic MOE within the processing chain are strong. Pre-grading timber promises higher yields of sawn timber strength classes. However, for a successful implementation tracing of boards from the standing tree or the long logs must be guaranteed along the production chain.

This investigation illustrates pre-grading long logs to achieve higher yields of strength graded dry boards (see Figure 8). For all strength class combinations, the yield for the highest strength class increases when logs are pre-graded. Depending on the strength class – the yield increase can be more than 20 percentage points for C24. It must be emphasized that the mechanical properties of the wood of trees used for this investigation seems to be on the lower side of the spectrum (see Table 1).

5 CONCLUSIONS

This study developed relationships of dynamic measurements between different stages of wood along the production chain: standing tree, long log, short log, wet board, and dry board. Eigenfrequency, length, weight, volume, and moisture were measured for all logs and boards in order to calculate the dynamic modulus of elasticity. The goal was to assess timber obtained from logs at an early stage of processing, not as end product. The investigation shows possibilities for which stage pre-grading is suitable. The effects of pre-grading long logs on the yield of different strength class combinations serves as an example.

ACKNOWLEDGEMENT

The study is part of a project which is financed by the Bavarian state. It investigates the relationship between growing-space-availability and the growth-dependent outer and inner wood quality for Douglas-fir in Bavaria. The wood was sponsored by the Bayerischen Staatsforsten. MiCROTEC is thanked for their support during grading.

REFERENCES

EVALUATION OF PHYSICAL PROPERTIES AND MACHINING OF AFRICAN MAHOGANY (Khaya ivorensis A. Chev.) WOOD

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ABSTRACT: This study had as main objective the evaluation of physical properties and the quality of the machining process results of Khaya ivorensis A. Chev wood. Samples of trees implanted in the Campus of the Rural Federal University of Rio de Janeiro with approximately 15 years of age were evaluated. Samples with dimensions of 50 x 30 x 20 mm was produced and appraised, perfectly oriented in the three cut plans (transversal, radial and tangential) for physical characterization of the material. 30 specimens with dimensions: 600mm x 120mm x 25 mm were produced for the machining tests of planning and sanding. The samples for physical characterization show that the evaluated wood of African mahogany presented medium to high basic and apparent density, and the results about the surface quality showed that the samples processed with smaller progress speed generated less superficial defects and better parameters of roughness.

KEYWORDS: African mahogany, wood roughness, wood machining.

1 INTRODUCTION

With the current and growing pressure on the suppression of Brazilian native forests for wood industrial segment intensified the demand for new maintainable sources of wood. So, the planted forests conquered place in the market as protagonists in the provisioning of wood industry.

The industries of the Brazilian wood segment to maintain the quality of the products still possess restrictions to the wood derived from reforestation, where prevail species of Eucalyptus and Pinus. Even so, the knowledge lack by the producers and consumers about these and other potential species for such purpose is still the main factor for such discrimination.

In the current days this picture comes being modified, the wood of reforestation are revealed promising once the old comparative advantage represented by the use of the wood of native forests becomes more and more inefficient.

The species Khaya ivorensis A. Chev., known commonly as African mahogany, bisselon and mahogany-of-Gambian, is an exotic forest species in Brazil that belongs to the Meliaceae family.

The Khaya gender is native of Africa and Madagascar and other species of this gender are also known as African mahogany (Khaya anthotheca, Khaya senegalensis, Khaya madagascariensis and Khaya nyasica) presenting great economic interest in the wood industry. [1]

The African mahogany supports the conditions of dry periods of the year (of 4 to 6 months), in which the plant paralyzes its growth. Having supplemental irrigation, the species does grow in the months more colds of the year. It grows better well in structured soils; it supports the conditions where the clay texts are smaller than 68 % and brief periods of high humidity. Starting from the 7 and 8 years of age, period that corresponds to the beginning of the fructification, the plant presents fast increments in diameter and opening of the cup. After about 16 years, it presents DAP of 60 cm. Between 16 and 20 years of age the trees of African mahogany present good dimensions for the obtaining of sawnwood. [1]
The African mahogany (Khaya ivorensis A. Chev.) was introduced in Brazil to substitute the Brazilian mahogany (Swietenia macrophylla King) due to its high resistance to the Hypsipyla grandella, the main plague of the Brazilian mahogany. The Brazilian mahogany wood (Swietenia macrophylla) became a luxury article and, due to the strong legal restrictions to its exploration in the natural condition and cultivation difficulties in commercial scale due to the attack of the plague-do-sprout-terminal, this species became substituted in Brazil by commercial cultivations of African mahogany (Khaya ivorensis), that still needs more studies on silvicultural techniques. [2]

The species Khaya ivorensis has been indicated for plantation in function of its good development and production of beautiful and resistant wood with high value in the international market. It is the exotic species of mahogany that presents better growth in Brazil, being used more and more by the forest planters. In favorable conditions the species has increment of up to 40 m³/hectare/year.

The objective of the present study was evaluated the physical properties and the behavior of the wood of Khaya ivorensis A. Chev. implanted in Brazil, in the rehearsals of machining tests of planning and sanding, with emphasis in the surface evaluation of the samples.

2 MATERIAL AND METHODS

In the present study were evaluated 3 boards retired of the first log (first 3 meters) from 3 individuals of Khaya ivorensis A. Chev., coming of the Rural Federal University of Rio de Janeiro Campus, in Seropédica, State of Rio de Janeiro, Brazil. Each log presented initially 210 cm, being divided in three smaller logs with 70 cm each one. These were processed later on again in three boards with dimensions of 36 cm of width and 7 cm of thickness (Figure 1).

Starting from the boards were produced 30 specimens with the following dimensions: 600mm x 120mm x 25mm (length x width x thickness) and separated for the three treatments, 10 for each speed progress planning (6m/min, 12.4m/min and 24m/min). Also were made 20 specimens with dimensions of 50mm x 30mm x 20mm for physical properties determination: basic density, apparent density and dimensional stability; the specimens were made according to the procedures of Brazilian standards for wood structures [3].

In the evaluation of the wood quality in the machining processes were made planning tests with three different progress speeds: 6m/min, 12.4m/min and 24m/min, being used a planning machine with one axis and two knives, with rotation of 3600 rpm; and sanding, being used a sanding machine with 2,70m of distance between the motor axis and the guide axis, with a sandpaper number 80 and 700 rpm. The sanding process was executed in the opposite face where the planning test was made, where each sample was submitted to the sandpaper by 20 seconds.

The methodology for the evaluation of the samples surface used a system of reading the roughness through a precision meter clock with resolution of 0.001mm with a coupled needle and a dendrometric table for displacement of the sample. The Figure 2 demonstrates the equipment and an example of the roughness graph obtained in one of the mensurations.

![Figure 2: Mensuration of the surface roughness](image)

Starting from the roughness graph were determinate the parameters Ra, Rz and Rt, related to the called “picks” and “valleys” of the graph and described in full detail in the study [4].

The mensuration areas or lines of the roughness were systematically chosen on the machining surface, highlighting representative areas of the face, where defects generated from the treatments were evidenced. (Figure 3)
The mensurations were made in two directions, transversal and longitudinal in relation to the disposition of the fibers, six mensurations were made in each specimen, three in the planned surface and three in sanded surface.

3 RESULTS AND DISCUSSION

The Tables 1, 2 and 3 present the results obtained along the tests of physical properties and machining.

<table>
<thead>
<tr>
<th>Table 1: Values of basic and apparent density of the samples of African mahogany</th>
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<tr>
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<td>(g/cm³)</td>
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<td>C.V. (%)</td>
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<th>Table 3: Averages and C.V. for the parameters Ra, Rt and Rz of the roughness for different progress speeds:</th>
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</tr>
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<td>6</td>
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<tr>
<td>(14,68)</td>
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<tr>
<td>24</td>
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<tr>
<td>(6,94)</td>
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In the physical evaluations the African mahogany the results presented medium to high basic and apparent density comparable to some native species of Brazil; the same happened with the values of volumetric retraction. For the values of apparent density and basic density the samples presented low values for standard deviation and variation coefficient, characterizing a homogeneous group with low variability of these characteristics or properties.

The percentile of volumetric variation of the samples also presented low values for standard deviation and for the variation coefficient, showing that this characteristic of the samples also possesses homogeneity. For the volumetric retraction, the obtained average was 12.57%.

The medium value of Ra found in the planning test in the same direction of the grain orientation, for the progress speed of 6 m/min and 12.4 m/min (26.26 µm and 27.50 µm, respectively) didn't obtain significant difference among them, even so, for the progress speed of 24 m/min there was significant difference (18.64 µm). Such values were larger than found by [5] in surfaces of Populus tremula prepared in the same conditions (7.05 µm), with the use of the electronic roughness meter.

In the directionse against the grain orientation there was not significant difference for the values of Ra found for the three evaluated progress speeds.

For the sanding test, with sandpaper of grain 80, there was significant difference for the values of Ra between the three treatments, being the progress of 6m/min the treatment with better result (12,22 µm).

In the machining tests of the wood the samples had reasonable results, presenting great variations among the pieces and inside of the same surface. The samples behaved well in the sanding process that improved the parameters of roughness evaluated.

4 CONCLUSIONS

With the results found in the present research was possible to list the following conclusions:

- In the physical evaluations the wood of African mahogany presented apparent and basic density considered medium to high, comparable to some Brazilian native species.

- The methodology for mensuration of superficial roughness presented considerable good results. In spite of being less precise than the use of electronic equipments, the obtained values with the used technique the technique can be compared with another superficial roughness meter equipments.

- The African mahogany wood evaluated showed good potential for use for the Brazilian wood industry sector.

- The genetic material needs to be better studied together the silvicultural characteristics of the specie.

- The material was evaluated with 15 years of age, an increment in the age possibly turns it stable and uniform as it happens in general in the other species.

- It should be evaluated a larger number of individuals and origins for a more precise qualification of the African mahogany planted in Brazil.

- Researches that quantify mechanical and resistance properties of this material should be motivated.

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REFERENCES


NON-DESTRUCTIVE TESTING OF WOOD MEMBERS FROM EXISTING TIMBER BUILDINGS BY USE OF ULTRASONIC METHOD

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ABSTRACT: This paper presents the results of an experimental study of non-destructive testing of wood members reclaimed from existing timber buildings. Destructive bending tests were also conducted to identify the moduli of elasticity and rupture as well as the specific gravity of wood. Relationships between the velocity of ultrasound and the moduli of elasticity and rupture were constructed in order to establish a relevant and ready-for-use protocol to determine in-situ the bending stiffness and strength of wood members of existing timber structures using non-destructive testing methods. The test results indicated good mapping relationship of the MOE and MOR with the specific gravity while the mapping of MOE, MOR and specific gravity to the sound propagation velocity from the ultrasonic tests was less desirable. By considering the mean values of the test results, such mapping relationships were improved slightly. More tests are in need to construct the database for a more robust and relevant relationship between these properties.

KEYWORDS: Non-destructive testing, timber structures, ultrasonic tests, strength, stiffness

1 INTRODUCTION

Wood members of historical timber structures may experience significant damages during their service life due to fungal decay, moisture changes and/or external loadings. These damages may significantly affect the integrity of wood members and impair their load carrying capacity. Inspection and retrofitting of such members are necessary and urgent to preserve the social and cultural values of the historical timber structures. For these members, non-destructive testing (NDT) techniques are the natural choice to provide information about the degree of damage and the remaining physical properties of wood, such as moduli of elasticity and rupture (MOE and MOR), since they do not cause further damages to the tested members of the existing buildings.

NDT is a wide group of analysis techniques used in science and industry. For wood based products, commonly used NDT methods include techniques such as ultrasonic testing, stress-wave, radiographic testing, drill resistance, screw withdrawal, pilodyn method, gamma radiation or X-ray method, etc [1-4]. When the NDT methods are used for evaluation of remaining strength of built-in structural members of existing buildings, difficulty can rise from many aspects, such as inaccessibility of the members, variable moisture distribution, seasonal temperature changes, interference from neighbouring members, localized discontinuities (such as nails), etc [4]. This brings questions of validity to even well developed NDT methods when old existing buildings are to be evaluated, where these difficulties can be exaggerated due to the long history of service and exposure to manmade and natural hazards.

Ultrasonic method in conjunction with measurement of density is a desirable choice since it can offer the ability to rapidly screen large volumes of timber [5]. This paper presents the results of an experimental study of non-destructive testing of wood members reclaimed from a historical building by use of ultrasonic method.
Destructive tests were also conducted to measure the MOE and MOR as well as the specific gravity of the wood. Relationships were then established based on regression analysis among the propagation velocity of the sound that transversely goes through the wood, the specific gravity, and the MOE and MOR. The generated knowledge can be used as reference for evaluation of MOE and MOR of wood members in historical buildings.

2 SPECIMEN PREPARATION AND TEST SETUP

The specimens used for the ultrasonic tests, static bending tests and tests of specific gravity were made from reclaimed wood members from a historical timber building that has been recently torn down, as shown in Figure 1. Most of the wood members came with nail holes (after nails were removed) and other types of damages. When making specimens for ultrasonic tests, special attention was paid to keep down the influence of the damages by using as much the interior material of the wood members as possible. Moisture content and cross-sectional dimension were measured for each specimen.

2.1 ULTRASONIC TEST

The specimens for ultrasonic tests were sized into 200×61×800 mm³ with the surfaces furnished. In total 91 specimens were prepared. Each specimen was tested edgewise, i.e., the ultrasound went through the shorter dimension of the cross-section. Twenty measurements were made from each specimen in the upper and lower zones of its flat sides, as shown in Figure 2. The spacing of the measurements was roughly 80 mm. The ultrasonic tests were conducted by use of a TICO® ultrasonic instrument. The two transducers (one as transmitter and the other as receiver) of the instrument were placed on the two opposite flat sides of each specimen. To increase the efficiency of the process by reducing the losses in the ultrasonic wave energy, butter was used at the interface as the coupling agent.

2.2 STATIC BENDING TEST

Static bending tests were carried out in accordance with ASTM standard D143 “Standard Test Methods for Small Clear Specimens of Timber” [7]. Every specimen used in the ultrasonic tests was sized into two bending test specimens in a size of 50×50×760 mm³. The corresponding test results were averaged for further discussion with ultrasonic test results. The tests were conducted on a Sintech® machine with a standard deflection measurement yoke for measuring the midspan deflection, as shown in Figure 3. The load was applied vertically via a wooden loading head at a constant rate of 2.5 mm/min. The test results included the applied load and the midspan deflection, based on which the moduli of elasticity and rupture were evaluated.
2.3 MEASUREMENT OF SPECIFIC GRAVITY

The specific gravity (relative density) of wood was also measured following the ASTM standard D 2395-02 “Standard Test Methods for Specific Gravity of Wood and Wood-Based Materials” [8]. From each static bending test specimen, one specimen was fabricated using the wood in the vicinity of the failure zone in a size of 50×50×50 mm³. The specimens were measured for weight before and after 24 hours of oven-dry. The weight change was then used for evaluation of the moisture content and the specific gravity of the wood as:

\[ MC = 100(I - F)/F \]

\[ SG = KW(1 + MC/100)Lwt \]  

(1)

where \( MC \) is the moisture content; \( I \) and \( F \) are the weight measured before and after 24 hours of oven-dry; \( SG \) is the specific gravity; \( W \) is the oven-dry weight; \( L, w, t \) are the length, width and thickness of the specimen, respectively; and \( K \) is a constant of 453.59. The test setup is shown in Figure 4.

![Figure 4: Oven-dry for measurement of specific gravity](image)

3 TEST RESULTS

The ultrasonic test results included the sound propagation velocities measured at 20 points on each specimen of totally 91 specimens, among which every 10 measurements of the sound propagation velocities were corresponding to the results of one bending test specimen, from which one measurement of the MOE and MOR, respectively, was obtained. For each bending test specimen, one measurement of specific gravity was also established via the over-dry method. Thus, the averaged measurement of the sound propagation velocity, the MOE and MOR, and the specific gravity can be mapped based on a one to one correspondence to establish mathematic relationships.

![Figure 5: Relationship between the specific gravity and sound propagation velocity](image)

![Figure 6: Relationship between the specific gravity and MOE and MOR](image)

The mapping results of the sound propagation velocity to the specific gravity are plotted in Figure 5; and the mapping results of the MOE and MOR, respectively, to the specific gravity are plotted in Figure 6.

Similarly, the mapping results of the sound propagation velocity and the MOE and MOR are plotted in Figure 7, respectively.
To quantify the variation of the test results, the mean value, standard deviation (Std.) and the coefficient of variation (COV) of the results of the sound propagation velocity, MOE, MOR and the specific gravity were evaluated, respectively, and the variation characteristics are listed in Table 1. For the sound propagation velocity, the mean value and COV were first evaluated every one of the 91 specimens considering two testing zones separately. Then their variation characteristics, i.e., mean value and COV, were further evaluated, respectively. The results are listed in the second and third rows of Table 1.

Table 1: Mean values and variation of the test results

<table>
<thead>
<tr>
<th>Test results</th>
<th>Mean</th>
<th>Std.</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultrasonic velocity (m/s)</td>
<td>2798</td>
<td>372</td>
<td>0.13</td>
</tr>
<tr>
<td>COV of each 10 measurements</td>
<td>0.07</td>
<td>0.08</td>
<td>1.14</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>12133</td>
<td>2248</td>
<td>0.19</td>
</tr>
<tr>
<td>Modulus of Rupture (MPa)</td>
<td>60</td>
<td>17</td>
<td>0.28</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>0.46</td>
<td>0.05</td>
<td>0.10</td>
</tr>
</tbody>
</table>

4 Discussion

As can be seen from Table 1, the test results of the ultrasonic tests exhibited significant variation, mainly due to the deterioration of wood, internal decay and damages caused by metal connectors (such as nails). This is confirmed by the mapping results in Figures 5 to 7, where the mapping between the destructive test results, i.e., the MOE, MOR and specific gravity, is generally fine, while that with the ultrasonic test results is statistically unsound.

To further investigate the possibility of improving the mapping between destructive test results and the ultrasonic test results, the results were reorganized based on the mean values of the sound propagation velocity and the specific gravity, respectively. First, the range of the results of sound propagation velocity was divided into certain groups (50 in this study), within which the results of the specific gravity were averaged. The reorganized mapping results are shown in Figures 8 and 9.
Secondly, the test results were divided into the same number of groups based on the values of the specific gravity and the test results of the MOE and MOR were averaged. The corresponding mapping results between the specific gravity and MOE and MOR, respectively, are shown in Figure 10.

It can be seen that such reorganization did improve the mapping quality although the improvement was limited due to the variation of wood quality and accuracy of the used ultrasonic testing method. The coefficient of determination (R-squared) was increased to 0.15 for MOE and 0.08 for MOR.

5 Conclusion

This paper presents the results of an experimental study of non-destructive testing of wood members reclaimed from existing timber buildings. Destructive bending tests were also conducted to identify the moduli of elasticity and rupture as well as the specific gravity of wood. Relationships between the velocity of ultrasound and the moduli of elasticity and rupture were constructed in order to establish a relevant and ready-for-use protocol to determine in-situ the bending stiffness and strength of wood members of existing timber structures using non-destructive testing methods. The test results indicated good mapping relationship of the MOE and MOR with the specific gravity while the mapping of MOE, MOR and specific gravity to the sound propagation velocity from the ultrasonic tests was less desirable. By considering the mean values of the test results, such mapping relationships were improved slightly. More tests are in need to construct the database for a more robust and relevant relationship between these properties.

ACKNOWLEDGEMENT

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REFERENCES


PROPERTIES OF CLEAR WOOD AND STRUCTURAL TIMBER OF PINUS HALEPENSIS FROM NORTH-EASTERN SPAIN

Eduard Correal-Mòdol¹, Marcel Vilches Casals²

ABSTRACT: The Aleppo pine is a common tree in the forests of the semi-arid climate area on north-eastern Spain (Catalonia) but there is a lack of information on the properties of its timber. Nowadays, it is mainly being used in packaging and on particle boards due to the low quality of the stems. The study characterizes the physical and mechanical properties of the clear wood following the UNE 56 series standards. It was also done a strength grading of the structural timber according to the UNE-EN 14081:2006. The clear wood had good physical and mechanical properties, and the visually strength graded timber like it was ME-2 resulted C14.

KEYWORDS: Pinus halepensis, Clear wood, Structural timber, Wood properties characterization, Strength class

1 INTRODUCTION

The Aleppo pine (Pinus halepensis) is a tree species spread all around the Mediterranean basin including the islands. Its continental range extends from northern Africa (Morocco, Algeria, Tunisia and Libya) and Middle East (Syria, Lebanon, Jordan, Palestina and Turkey), up to southern Mediterranean Europe (eastern Greece, Croatia, northern Italy, eastern France and eastern Spain) (Bioversity International, 2011).

Figure 1: Pinus halepensis distribution (Bioversity International, 2011)

It grows mostly from sea level to 1,000 m. In Spain is most abundant from 200 m to 600 m because lower altitudes are occupied by crops or urbanized areas. In meridional latitudes can be found even at 1,600m. The Aleppo pine usually prefers poor carbonated soils but can also live on chalky ones. It bears semi-arid climate with dry and hot summers, cold winters and large annual thermal range. The species needs an average annual rainfall of 350-700 mm but can survive with only 200-300 mm.

It is a xerophilous and heliophilous species with little and irregular annual growth. The biggest trees may reach 50 cm of diameter and 20 m height but they usually don’t get over 40 cm and 15m. The stem it is not very straight, large branches are common, therefore has low quality. It has an average growth of 2.4 m³/ha/yr (Burriel et al. 2000-2004).

For all these reasons, the Aleppo Pine nowadays in Spain it is mainly being used by the packaging industry for producing pallets. However, traditionally round trunks were used as beams in isolated houses located nearby the forests, especially in the Mediterranean coast where it is the most abundant tree. The goal of the study was to characterize the physical, mechanical and structural properties for knowing the potential of the Aleppo pine for building. This is quite relevant because nowadays is widely believed that Aleppo pine timber is not suitable for structural use despite its beams are found in ancient buildings.

2 MATERIALS AND METHODS

The study is divided in two: characterization of clear wood and characterization of structural timber. The timber used in the experiment came from two locations in Catalonia (North-eastern Spain). Both locations belong to the region of provenance “Catalunya interior”
as defined by Gil et al. (1996). 100 beams were from the “Serra de Prades” (Tarragona) and they had nominal dimensions of $50 \times 150 \times 2850$ mm. There were other 81 smaller beams from “Prats de Lluçanes” (Barcelona) with nominal dimensions of $45 \times 110 \times 2000$ mm. Prior to the visual grading, the beams were kiln dried up to the 12% of moisture to avoid dimensional changes, deformations or fungal attacks.

68 beams of Aleppo Pine from “Serra de Prades” and “El Lluçanès” were accepted after being visually graded according to the criteria of UNE-EN 14081-1:2006+A1 and UNE 56544:2007. The bending tests were done on a Hoytom CM-DF 300/A1500 according to UNE-EN 408:2011. The experiment design and the characteristic values calculation followed the norm UNE-EN 384:2010 and the strength class was assigned in agreement of the UNE-EN 338:2010 standard.

![Figure 2: Bending strength test geometry for the structural samples (UNE 408:2011)](image)

The UNE 56544:2007 is the Spanish standard related to the EN 14081 and is intended to define the criteria to visually grade the Spanish softwoods. The norm defines two strength classes for beams less than 7 mm thick, ME-1 and ME-2. ME-1 is first quality wood, ME-2 stands for average quality, and lower grade timber is rejected. At the moment Pinus halepensis is not yet included on the UNE 56544:2007. However, the timber was classified following the criterion of the norm to be able to compare the grading yield and the structural strength with the other four species of pines already included.

Clear wood test samples were made according to UNE 56528:1978 (AENOR, 1978) and afterwards conditioned at 20°C and 65% of relative air humidity until they reached 12% of moisture content. All the clear wood specimens were extracted from the rejected beams from “Serra de Prades”. 146 specimens were tested on each of the six properties analyzed on an Incotecnic MUTC-200.

![Figure 3: Clear wood sample on the bending test](image)

### Table 1: Properties of clear wood and UNE norms

<table>
<thead>
<tr>
<th>Property</th>
<th>UNE norms</th>
<th>Test samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>56531:1977</td>
<td>20 \times 30</td>
</tr>
<tr>
<td>Shrinkage (%)</td>
<td>56533:1977</td>
<td>20 \times 40</td>
</tr>
<tr>
<td>Hygroscopicity (kg/m³)</td>
<td>56532:1977</td>
<td>20 \times 40</td>
</tr>
<tr>
<td>Hardness (mm⁻¹)</td>
<td>56534:1977</td>
<td>20 \times 40</td>
</tr>
<tr>
<td>Compression strength (kg/cm²)</td>
<td>56535:1977</td>
<td>20 \times 60</td>
</tr>
<tr>
<td>Bending strength (kg/cm²)</td>
<td>56537:1979</td>
<td>20 \times 300</td>
</tr>
</tbody>
</table>

### 3 RESULTS AND DISCUSSION

#### 3.1 CLEAR WOOD PROPERTIES

The tests conducted on clear wood specimens showed results similar to those published by Peraza Oramas (1964), Peraza Sánchez (2004) and Burillo (2006). The hardness and the density are slightly higher and the compressive strength is greater. See following table.

### Table 2: Physical and mechanical clear wood properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Results</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>$\bar{x}$</td>
<td>589.91</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>55.01</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>500.71</td>
</tr>
<tr>
<td>Shrinkage (%)</td>
<td>$\bar{x}$</td>
<td>14.13</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>2.33</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>10.11</td>
</tr>
<tr>
<td>Shrinkage coefficient (%)</td>
<td>$\bar{x}$</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>0.072</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>0.335</td>
</tr>
<tr>
<td>Hygroscopicity (kg/m³)</td>
<td>$\bar{x}$</td>
<td>0.0032</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>0.0004</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>0.0027</td>
</tr>
<tr>
<td>Hardness (mm⁻¹)</td>
<td>$\bar{x}$</td>
<td>4.26</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>2.44</td>
</tr>
<tr>
<td>Axial compressive strength (kg/cm²)</td>
<td>$\bar{x}$</td>
<td>594.91</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>76.57</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>453.22</td>
</tr>
<tr>
<td>Bending strength (kg/cm²)</td>
<td>$\bar{x}$</td>
<td>1,106.82</td>
</tr>
<tr>
<td></td>
<td>$S_{n-1}$</td>
<td>175.23</td>
</tr>
<tr>
<td></td>
<td>$P_{3}$</td>
<td>804.07</td>
</tr>
</tbody>
</table>

The Aleppo pine clear wood is slightly denser, harder, and has more bending and compression strength than the other most common pines grown in Spain as a source of timber. However, the reaction to moisture is more unstable because it has more shrinkage. (See table below).
Table 3: Physical and mechanical clear wood properties of the most common pines in Spain used as timber

<table>
<thead>
<tr>
<th>Property</th>
<th>Sn-1</th>
<th>P5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>523.2</td>
<td>29.4</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>579.1</td>
<td>80.9</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>527.5</td>
<td>60.5</td>
</tr>
<tr>
<td>P. nigra</td>
<td>563.8</td>
<td>57.0</td>
</tr>
<tr>
<td>Shrinkage (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>12.5</td>
<td>2.1</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>12.5</td>
<td>2.2</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>16.3</td>
<td>8.9</td>
</tr>
<tr>
<td>P. nigra</td>
<td>14.8</td>
<td>2.8</td>
</tr>
<tr>
<td>Shrinkage coefficient (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>0.38</td>
<td>0.07</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>0.38</td>
<td>0.06</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>P. nigra</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Higroscopicity (kg/m³)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>0.0032</td>
<td>0.0004</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>0.0035</td>
<td>0.0005</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>P. nigra</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Hardness (mm⁻¹)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>2.59</td>
<td>0.61</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>2.85</td>
<td>1.20</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>2.62</td>
<td>0.97</td>
</tr>
<tr>
<td>P. nigra</td>
<td>3.25</td>
<td>0.97</td>
</tr>
<tr>
<td>Axial compressive strength (kg/cm²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>471.0</td>
<td>34.7</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>439.9</td>
<td>77.2</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>570.1</td>
<td>299.3</td>
</tr>
<tr>
<td>P. nigra</td>
<td>539.0</td>
<td>77.2</td>
</tr>
<tr>
<td>Bending strength (kg/cm²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. radiata</td>
<td>917.0</td>
<td>63.8</td>
</tr>
<tr>
<td>P. pinaster</td>
<td>845.6</td>
<td>178.9</td>
</tr>
<tr>
<td>P. sylvestris</td>
<td>995.1</td>
<td>186.5</td>
</tr>
<tr>
<td>P. nigra</td>
<td>1,053.0</td>
<td>178.7</td>
</tr>
</tbody>
</table>

* INCAFUST

3.2 MECHANICAL CHARACTERIZATION

The 77% of the beams were rejected on the visual grading. The batch of “Serra de Prades” was entirely rejected because big knots and large deformations, however 41 beams of the batch of “El Lluçanes” were classified as ME-2-like specimens. Unfortunately there were very few ME-1.

Table 4: Beams according location and quality

<table>
<thead>
<tr>
<th>Location</th>
<th>Beams</th>
<th>ME-1</th>
<th>ME-2</th>
<th>Rejected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prades</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>100 (27%)</td>
</tr>
<tr>
<td>Lluçanes</td>
<td>81</td>
<td>21</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>Total</td>
<td>181</td>
<td>21</td>
<td>20</td>
<td>140</td>
</tr>
<tr>
<td>Proportion</td>
<td>100%</td>
<td>11.6%</td>
<td>11.1%</td>
<td>77.3%</td>
</tr>
</tbody>
</table>

Table 5: MOR, MOE and density values for visually graded timber. Descriptive statistical values for the batch

<table>
<thead>
<tr>
<th>Property</th>
<th>ME-2-like</th>
<th>Rejected</th>
</tr>
</thead>
<tbody>
<tr>
<td>MOR (N/mm²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sn-1</td>
<td>3.86</td>
<td>6.96</td>
</tr>
<tr>
<td>P5</td>
<td>18.76</td>
<td>8.77</td>
</tr>
<tr>
<td>MOE (N/mm²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sn-1</td>
<td>1,150.35</td>
<td>1,797.28</td>
</tr>
<tr>
<td>P5</td>
<td>8,901.50</td>
<td>8,168.56</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sn-1</td>
<td>40.29</td>
<td>37.28</td>
</tr>
<tr>
<td>P5</td>
<td>575.24</td>
<td>513.65</td>
</tr>
</tbody>
</table>

The norm UNE-EN 384:2010 considers several correction coefficients. According to it and the available sample we considered the size of the batch (ks=0,78), the classification procedure (kv=1), the depth of the beam (kD=1 on average) and the test sample length (kl=1). Such corrections are essential in order to avoid an overestimation of the given strength class. Afterwards the characteristic values are as follow.

Table 6: Characteristic values according to UNE-EN 384:2010

<table>
<thead>
<tr>
<th>Property</th>
<th>ME-2-like</th>
<th>Rejected</th>
</tr>
</thead>
<tbody>
<tr>
<td>MOR (N/mm²)</td>
<td>13.62</td>
<td>6.37</td>
</tr>
<tr>
<td>MOE (N/mm²)</td>
<td>8.777.27</td>
<td>8422.83</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>679.42</td>
<td>606.67</td>
</tr>
<tr>
<td>Strength class</td>
<td>C14</td>
<td>Rejected</td>
</tr>
</tbody>
</table>

Contrasting the three characteristic values it can be seen that the Aleppo pine ME-2-like timber is relatively stiff and heavy comparing it with its loading capacity. The MOR is what limits the strength class to C14. The cause of this behaviour is the high incidence of the defects on the resistance. This is clearly seen on the great difference between the MOR of the clear wood and the MOR of the structural timber. While the MOR of the Aleppo's pine wood of small dimensions is similar or even higher than on other pines, the MOR of the structural timber is quite low. Thus, the presence of imperfections in the timber of Aleppo pine resulted critical. This is also seen on the great difference on the MOR of the rejected timber and of the ME-2-like type, especially if compared with MOE and density.

Unfortunately, the Aleppo pine demonstrated a far lower strength class than the Spanish species that already have a visual grading norm: C18 vs C14 (UNE 56544). Another special feature of the Aleppo pine timber is the behaviour when breaking. The beams don't crunch or crack much during the load because the timber is rigid, but the breakings are sudden and violent and all the energy is released at once. Most beams resulted seriously damaged when broke. This behaviour is probably due to the material can't much cope with plastic strain.
Figure 4: Shattered beam of Aleppo pine on bending test

4 CONCLUSIONS

The clear wood of Pinus halepensis has better resistance than other Spanish coniferous used in timber structures. However there is little difference. Unfortunately the timber has a low quality and a lot of rejection. Thus, it is necessary to invest on silvicultural works.

The Aleppo Pine from North-eastern Spain (Catalonia) is the native pine timber with the lowest load capacity. The timber visually graded according to the criteria of the ME-2 class on the UNE 56544:2007 might be considered C14.

Only one quality strength class should be purposed for this species if included on a visual grading norm. On one hand, the high quality timber is rare, and on the other, it will ensure at least a C14 strength class when including also the best samples on a unique batch.

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ASSESSING PROPERTIES OF TROPICAL HARDWOODS TO MANUFACTURE ENGINEERED WOOD PRODUCTS

Cláudio H. S. Del Menezzi

ABSTRACT: The engineered wood product (EWP) is a class of products whose market and utilization have risen significantly, mainly in North America, Europe and Japan. These products are manufactured from several types of wood, adhesives and additives and they combine the better property of each one in order to produce an enhanced material. In spite of huge production and consumption of tropical hardwoods, unfortunately EWP manufacturing plants, even from plantation wood, does exist in Brazil. In this context, the paper presents a general overview of the research efforts that have been done in Brazil to assess properties of tropical hardwoods aiming to produce EWP. These studies have been focused on adhesive bonding, nondestructive evaluation, engineering properties and experimental manufacturing and testing of EWP. At least 12 Brazilian Amazonian hardwoods species have been preliminarily studied so far. The majority of these properties have never been determined before for these species. The results regarding the properties of laminated veneer lumber, glulam beams and I-beams from these species are discussed. In general, the potential of the tropical hardwood to manufacture EWP has been demonstrated and studies should be continued and deepen.

KEYWORDS: tropical hardwood, engineered wood product.

1 INTRODUCTION

The engineered wood product (EWP) is a class of material whose utilization is very common in North America, Europe and Japan. They combine the properties of wood based materials, water resistant adhesives and additives in order to produce a material with enhanced properties. The following products can be classified as EWP: laminated veneer lumber (LVL), oriented strand lumber (OSL), parallel strand lumber (PSL), laminated strand lumber (LSL), and glued laminated lumber (glulam) and wood I-beams.

The utilization of tropical hardwood to manufacture EWP is rare, or even it has not been happening, and most of the production is based on temperate softwood and hardwood. Despite of this, it is believed that tropical hardwood has a huge potential for manufacturing EWP. The conception behind the EWP together with sustainable forest management practices can help open the market to the tropical hardwood.

Thus, it is very important to begin the studies about properties which are not usually determined for this kind of wood. In this context, the paper aimed at presenting a general overview of the research effort that has been developed in Brazil to assess these properties.

2 MATERIAL PROPERTIES

In order to manufacture EWP it is relevant to study a group of properties which is not usually determined for tropical hardwood, such as: non destructive evaluation, adhesive bonding and wettability. Additionally, the experimental evaluation of EWP taking in account these properties it is very important in order to improve reliability and safety.

2.1 NONDESTRUCTIVE EVALUATION

The nondestructive evaluation (NDE) is the science that aims to obtain properties of the material without altering its end-use, and use this information to make decisions regarding appropriate applications. For isotropic materials, NDE is used to detect voids, inhomogeneous spots and other irregularities. However, in wood products these irregularities are common and then NDE is used to evaluate its effect on physical and mechanical properties.

NDE of tropical hardwood is not commonly performed, but recently several species have been studied: Sextonia rubra, Dinizia excelsa, Pouteria guianensis, P. pachycharpa, Holopyxidium jarana, Vatairea sericea, Chrysophyllum venezuelanense, Astronium lecontei, Endopleura uchi, Lecythis pisonis Balfourodendron...
The species were chosen according to density: Balfourodendron riedelianum, Cedrela fissilis, Cordia goeldiana, Bowdichia virgilioides, Dipteryx odorata and Tabebuia sp. Twenty samples from each species were cut and tested to determine flexural properties, totaling 120 samples. Before mechanical testing, the samples were nondestructively tested using the stress wave method, and their dynamic modulus of elasticity (E_{dw}) was determined.

According to the results, the models generated had higher values of $R^2$ (Figure 2). Thus, it can be concluded that the stress wave method is suitable for predicting flexural properties where there is wood variability, for instance among species. On the other hand, when the variability is low, for instance within wood species, the models generated had low predictability.

The utilization of transverse vibration method to grade and to test glulam beam made from Sextonia rubra wood has been evaluated [3]. Initially, laminations were nondestructively tested using the transverse vibration method. Ten glulam beams were manufactured and nondestructively tested using the same method. Afterwards, the material was tested in static bending up to rupture. It was noted that the E_{dv} value of glulam beams was only 2% lower than the E_{sb} value. Using laminations with a high modulus of elasticity did allow effective gain in stiffness (Figure 3). Therefore, the results indicated that the transverse vibration method is a valid, reliable technique both for testing glulam laminations and for assessing stiffness in finished glulam beams.

These same authors have studied the interrelationship between physical-acoustical properties of three nondestructive methods to predict the stiffness of the above mentioned tropical hardwood [4]. Fifty-two boards measuring 20 mm x 50 mm x 2000 mm (t x w x l)
were collected, ripped, planned and then tested using three methods: stress wave, transverse vibration and static bending.

![Figure 3: Relationship between nondestructive transverse vibration stiffness and static bending modulus of elasticity of glulam beam made from S. rubra. [4]](image)

According to the results, linear regression models could be fitted to explain the interrelationship between the nondestructive properties. The results of the Pearson correlation between the nondestructive variables are presented in Table 2. All the $r$ values were highly significant, except for resonance frequency $\times$ density and stress wave $\times$ density (not showed). It can also be observed a direct relationship between the three nondestructive moduli of elasticity.

Table 2: Pearson correlations ($r$) between the nondestructive variables.[4]

<table>
<thead>
<tr>
<th>Variables</th>
<th>$E_{dv}$</th>
<th>$E_{dv}$</th>
<th>$E_{ab}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{dv}$</td>
<td>0.923**</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>0.610**</td>
<td>0.620**</td>
<td>0.628**</td>
</tr>
<tr>
<td>Frequency</td>
<td>0.708**</td>
<td>0.799**</td>
<td>0.758**</td>
</tr>
<tr>
<td>$wv$</td>
<td>0.724**</td>
<td>0.764**</td>
<td>0.719**</td>
</tr>
<tr>
<td>$\delta$</td>
<td>-0.856**</td>
<td>-0.909**</td>
<td>-0.928**</td>
</tr>
</tbody>
</table>

The $r$ values ranged from 0.913 ($E_{dv} \times E_{ab}$) to 0.973 ($E_{dv} \times E_{dv}$) and were highly significant. Thus, it could be stated that simple linear regression models are suitable to explain the interrelationship between them. Regarding the variables required to determine the two dynamic moduli of elasticity, it should be highlighted that stress wave velocity achieved high $r$ (0.724) value with $E_{dv}$. It is a very important finding since it means that only measuring the stress wave velocity it is possible to predict the stiffness of the board at reasonable level.

2.2 WOOD BONDING AND WETTABILITY

The utilization of structural adhesives is mandatory in EWP manufacturing process. Therefore, the glue-line shear strength testing is required to determine the best adhesive-wood combination. Glue rate spread and pressure of assembling have been studied [5-7]. Usually, resorcinol-formaldehyde (RF) resin requires 300g/m² to achieve the strength of solid wood from Allantoma lineata, Cedrelinga catenaeformis, P. oblanceolata, Simarouba amara and S. rubra. According to [5], the best glue-line shear strength was obtained around 300 g/m² for A. lineata and S. amara. The highest glue-line shear strength value was 9.5 N/mm², for A. lineata wood. However, for both species the adhesive spread rate of 300 and 400 g/m² met the Brazilian wood design standard.

Recently, mono component polyurethane (PU) based resin has been tested to bond finger-joint from Micropholis venulosa wood [6]. Two kind of assembling positions were tested (horizontal and vertical), and spread rate and assembly pressure were 200g/m² and 1.6 N/mm² for 3 hours, respectively.

The results have not been fully satisfactory because of the wood density (0.79g/cm³) which hinder the penetration and the spreading of this kind of viscous resin. Therefore, the bending strength of both finger-jointed was about half of that for solid wood (Figure 4).

![Figure 4: Bending strength and stiffness of finger-joint and solid wood from M. venulosa. [6]](image)

It has been also studied the effect assembly pressure on the glue-shear strength of two well-known tropical hardwoods: Cedrelinga catenaeformis and Pouteria guianensis [7]. They are respectively medium (0.62 g/cm²) and high (0.98 g/cm²) density wood species. Commercial resorcinol-formaldehyde (RF) adhesive was applied at spread rate of 350g/m² and the following pressures were evaluated: 0.4, 0.6, 0.8, 1.0 and 1.2 N/mm². For each species/pressure combination 12 specimens were tested totalizing 120 specimens.

The results indicated that the glue-line shear strength for the C. catenaeformis wood was affected by the pressure improvement. The better result (13.0 N/mm²) was obtained when the highest level of pressure (1.2 N/mm²) was applied. The opposite behaviour was observed for the P. guianensis wood: lower the pressure (0.4 N/mm²), higher the glue-line shear strength (17.1 N/mm²) (Figure 5). Both wood species presented glue-line shear strength values higher than the solid wood shear strength. Nevertheless, the wood failure values were relatively low: 50.4% for C. catenaeformis; and 35.3% for P. guianensis.

Finally, it could be concluded that denser wood species can be benefited from applying low pressure, while lighter wood might require higher pressure to produce stronger bonding. It has been shown that lower wood density species (C. catenaeformis) requires higher
assembly pressure (1.2 N/mm²) than higher wood density species (*P. oblanceolata*), 0.4 N/mm².

The wettability refers to ability of a solid to be wetted by a liquid. The wetting phenomenon comprises absorption, penetration and spreading of the liquid on the wood surface. The following tropical hardwood species have been studied: *Balizia elegans*, *Trattinnickia burserifolia*, *Tachigali myrmecophyla*, *Virol michelii*, *Brosium sp.*, *Qualea dinizii*, *Rhodostemonodaphne dioica*, *Protium sagotium*, *Chamaecrysta scleroxyllum*, *Terminalia glabrescens* and *Swartezia laurifolia*.

Wood I-beam is a kind of engineered wood product widely used in Europe and North America, but is lesser-known material in Latin America. It is composed by two main parts: flange and web [8]. Then, it is easy to understand that it is very important to provide a strong connection between these parts, and it is usually done using water-resistant and durable adhesives. Therefore, a study was conducted [8] to evaluate the glue-line shear strength between three tropical hardwoods (*Cariniana micrantha*, *Cedrelinga catenaformis* and *Brosimum parinarioides*) and two types of panels (OSB, MDF). Two types of adhesives were also tested: resorcinol-formaldehyde (RF) and castor oil-based polyurethane (PU).

According to the results, the contact angle (θ) has not been consistently affected by the surface plane and the value has been found to be moderately correlated (R²=0.44) to wood density up to 0.82 g/cm³. (Figure 7) From this point, θ value has not been affected by higher wood density values.

The results showed that flexural properties of I-beams is highly affected by the flange stiffness, and transversal...
The effect of the OSB structural axis orientation on the flexural properties of I-beams has been studied by. [10] Ten I-beam measuring 200 cm of length and 20 cm of height were produced. They were flanged with tropical hardwood S. rubra and webbed with oriented strandboard (OSB): five beams were produced putting the strength structural axis parallel and five perpendicular to flange (Figure 9). The flexural properties, modulus of rupture ($f_M$) and modulus of elasticity ($E_M$), were determined. The results pointed out that I-beam made with parallel web obtained $f_M$ around 22.8 N/mm$^2$, practically the same value observed for perpendicular webbed I-beam (22.3 N/mm$^2$). Nevertheless, the difference between $E_M$ values were more pronounced, and parallel beams presented higher values (20465 N/mm$^2$) than perpendicular one (15980 N/mm$^2$). Nevertheless, bending strength was not affected by the wood used as flange, and then the three groups presented values statistically equivalents. The rupture analysis showed that most of VC beams failed because lack of adhesion between web and flange.

Strand lumber is a structural composite lumber, which has been recently introduced in building construction market, as an alternative to solid wood utilization. Due to the lack of technological information concerning this sort of material, a research was conducted to analyze the technical viability of producing LSL from Chrysophyllum sp. Six wood panels were produced, utilizing particles of 15 cm (oriented strand lumber, OSL) and 30 cm (laminated strand lumber, LSL) of length. The following tests have been performed in both samples: the mechanical tests for static bending and compression parallel; and the physical tests for thickness swelling, water absorption and linear expansion for 2 and 24 hours. In the mechanical tests there was no significant difference between both of them (Table 4). In the physical tests, the OSL presented itself more efficient since the average values for swelling, absorption and linear expansion were lower. Even though these tests have obtained lower results when compared to Chrysophyllum sp., when evaluating the physical and mechanical properties, one can assume that these types of structural composites lumber can substitute the solid sawnwood in several end uses.

### Table 3: Bending strength of some EWP made from tropical hardwood [9].

<table>
<thead>
<tr>
<th>Property</th>
<th>CM-RF</th>
<th>SA-RF</th>
<th>SA-PUR</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_M$ (N/mm$^2$)</td>
<td>11306.0</td>
<td>7300.0</td>
<td>6409.0</td>
</tr>
<tr>
<td>$f_M$ (N/mm$^2$)</td>
<td>46.3</td>
<td>29.8</td>
<td>32.1</td>
</tr>
<tr>
<td>$P_{rup}$ (kN)</td>
<td>20.1</td>
<td>12.8</td>
<td>13.8</td>
</tr>
</tbody>
</table>

Note: CM: C. micrantha; SA: S. amara; RF: resorcinol-formaldehyde resin; PUR: polyurethane resin.

The evaluation of the rupture mode revealed that all beam failed in zone subject to shear tension. The most common kind of rupture can be described as that where the failure begins between flange and web connection and runs a 45º-path way through the web up to reach the other flange.
Bending strength of LVL, LSL and OSL made from Schizolobium parahyba, and Chrysophyllum sp has been achieved higher or equivalent value than that observed in solid wood [12-13]. It has been observed that I-beam $f_M$ is sometimes equivalent to $f_m$ of the wood and it often presents failure of the top flange, which is under compression. The utilization of NDT has been improved significantly bending strength and stiffness of glulam beams made from S. rubra [3].

### Table 4: Mechanical properties of solid wood and EWP made from Chrysophyllum sp [13].

<table>
<thead>
<tr>
<th>Property</th>
<th>Solid</th>
<th>Wood</th>
<th>OSL</th>
<th>LSL</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_M$ (N/mm$^2$)</td>
<td>14530a</td>
<td>11747b</td>
<td>15572b</td>
<td></td>
</tr>
<tr>
<td>$E_{dw}$ (N/mm$^2$)</td>
<td>12616a</td>
<td>10398b</td>
<td>10993b</td>
<td></td>
</tr>
<tr>
<td>$f_m$ (N/mm$^2$)</td>
<td>116.7a</td>
<td>80.1b</td>
<td>68.1b</td>
<td></td>
</tr>
<tr>
<td>$f_{0}$ (N/mm$^2$)$^*$</td>
<td>51.1a</td>
<td>47.6b</td>
<td>46.9b</td>
<td></td>
</tr>
</tbody>
</table>

Note: RF: resorcinol- formaldehyde resin; PUR: polyurethane resin.

### Table 5: Bending strength of some EWP made from tropical hardwood [9-13].

<table>
<thead>
<tr>
<th>EWP-wood species-adhesive</th>
<th>$f_M$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-beam-Cariniana micrantha-RF</td>
<td>46.3</td>
</tr>
<tr>
<td>I-beam-S. amara-RF</td>
<td>29.8</td>
</tr>
<tr>
<td>I-beam-S. amara-PU</td>
<td>32.1</td>
</tr>
<tr>
<td>I-beam-M. venulosa-PU</td>
<td>22.6</td>
</tr>
<tr>
<td>I-beam-S. rubra-RF</td>
<td>22.8</td>
</tr>
<tr>
<td>Glulam-S. rubra-RF</td>
<td>64.8</td>
</tr>
<tr>
<td>LVL-Schizolobium parahyba-RF</td>
<td>61.5</td>
</tr>
<tr>
<td>LSL- Chrysophyllum sp.</td>
<td>74.3</td>
</tr>
<tr>
<td>OSL- Chrysophyllum sp.</td>
<td>85.5</td>
</tr>
</tbody>
</table>

Note: RF: resorcincol- formaldehyde resin; PUR: polyurethane resin.

3 CONCLUSIONS

The results regarding the properties of wood bonding and wettability, non-destructive evaluation, laminated veneer lumber, glulam beams and I-beams from tropical hardwood species are discussed. In general, the potential of the tropical hardwood to manufacture EWP has been demonstrated and studies should be continued and deepen.

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REFERENCES


ABSTRACT: Understanding and detailing to accommodate differential movement that occurs during the construction and life of a building is necessary to ensure long-term structural safety, building serviceability and building envelope integrity. This paper summarized the existing knowledge on vertical movement in wood-frame construction from the following aspects: the fundamental causes of wood dimensional changes, major wood-based materials used for wood-frame construction and their shrinkage characteristics, movement amounts in publications based on limited field measurement, and movement estimations by practitioners in North America. Movement analysis and calculations were also demonstrated by focusing on wood shrinkage based on common engineering design assumptions, using six-storey platform buildings as examples. The paper also emphasized the importance of comprehensive analysis during design and construction to reduce and accommodate differential movement. Recommendations for future work including field measurement of movement and construction sequence optimization were also provided.

KEYWORDS: differential movement, wood-frame construction, shrinkage, creep, wood moisture content, loads

1 INTRODUCTION

Wood-frame residential construction is permitted to be built up to six storeys in the province of British Columbia, Canada, as of April 6, 2009. This is a 50% increase in height from the four storeys allowed by the current National Building Code of Canada. In building design and construction, differential movement caused by differences in movement between connected components, structural or non-structural, should be taken into consideration in order to ensure structural safety, serviceability and building envelope integrity. The consideration of differential movement becomes more important for taller buildings due to its cumulative nature and the movement amounts caused by the additional elements. Movement of wood materials is primarily related to moisture loss or gain. Thermal expansion does not cause significant dimensional changes of wood; however, it could be a major cause for the movement of other materials used in wood construction, such as steel elements and masonry cladding. Differential movement can occur where wood frame is connected to rigid components such as masonry cladding, concrete elevator shafts, mechanical services and plumbing; where mixed wood products such as lumber and engineered wood products are used; and even where connected wood elements are subjected to different environmental conditions. It is always important for designers to evaluate the differential movement amounts that may occur, and then provide appropriate design detailing where the differential movement may have impact on building performance.

2 TYPES OF MOVEMENT IN WOOD-FRAME CONSTRUCTION

2.1 MOVEMENT CAUSED BY MOISTURE CHANGES

Wood is a hygroscopic bio-polymer material. Moisture exists in wood either as "bound" water that is held within the cell walls or as "free" water that is stored in the cell cavities. As freshly cut, i.e. "green" wood dries, the free water evaporates first. Wood reaches fibre saturation point when all the free water is gone, leaving only the bound water within the cell walls. The fibre saturation point averages about 30% among different species [1], and 28% is commonly used for design-related calculations [2]. The practical importance of this concept is that it marks the turning point of a relationship curve between most physical and mechanical properties and moisture content (MC). For example, wood experiences little shrinkage until the MC is below the fibre saturation point; most strength indexes hardly change when the MC is above the fibre saturation point, but they usually increase with the decrease of MC below the fibre saturation point.
Wood exchanges moisture with surrounding air and the amount of moisture gain or loss depends on the relative humidity (RH) and temperature of the air, and the existing moisture amount in the wood. Wood achieves equilibrium moisture content (EMC) for certain environmental conditions when it no longer gains or loses moisture. Theoretically wood never reaches an EMC in service because the environmental conditions are always changing. However, because wood has a certain delay in responding to the environmental changes due to a known “sorption hysteresis” effect (shown as the difference between desorption and adsorption in Figure 1 [1]), the MC of wood will normally stabilize and fluctuate over a small range, and this is usually considered as EMC in service. On the West Coast of Canada, the EMC usually ranges from 8% to 12% for indoor wood members protected from liquid moisture [2].

Short-term exposure of lumber or timbers to rain or other water sources during storage, transportation or on construction sites usually only results in end grain absorption and surface wetting and will not cause considerable change of either the MC or the dimensions of the whole piece. Measures should be taken to prevent prolonged wetting incidents and ensure there is sufficient ventilation and drying during storage and on site.

Wood shrinks when it loses moisture and swells when it gains moisture. Shrinkage or swelling does not occur when the MC is above the fibre saturation point. The change in dimension is almost in direct proportion to its moisture change for the portion below the fibre saturation point (Figure 2). By nature wood is an anisotropic material. Shrinkage is different in the three principle directions in wood. The change in the length of lumber or timbers (along the grain) is small and can usually be ignored, except where there are severe grain irregularities, or the lumber has been sawn at an angle to the grain, or when the structure is very sensitive to any dimensional movement, or when the overall wood length is too large to be neglected such as in a tall building (such as six-storey buildings). Shrinkage is greatest in the circumference of a log (perpendicular to grain but tangential to the growth rings) compared to the radial direction (perpendicular to the grain but radial to the growth rings). For example, the average tangential shrinkage of spruce from the fibre saturation point to the oven-dried state is about 7-8%, the average radial shrinkage is about 4%, and the average longitudinal shrinkage is only 0.1-0.2% over the same MC change range [2]. Therefore only the cross sections (horizontal members) on a load path are usually taken into account during shrinkage calculation. Figure 2 shows the typical shrinkage values of wood in the three grain orientations, expressed as percentages of the green dimensions [2]. In service wood never experiences drying from “green” to “oven-dry”, the related shrinkage amounts are therefore usually much smaller.

The shrinkage of wood varies between and within wood species and even within the same tree. Density, grain orientations and microscopic structures such as detailed cell wall structures as well as chemical components all have an influence on the shrinkage amount. Normally a user has no control on species due to the mixed species of commodities, in North America in particular, or ring orientations which are mainly decided by tree growth and lumber manufacturing. Moreover, it is not possible or necessary to calculate shrinkage in design so precisely due to the many other variations involved in construction. Therefore, it is recommended that for transverse grain shrinkage, a rate of 0.2% per 1% change in MC be used for multi-story wood frame design [2,3,4,5]. The American Softwood Lumber Standard [6] recommends using an average shrinkage coefficient of 1% per each 4% drop in MC, i.e. 0.25% per 1% change in MC for cross sections of most softwood lumber. The precision of such composite shrinkage coefficients should be validated when field movement data are available.

### 2.2 MOVEMENT FROM INDUCED STRESSES

Load-induced deformation is another important part of dimensional movement for wood construction. Wood has a high modulus of elasticity (MOE) along the longitudinal axis, parallel to the grain, the consequent deformations from vertical components such as studs are therefore very small and can usually be neglected. It was found that even for a multi-storey building, the total movement is small compared to the shrinkage movement.
shortening of the studs accounted for only 1 mm at the eave level, but wood members loaded perpendicular to grain may undergo appreciable instantaneous and time-dependent deformations under load [7]. The transverse compression behaviour of wood depends on load, wood species, grain orientation, density, and MC. Under low short-term static compression, wood undergoes “elastic” deformation, which is mostly recoverable when the load is removed. But once the stress is above the proportional limit (this may not be the same as elastic limit), wood may experience substantial deformation, which is mostly unrecoverable when the load is removed, i.e. so-called plastic deformation. MC is also a critical factor affecting transverse compression behaviour in service.

As the time-dependent component of deformation, creep occurs under long-term compression and is greatly accelerated by high load, high MC and large MC fluctuations. For platform-frame construction, it was found from the TF2000 Project conducted in the UK that the majority of movement was caused by wood shrinkage and elastic compression, with approximately 39% attributable to bedding-in (settlement) and creep, and the majority of the wood movement was predicted to take place during the first 3 years of occupation [7,8]. Lund University of Sweden also conducted a study on movement monitoring using a five-storey wood-frame building built with modified balloon floor joists. The field monitoring of movement over the height of the whole building started upon the completion of the top floor. To completely finish the construction took another two months, during which period the construction load was increasing and the wood was allowed to dry. The results showed that the majority of the movement occurred in those two months and little movement was recorded after that in the total five months’ field measurement. Meanwhile, the MC of wood decreased from about 18% to 12% during this period [9,10].

Due to the variability and complexity to quantify creep, it is not realistic to precisely calculate and predict creep amount in building design. It was recommended that the amount of deformation due to creep be considered equal to the elastic compression movement for the total dead and imposed loads on the building [7], which was believed to be later integrated into the Euro Code. While the calculation of elastic and creep deformation is mandatory in Europe, it is not required in North America and therefore not taken into consideration in the movement calculations in this paper. But more research is certainly required to further determine the impact of instantaneous and time-dependent deformation on construction of mid-rise wood-frame construction.

2.3 SETTLEMENT (OR BEDDING-IN MOVEMENT)

Due to imperfections of product manufacturing and building construction, small gaps between framing members in the walls and floors may be created during building construction. But as the construction proceeds, these gaps are usually gradually reduced with the increase in load. In general, it is accepted that once the roof has been assembled, such settlement will be complete and does not cause long-term problems as long as the construction practice allows some level of vertical movement.

3 MOVEMENT AMOUNTS

3.1 WOOD MATERIALS USED FOR CONSTRUCTION

In North America, dimension lumber is produced with the designation of either “S-Dry” (Surfaced Dry) or “S-Grn” (Surfaced Green). Lumber with designation “S-Dry” means its MC is 19% or lower when it is planed or surfaced to the standard lumber dimension. “S-Grn” is not checked for MC at the time of surfacing and the MC is usually above 19% when surfaced. The designation of lumber does not provide any assurance that the lumber MC will not change after manufacturing. In most cases, both “S-Dry” and “S-Grn” lumber usually continue to lose moisture during storage, transportation and construction as the wood is kept away from liquid water sources and adapts to different atmospheric conditions. But the MC could increase if the lumber is subjected to considerable wetting. Field survey of MC of lumber used in construction based on typical construction practices in early 1990s in Canada confirmed both of these trends of MC changes [11,12]. For the purpose of shrinkage calculation, it is usually customary to assume an initial MC of 30% for “S-Grn” lumber and 19% for “S-Dry” lumber (Part 4.2). However, the real MC of wood products before building assembling and enclosure varies with manufacturing, handling and construction details, and should be further investigated in the field.

Different from solid wood products, Structural Composite Lumber (SCL), or called engineered wood products, is usually manufactured with MC levels close to or even lower than the EMC in service. Plywood, Oriented Strand Board (OSB), Laminated Veneer Lumber (LVL), and Parallel Strand Lumber (PSL) are usually manufactured at MC levels ranging from 6% to 12%. Engineered wood I-joists are made using kiln dried lumber or composite lumber flanges and plywood or oriented strandboard webs, therefore they are usually relatively dry and more dimensionally stable than lumber floor joists. Glued-laminated timbers (Glulams) are manufactured at MC levels from 11% to 15% [2], so are the recently-developed cross-laminated timbers (CLT). For all these products, minimal shrinkage can be expected and sometimes small amounts of swelling can be expected if their MC at manufacturing is lower than the service EMC. In order to fully benefit from using these dried products including “S-Dry” lumber and SCL products, care must be taken to prevent them from wetting during shipment, storage and construction.

3.2 CALCULATION OF SHRINKAGE

Wood shrinkage can be estimated using the following equation:

\[ S = D \times M \times C \]

Where:

- \( S \): Shrinkage
- \( D \): Total dead load
- \( M \): Maximum MC change
- \( C \): Creep constant
S = shrinkage amount (mm)
D = actual dimension (thickness or width, mm)
M = percentage of MC change below the fibre saturation point
C = shrinkage coefficient

The dimension to be used in the above equation is determined by the total cross sections of wood members on a load path, i.e. the plates and floor joists. The number of plates used is mainly decided by structural and acoustical requirements and construction practices. For the first floor, one bottom plate (or sill plate) can be used if there is no subsequent application of concrete screen or topping. The use of a preservative treated sill plate depends on local climate and termite hazard conditions and may not be required by the building code. However, it is assumed in the calculations that a pressure treated 38 x 140 mm (2x6”) dimension lumber with an initial MC of 30% is used between the bottom plate and the concrete foundation. In most cases, sill plates in direct contact with concrete slabs have MC higher than other elements anyways. For the upper storeys, two bottom plates are assumed for buildings with concrete topping (Concrete topping is used to improve acoustical performance and accommodate radiant floor heating in multi-floor buildings), and two top plates are assumed for each storey. Floor joists are made with either lumber or SCL in the calculations. In terms of the percentage of MC changes of wood, assumptions are made in the calculations that “S-Dry” lumber has an initial MC of 19%, since “S-Grn” lumber is rarely used for multi-family wood buildings in Canada, and any MC higher than 19% before building enclosure is not allowed by the building code. An initial MC of 10% is assumed for floor joists made of SCL. For all framing members, it is assumed that the final MC reached in service is 8%. A composite shrinkage coefficient of 0.25% per 1% change in MC is used for all cross sections, and a shrinkage coefficient of 0.0053% per 1% change in MC is used for studs in order to account for the tall building effect (assuming a longitudinal shrinkage rate of 0.15% from green to oven-dry state, [2]).

The results listed in Tables 1 indicate that the shrinkage amounts are much lower when SCL is used as floor joists instead of using lumber as floor joists. In addition, the calculations here also show that there are a lot of uncertainties in determining all required numbers used for shrinkage calculations and there is a lack of hard data related to building shrinkage. It would be helpful to survey the average MC of wood products used before assembling, before building enclosure and during building service life based on the current typical construction practices, and monitor building movement during construction and service. This has become particularly important for mid-rise platform frame buildings in order to provide better vertical movement-related information for the related building design.

### Table 1: Estimated shrinkage of a six-storey wood-frame building, mm

<table>
<thead>
<tr>
<th>Level</th>
<th>Case 1 (S-Dry joists, S-Dry plates)</th>
<th>Case 2 (SCL joists, S-Dry plates)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shrinkage per floor</td>
<td>Accumulative shrinkage*</td>
</tr>
<tr>
<td>Roof eave</td>
<td>74</td>
<td>42</td>
</tr>
<tr>
<td>6th floor</td>
<td>6</td>
<td>68</td>
</tr>
<tr>
<td>5th floor</td>
<td>13</td>
<td>55</td>
</tr>
<tr>
<td>4th floor</td>
<td>13</td>
<td>41</td>
</tr>
<tr>
<td>3rd floor</td>
<td>13</td>
<td>28</td>
</tr>
<tr>
<td>2nd floor</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>1st floor</td>
<td>14</td>
<td>8</td>
</tr>
</tbody>
</table>

* At the upper surface of the floor joist

### 3.3 ESTIMATED MOVEMENT AMOUNTS IN THE LITERATURE

Movement amounts in publications were summarized based on the very limited field measurement in wood-frame buildings (Table 2). The data varied greatly with wood MC, products and building designs. It appeared that the vertical movement amount was about 8 or 9 mm when “S-Dry” lumber was used.

### Table 2: Estimated movement amounts per storey based on field monitoring

<table>
<thead>
<tr>
<th>Sources</th>
<th>Estimated movement amount per storey</th>
</tr>
</thead>
<tbody>
<tr>
<td>TF2000, S-Dry lumber installed at 20% MC</td>
<td>9 mm</td>
</tr>
<tr>
<td>TF2000, super dry lumber installed at 12% MC or lower</td>
<td>4 mm</td>
</tr>
<tr>
<td>TF2000, engineered wood products installed at 12% or lower</td>
<td>2.5 mm</td>
</tr>
<tr>
<td>Land University, modified balloon floor joists using S-Dry lumber [9]</td>
<td>4 mm (4% of cross section)</td>
</tr>
<tr>
<td>Land University, prediction for platform frame with S-Dry lumber [9]</td>
<td>8 mm</td>
</tr>
<tr>
<td>Murray Grove building built with CLT in London [13]</td>
<td>3 mm</td>
</tr>
<tr>
<td>Limnologen project built with CLT in Sweden [14]</td>
<td>3 mm</td>
</tr>
</tbody>
</table>

A small survey was also conducted in the building and construction industry about the movement amounts of wood-frame construction. A number of anecdotal movement numbers were provided by design and construction professionals based on their construction experience. All the estimates vary, and certainly building designs and construction practices also have effect. It was estimated that the vertical movement amounts in wood-frame construction would typically range from 12 to 18 mm per floor. The survey found that there was a large demand in the construction industry for measurement of vertical movement in real buildings. It
was hoped that hard data from field will also help validate and improve future shrinkage estimation.

4 CONCLUDING REMARKS

Movement is intrinsic to building materials and the movement of wood is mainly caused by moisture loss and gain when the MC is below the fibre saturation point. Differential movement between connected components may compromise building performance and requires special attention in design and construction. It is of importance for designers to analyze a design in advance, estimate the differential movement to be anticipated, and then provide appropriate design and construction detailing. This becomes critically important for five- and six-storey wood-frame buildings due to the cumulative effect of differential movement. Vertical movement in wood-frame construction can certainly be reduced by specifying materials with lower shrinkage amounts, such as engineered wood products and lumber drier than “S-Dry”, if the increased costs can be justified. However, simply specifying materials with lower moisture content at time of delivery to job site does not guarantee that wood will not get wet during construction. On the other hand, effective drying can occur before the lumber is closed into building assemblies. To conclude, appropriate measures should be taken during construction to protect wood products against wetting, protect prefabricated panels on site, and use good construction sequence to facilitate air drying and accommodate potential differential movement. The use of supplementary heating can effectively accelerate wood drying if wood gets severely wet during construction, and thereby reduce the movement amounts after the completion of the construction.

5 RESEARCH GAPS AND RECOMMENDATIONS

The shrinkage of individual wood species in the three principle directions has been tested extensively. However, wood movement under realistic MC change and loading conditions still remains to be clarified. There still are many uncertainties involved in movement estimations for wood alone, and more data are required in order to assist designers in providing detailing to accommodate differential movement for wood-frame construction. It is recommended that the future investigations should focus on the following areas:

- Field movement monitoring from construction to occupation of buildings with typical structures, materials and design loads, five- and six-storey wood buildings in particular.
- Survey the common construction sequencing used in the construction industry and develop optimized construction procedures in order to minimize wood wetting and maximize drying during construction.
- Quantify movement differences between “S-Dry”, “S-Grm” lumber and engineered wood products such as Glulam, parallam, LVL and CLT under typical loading and MC change conditions in the laboratory, in couple with field movement measurement.
- Investigate the effect of rain exposure on the MC and dimensions of wood products.

REFERENCES

EXPERIMENTAL STUDY ON THE BOND AND ANCHORAGE BEHAVIORS OF GLUED-IN ROD JOINTS IN GLULAM

Hui-Feng Yang, Wei-Qing Liu, Wei-Dong Lu, Ju Tang

ABSTRACT: This paper provides a description of the testing, failure modes and mechanism analysis, and bond-anchorage behaviour of rod bonded in glulam (glued laminated timber) parallel to the grain. A total of 45 pull-out specimens were tested to failure using a uniform loading pattern. The effects of anchorage length, rod diameter, glue-line thickness and rod surface conditions on pull out strength and bond-anchorage properties were evaluated. The test results showed that pull-out strength of the specimens are mainly relative to anchorage length and rod surface conditions, while bond shear stiffness mainly relative to rod diameter and glue-line thickness. Finally, some preliminary conclusions have been presented and further study subjects are indicated.

KEYWORDS: Glued-in rod joints, Timber structure, Bond-anchorage behaviours, Pull-out strength, Experiment study

1 INTRODUCTION

Steel rods bonded in glulam elements through epoxy have an efficient performance. The steel rods and wood member are bonded together through the adhesive, which can transfer the loads between the steel rods and wood member. And it can be widely used into the field of timber connections, timber member reinforcements or strengthening for heavy timber construction. The joints made with glued-in bars are an option of great interest for the design of timber structures. While compared to the traditional joint systems, Glued-in rod connections have the advantage of improved aesthetics associated with the hidden design of these joints, high local force transfer, higher joint stiffness, easy installation, low material and production costs and higher fire properties as the rod is embedded and protected by the timber [1,2].

Research on glued-in rods started in the late eighties of the last century [3,4]. Attempts to develop design methods that would optimize the application of such joints were intensified within the last 10 years. Over the last 20 years, there has been an increasing interest in the use of steel threaded rod bonded into glulam timber members in order to fabricate moment-resisting joints. The use of glued in steel rod technology for timber connections began in Denmark about 1980. Since then many studies have been carried out in Europe to investigate the reliability of this jointing method and a number of formulae have been presented to aid engineers in designing this type of timber connection. In 1990s, a research program named of GIROD, which was taken on by Swedish National Testing and Research Institute, was carried out to study the glued in rods in timber members. And a number of conclusions had been drawn in this program.

In the following years, many studies have been carried out in Europe to investigate the reliability of this jointing method. The research has focused on the bond strength and its influencing factor, such as adhesive type, glue line thickness, anchorage length, rod diameter and wood density [5–10].

Glued in rod timber connection is a new type timber joint system for China, and it is necessary to carry out some basic research work with more and more engineering applications for heavy timber constructions. The objective of this study was to study the pull-out strength of glued-in rod joints and its influencing factors.

2 TEST SPECIMENS AND PROCEDURES

2.1 MATERIALS AND FABRICATION DETAILS

Timber specie is North America Douglas fir and it was glued together with PUR adhesives. The average physical and mechanical properties as obtained from
preliminary testing of glulam samples are reported in 未找到引用源。.

**Table 1: Physical and mechanical properties of glulam specimens (EN 408-2006)**

<table>
<thead>
<tr>
<th>Item</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture of content (%)</td>
<td>10.5</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>475</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>49.7</td>
</tr>
<tr>
<td>Tensive strength (MPa)</td>
<td>42.2</td>
</tr>
<tr>
<td>Modulas of elasticity parallel to grain (MPa)</td>
<td>9870</td>
</tr>
</tbody>
</table>

**Table 2: Mechanical properties of grade HRB335 deformed rebar (GB/T 228-2002)**

<table>
<thead>
<tr>
<th>Item</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength (MPa)</td>
<td>350</td>
</tr>
<tr>
<td>ultimate strength (MPa)</td>
<td>571</td>
</tr>
<tr>
<td>Modulas of elasticity (MPa)</td>
<td>2.06×10⁶</td>
</tr>
</tbody>
</table>

**Table 3: Mechanical properties of epoxy adhesives certified by manufacturer (GB/T 2567-2008)**

<table>
<thead>
<tr>
<th>Item</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Splitting tensile strength (MPa)</td>
<td>14.5</td>
</tr>
<tr>
<td>Bending strength (MPa)</td>
<td>92.7</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>84.3</td>
</tr>
</tbody>
</table>

The mechanical properties (mean value) of deformed rebar and the epoxy adhesives are reported in Table 2 and Table 3, respectively.

2.2 TEST SPECIMENS

A total of 45 specimens were tested to failure under monotonic loading. And the double-sided pull-out test specimens (see Figure 1) have different anchorage lengths, rebar diameter, glue line thickness and rebar surface conditions according to the test variables (see Table 4). The cross-section width of the different glulam specimens were 10 times of the rod diameter. And the clear distance between the bars ends is 1.4 times of the anchorage length. The test specimens were longitudinally drilled centrally of the section at both ends. Then the drilled hole was partially filled with the epoxy adhesive with a mixing gun. The elastic Silicone plug was used at the end of the drilled holes to prevent leaning or displacement of steel rods. The steel rod was then twisted manually into the drilled hole in order to expel the air. The adhesive was left to cure for at least 24 hours according to the manufacture’s recommendations.

**Table 4: Parameters of test specimens**

<table>
<thead>
<tr>
<th>Type</th>
<th>Rod diameter</th>
<th>Anchorage length</th>
<th>Glue line thickness t</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>RB08L10T4</td>
<td>8</td>
<td>100</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>RB12L09T4</td>
<td>12</td>
<td>90</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>RB12L12T4</td>
<td>12</td>
<td>120</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>RB12L15T2</td>
<td>12</td>
<td>150</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>RB12L15T4</td>
<td>12</td>
<td>150</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>PB12L15T4</td>
<td>12</td>
<td>150</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>RB12L15T6</td>
<td>12</td>
<td>150</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>RB12L18T4</td>
<td>12</td>
<td>180</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>RB16L20T4</td>
<td>16</td>
<td>200</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

2.3 TEST SETUP AND INSTRUMENTATION

In this paper, double-sided pull-out test of glulam specimens were performed. As is shown in Figure 2, the specimens were tested on double-side-tensile device. The relative displacement between the loaded-end of the rod and the glulam block was monitored by two LVDT, which were connected to an electronic data acquisition system.

2.4 LOADING SCHEME

The tests were conducted in displacement-control mode with the rate of displacement was 2mm/min. And the failure value was reached within 10 minutes.
3 TEST RESULTS AND DISCUSSION

In this section, the failure modes, the effects of anchorage length, rod diameter, glueline thickness and rod surface conditions on bond strength and stiffness were studied.

3.1 FAILURE MODES

In the whole test specimen, the rod and timber piece are subjected to the tensile load, while the glueline, as well as rod-adhesive and adhesive-timber surfaces are under shear stress.

The failure modes of this type of joints mainly include: timber shear failure, rod yielding and adhesive shear failure, as shown in Figure 3.

![Figure 2: Test setup and instrumentation](image)

![Figure 2: Test setup and instrumentation](image)

![Figure 3: Failure modes of the test specimens](image)

![Figure 3: Failure modes of the test specimens](image)

3.2 COMPARISON ON LOAD-SLIP CURVES OF DIFFERENT SPECIMENS

It is shown in Figure 4 that pull-out strength are mainly relative to anchorage length and rod surface conditions, while bond shear stiffness mainly relative to rod diameter and glue-line thickness.

Timber failure due to the shear stress is a brittle behavior. And this can be avoided by using steel rods with suitable anchorage length, in which shows a ductile failure mode, as shown in Figure 4(a).

![Figure 4: Load-slip curves of the test specimens](image)

![Figure 4: Load-slip curves of the test specimens](image)

3.3 BOND STRENGTH

The comparisons of ultimate pull-out load and bond strength have been put forward. And as shown in Table 5 and Figure 5 (The $P_u$ and $\tau_m$ data were the average value of a group). It can be seen that the ultimate pull-out load will increase when the anchorage length rises, until the
anchorage length reaches 150mm or 12.5d, in which d is the rod diameter. While the bond strength shows an increasing firstly and decreasing when the anchorage length increases from 120mm to 180mm.

**Table 5: Parameters of test specimens**

<table>
<thead>
<tr>
<th>Type</th>
<th>Rod diameter d/mm</th>
<th>Anchorage length l_a/mm</th>
<th>Glue line thickness t/mm</th>
<th>Ultimate load P_u/kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>RB12L09T4</td>
<td>12</td>
<td>90</td>
<td>4</td>
<td>16.1</td>
</tr>
<tr>
<td>RB12L12T4</td>
<td>12</td>
<td>120</td>
<td>4</td>
<td>48.3</td>
</tr>
<tr>
<td>RB12L15T4</td>
<td>12</td>
<td>150</td>
<td>4</td>
<td>56.2</td>
</tr>
<tr>
<td>PB12L15T4</td>
<td>12</td>
<td>150</td>
<td>4</td>
<td>35.9</td>
</tr>
<tr>
<td>RB12L18T4</td>
<td>12</td>
<td>180</td>
<td>4</td>
<td>56.9</td>
</tr>
</tbody>
</table>

It can be seen from Table 6 that the bond stiffness improves with the increasing of the rod diameter and decreasing of the glueline thickness, if given the same \( l_a/d \) ratio.

**Table 6: Parameters of test specimens**

<table>
<thead>
<tr>
<th>Type</th>
<th>Rod diameter d/mm</th>
<th>Anchorage length l_a/mm</th>
<th>Glue line thickness t/mm</th>
<th>Bond stiffness / N/mm²/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>RB08L10T4</td>
<td>8</td>
<td>100</td>
<td>4</td>
<td>4.75</td>
</tr>
<tr>
<td>RB12L15T4</td>
<td>12</td>
<td>150</td>
<td>4</td>
<td>11.8</td>
</tr>
<tr>
<td>RB16L20T4</td>
<td>16</td>
<td>200</td>
<td>4</td>
<td>15.3</td>
</tr>
<tr>
<td>RB12L15T2</td>
<td>12</td>
<td>150</td>
<td>2</td>
<td>14.9</td>
</tr>
<tr>
<td>RB12L15T6</td>
<td>12</td>
<td>150</td>
<td>6</td>
<td>8.16</td>
</tr>
</tbody>
</table>

### 4 CONCLUSIONS

Glued-in rod joints can be widely used into the field of timber connections, timber member reinforcements or strengthening for heavy timber construction. Pull-out strength of glued-in rod joints are mainly relative to anchorage length and rod surface conditions, while bond shear stiffness mainly relative to rod diameter and glue-line thickness. Timber failure due to the shear stress is a brittle behavior. And this can be avoided by using steel rods with suitable anchorage length, in which shows a ductile failure mode.

### ACKNOWLEDGEMENT

This research was supported by National Natural Science Foundation of China/NSFC (Grant No. 51108233). The support of NSFC is gratefully acknowledged.

### REFERENCES


EFFECT OF THE PRESSURE ON THE GLUE-LINE SHEAR STRENGTH OF TWO TROPICAL HARDWOODS

Emanuella S. R. Silva¹, Cláudio H. S. Del Menezzi², Mário R. Souza³

ABSTRACT: The paper aimed at evaluating the effect of assembly pressure on the glue-shear strength of two well-known tropical hardwoods: cedroarana (Cedrelinga catenaeformis) and tuturubá (Pouteria guianenisis). They are respectively medium (0.62 g/cm³) and high (0.98 g/cm³) density wood species. Commercial resorcinol-formaldehyde (RF) adhesive was applied at spread rate of 350g/m² and the following pressures were evaluated: 0.4, 0.6, 0.8, 1.0 and 1.2 N/mm². For each species/pressure combination 12 specimens were tested totaling 120 specimens. According to the results the glue-line shear strength for the cedrelinga wood was affected by the pressure improvement. The better result (13.0 N/mm²) was obtained when the highest level of pressure (1.2 N/mm²) was applied. The opposite behavior was observed for the tuturubá wood: lower the pressure (0.4 N/mm²), higher the glue-line shear strength (17.1 N/mm²). Both wood species presented glue-line shear strength values higher than the solid wood shear strength. Nevertheless, the wood failure values were relatively low: 50.4% for cedroarana; and 35.3% for tuturubá. Finally, it could be concluded that for denser wood species can be benefited from applying low pressure, while lighter wood might require higher pressure to produce stronger bonding.

KEYWORDS: tropical hardwood, engineered wood product.

1 INTRODUCTION

The glued laminated lumber (glulam) beam is a kind of engineered wood widely used in wood construction. The utilization of tropical hardwood to manufacture EWP is rare, or even it does not happen, and most of the production is based on temperate softwood and hardwood. Despite of this, it is believed that tropical hardwood has a huge potential for manufacturing EWP. Since a suitable structural adhesive is required for each class of EWP, it is very important to evaluate the behaviour of the wood glued joint. Nevertheless, information about strength of glued joint from tropical hardwood is still rare. It is well-known that the strength of wood is highly affected by the density, which also affects the wood glueability. Therefore, low density wood can develop starved glue-line, since the glue can flow fully into wood because of the high porosity. On the other hand, high density wood show an opposite behaviour, and insufficient glue penetration can happen. In both situations the strength of the glued member is severely affected and the bearing capacity can be drastically reduced. The pressure applied during the assembling of glued joint is a key processing factor. It plays an important role helping the wood wetting, the adhesive penetration, the reduction of glue-line air-bubbles and others factors.

In this context, the paper aimed at evaluating the effect of assembly pressure on the glue-shear strength of two well-known Brazilian tropical hardwoods: cedroarana (Cedrelinga catenaeformis) and tuturubá (Pouteria guianenisis).

2 MATERIALS AND METHODS

Two wood species were chosen based on the density: cedroarana (Cedrelinga catenaeformis) and tuturubá (Pouteria guianenisis). The first is a medium density (0.62 g/cm³) while the second is a high density wood species (0.98 g/cm³). The wood material was macroscopically identified by comparing with the material deposited at Wood Anatomy Sector of the Forest Products Laboratory (Index Xylarium FPBw), Brazilian Forest Service. The lumber was cut, and pieces measuring 30 mm x 60 mm x 750 mm (t x w x l) were produced and kept in the conditioning room (20°C; 65% RH) until reaching constant weight. Resorcinol-formaldehyde (RF) was
used in this research and was prepared mixing 5 parts in weight of resorcinol and 1 part in weight of formaldehyde. The adhesive spread rate was about 350g/m². Five assembly pressure levels were evaluated per specie: 0.4 N/mm², 0.6 N/mm², 0.8 N/mm², 1.0 N/mm² and 1.2 N/mm². The pressure was applied using a device composed by steel I-beams and long threaded bolts, while the pressure level was adjusted using torque meter and load cell (Figure 1).

After assembling the pieces were cut according to ASTM D905 (2000) to evaluate the glue-line shear strength \(f_{gv,0}\) (Figure 2). For each species-pressure combination 12 specimens were cut and tested, which meant 120 specimens tested. The \(f_{gv,0}\) values were corrected for a 12% moisture content base, according to the equation 1. After testing the percentage of wood failure was evaluated and the bonding efficiency \((Ef)\) - relationship between \(f_{gv,0}\) and solid wood shear strength \(f_{v,0}\) - was calculated according to the equation 2.

\[
f_{gv,0\%} = f_{sv,0\%} \times \left[1 + \frac{3 \times (\mu\% - 12\%)}{100}\right]
\]

\[
Ef(\%) = \left[\frac{f_{gv,0\%}}{f_{v,0}}\right] \times 100
\]

where: \(f_{gv,0\%}\): strength at 12%; \(f_{sv,0\%}\): strength at testing; \(\mu\%\): specimen moisture content; \(f_{v,0}\): solid wood shear strength

Analysis of variance (ANOVA) and Tukey HSD test were run in order to identify the effect of the pressure on the \(f_{gv,0}\). Analysis of variance (ANOVA) and Tukey HSD test were run in order to identify the effect of the pressure on the \(f_{gv,0}\).

3 RESULTS AND DISCUSSION

Figure 3 presents data distribution of the glue-line shear strength of both wood species. It can be observed that strength is higher when higher pressures assemblies are used for \(C. catanæiformis\). The opposite behaviour can be seen for \(P. oblanceolata\). These results can be better observed in Figure 4 which shows the results of \(f_{gv,0}\) according to assembly pressure for each species. It can be observed that \(f_{gv,0}\) of \(C. catanæiformis\) wood was affect only when highest pressure was applied. Therefore, up to 1.0 N/mm²
pressure level the \( f_{gv,0} \) values were statistically the same, but at 1.2 N/mm\(^2\) the value was improved statistically. For all pressure levels the bonding efficiency was higher than 100% (101.3% to 128.0%), which means \( f_{gv,0} \) higher than solid wood.

<table>
<thead>
<tr>
<th>Pressure (N/mm(^2))</th>
<th>( f_{gv,0} ) C. cataneaformis</th>
<th>( f_{gv,0} ) P. oblaneolata</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>53.1</td>
<td>25.6</td>
</tr>
<tr>
<td>0.6</td>
<td>46.3</td>
<td>56.7</td>
</tr>
<tr>
<td>0.8</td>
<td>55.0</td>
<td>28.8</td>
</tr>
<tr>
<td>1.0</td>
<td>47.1</td>
<td>47.3</td>
</tr>
<tr>
<td>1.2</td>
<td><strong>50.4</strong></td>
<td>18.0</td>
</tr>
<tr>
<td>Mean</td>
<td>50.4</td>
<td>35.3</td>
</tr>
</tbody>
</table>

Table 1 shows the percentage of wood failure according to pressure level. For both wood species wood failure values can be considered low, less than 50%. The result was better for lighter wood \( C. cataneaformis \) and no trend was identified according to the pressure level. It can be observed that the better pressure level (1.2 N/mm\(^2\)) for \( C. cataneaformis \) did not imply higher wood failure (50.4%). Nevertheless, this behaviour was much more evident in \( P. oblaneolata \), whose wood failure value achieved only 25.6% for the better pressure level found (0.4 N/mm\(^2\)). Figure 5 depicts the appearance of the surface of the rupture for both studied species.

For \( P. oblaneolata \) wood, the \( f_{gv,0} \) presented the highest value when the lowest pressure was applied (0.4 N/mm\(^2\)). From 0.6 N/mm\(^2\), \( f_{gv,0} \) presented decreasing values although they were not statistically significant. The bonding efficiency ranged from 81.6 to 110.3, which means was quite different in comparison with \( C. cataneaformis \). This way, only the lowest pressure level showed value higher than 100%. According to these results, it can be inferred that the lighter wood species requires higher assembly pressure in order to yield higher glue-line shear strength.

Contrarily, the higher wood density species is benefited from lower assembly pressure. Undoubtedly, these are unexpected results. However, it was observed that when higher pressure was applied for assembling \( P. oblaneolata \) (0.98 g/cm\(^3\)) joints the resin flowed to the edge instead of penetrating the wood surface. It happened because of the low porosity of wood, which imparted low penetration of this viscous (2500 cP) resin. Since \( C. cataneaformis \) has lower density (0.62 g/cm\(^3\)), therefore high porosity, the resin could easily flow into wood surface, and pressure assembly did not affect \( f_{gv,0} \) up to 1.0 N/mm\(^2\).

The wood density is the most important factor affecting the strength of glue-line. For this species the benefit of improving pressure might come from the ability of providing closer contact between wood surfaces, reducing voids and gaps on the glue-line. Other tropical hardwoods have been studied [1-3]. It has been observed that \( Erisma \) sp (0.60 g/cm\(^3\)) and \( Moquina polymorpha \) (0.63 g/cm\(^3\)) require higher assembly pressure (1.6 N/mm\(^2\)) than \( Bertholletia excelsa \) (0.70 g/cm\(^3\)), 0.8 N/mm\(^2\). On the other hand, \( Couratari guianensis \) (0.52 g/cm\(^3\)), \( Parkia oppositifolia \) (0.50 g/cm\(^3\)) and \( Simarouba amara \) (0.44 g/cm\(^3\)) have required the same pressure level: 0.8 N/mm\(^2\).

**Figure 4:** Mean value, standard deviation and Tukey test of \( f_{gv,0} \) according to the pressure assembly (Dash line: solid wood shear strength).

**Figure 5:** General overview of the rupture surface of \( C. cataneaformis \) (above) and \( P. guianenisis \) (below).
4 CONCLUSIONS

Both wood species presented glue-line shear strength values higher than the solid wood shear strength. The wood failure was relatively low. It could be concluded that for denser wood species is benefited from applying low pressure, while lighter wood requires higher pressure to produce stronger bonding.

ACKNOWLEDGEMENT

To CNPq and CAPES through personal and project research grants and to FINATEC for providing travel funds which have made it possible to attend this conference.

REFERENCES


TIMBER TRUSS BRIDGE FOR CASCADE USE

Hideyuki Hirasawa¹, Tomoya Yoshida², Jun Tonuma³, Tetsuya Sato⁴ and Hiroshi Watanabe⁵

ABSTRACT: A timber truss bridge for a temporary use such as an emergency bridge is developed. It is appropriate to use timber for disaster recovery because the timber construction is easy to work and takes not long time. The timber is also appropriate for the cascade use to bridge, erection materials, foot walk path, retaining walls, etc. which are civil engineering field use. The truss bridge developed here has a character of unified member form such as length, cross section shape, bolt hole location. This unification of truss members contributes to be timesaving for assembling. As a result of substantiation tests of a real bridge, it is found that to assemble into a bridge by human power using working tools was possible.

KEYWORDS: Timber truss bridge, Timber bridge, Cascade use, Emergency bridge

1 INTRODUCTION

Civil engineering structures such as bridges, erection materials, retaining walls, guard rails, etc. are mostly constructed from concrete and steel which exhaust CO₂ in producing them. If the wood is used as a material in the structures instead of concrete or steel, the contribution for the restraint of global warming will be expected. To use wood for civil engineering field should be pushed on because wood has not been used much for civil engineering structure yet[1, 2].

This study deals with a timber bridge as one of example of using wood for the civil engineering field. The bridge is the timber truss bridge which can be applied to temporary use such as an emergency bridge and to cascade use. The timber bridge with these new concept is proposed in this study.

In 2011, a big earthquake and a tsunami occurred in east Japan and many roads and bridges were broken. After that, it takes long time to restore the damaged infrastructure. However, it is necessary to be opened to traffic as soon as possible. It is important that the roads are opened to traffic even for temporary use. Long time during the construction of permanent roads and bridges can not be waited in the damaged area. The timber truss bridge developed in this study is appropriate to the emergency use such as this earthquake broken road net.

2 CASCADE USE OF TIMBER

There have been wood structures widely in the civil engineering field such as road structures, safety fences, river structures, foundation structures, etc. The required performance for wood is different from each use. In the cascade use, bridge members should be used in the first stage because they need high strength and durability. On the other hand, retaining walls may be good for the second stage because they do not need so high strength.

Figure 1: Using style of wood as usual
Figure 1 shows the use of wood as usual, that is, wood from forest is used to structures or energy only one time. However, in cascade use, wood is used to different structures cyclically as shown in Figure 2. The first use is for the structures requiring high strength, the next use for requiring not high strength. The last use may be for energy. The cascade use can effectively exploit the wood resource.

In addition to the material performance, whether easy or difficult construction in the next stage have to be considered. For that reason, it is important to develop the structural form in which the assembling and disassembling are easy, in the cascade use.

3 TIMBER TRUSS BRIDGE

3.1 NECESSARY CONDITIONS OF DESIGN AND CONSTRUCTION

The timber truss bridge developed in this study is considered for temporary use in restoring from disaster or in bridge erection. Necessary conditions for the temporary bridge are as follows:

- The strength of the whole bridge is not so high[3, 4].
- The durability is not high because of temporary use.
- Materials of the bridge can be obtained easily.
- Assemble and disassemble of the bridge is quick and easy[5].
- Cost of the bridge is low.

3.2 THE WHOLE STRUCTURE

The timber truss bridge is developed for pedestrian bridge, which has 10[m] span, as shown in Figure 3. The bridge was constructed in the college campus. The characteristics of this bridge is the following. (1) The size of the bridge member is only two kinds, one is 2.5[m] long for the truss and the slab member and the other is 1.2[m] long for the cross beams. This unified size is very convenient for the cascade use. (2) The construction is easy because the assembling of timber members is by the bolts with connection plates which have also unified size as well as timber member. Slab is also assembled easily by putting the end slit of timber to the cross beams plate as shown in Photo 1. (3) It is easy to obtain the timbers because the timber, 120 × 120 [mm] cross section, is being circulated. (4) This bridge can resist the distributed load of 3.5[kN/m²] which means full persons on the slab. These characteristics (2) and (3) are convenient for the temporary use.
3.3 TRUSS CONNECTION

The panel point of truss members is the bolt connection with inserted steel plates. Photo 2 shows this steel plates used in the bridge. The plate at the intermediate part does not have the difference between for upper chord and for lower chord. The steel plate is 4.5[mm] thick and has 6 bolt-holes per one truss member. The diameters are 23[mm] for the holes and 20[mm] for bolt shank.

The end of truss members has bolt holes and slit as shown in Photo 3. The diameter of bolt holes is 23[mm] and slit is 6[mm] wide. The distance between the holes is 140[mm] and end distance is 140[mm].

Photo 4 shows panel point of upper chord. The panel point is connected such that the axial forces of upper chords, diagonal members and a cross beams meet at one point. The inserted steel plate has also a plate parallel to floor slab. This plate can connect the lateral bracings with turn buckles. Stainless wire ropes with 6φ are used as lateral bracings.

4 CONSTRUCTION OF BRIDGE

4.1 SHOP ASSEMBLY

Before the construction on site, an assembling test as shown in Photo 5 was carried out in the college gym to confirm the penetration of bolts, the assembling time and something trouble. It is found that the assembling time and the pulling down time are four and a half hours and one hour, respectively, by ten workers. These short times are very convenient for temporary use, especially for emergency bridge under a disaster.

As some trouble points, the followings were obtained by the assembling test : (1)There were some difficult parts for penetrating bolts and inserting steel plates. (2)The slab was composed by 9 timber members in the right angle of bridge direction, though 10 members were placed in the original design. (3)It was easy to deform to the direction of the right angle of the bridge. (4)The heaviest connection plate was 17[kgf], which brought a little difficulty in working.

As for the trouble of (1), batting a pin to the bolt hole by hammer and batting the connection plate to the slit became their solution. And for (3), portals were to be attached in the construction on site.

4.2 CONSTRUCTION ON SITE

After the indoor test, the bridge was constructed in the college campus. Photo 6 shows the assembling a truss on the ground. In this time, total fifteen engineers and students were working in the construction. When the truss members are connected to each other, it was difficult to insert some steel plates to the slits at the end of timber members. In some works of fastening the bolts and nuts, there were difficulties to fix the bolts and nuts because the both of them turn. However, these working demerits are not big problem in constructing the bridge on site.

After assembling a two-dimensional truss structure, it was hung up by a crane and was put on the concrete abutment. Since the construction site is surrounded by school buildings, a crane erection had to be adopted as a safe construction method. Photo 7 shows erection...
working with civil engineering students and construction company workers. If there is not the building backward, launching method which is bridge construction method may be adopted. A small size crane is good for this erection because a two-dimensional assembled truss is less than 0.5[tf] weight. After both truss surfaces were put on the concrete abutment, upper and lower cross beams which have 1.2[m] length were attached and stainless wire ropes were put together as lateral bracings.

A small size crane is good for this erection because a two-dimensional assembled truss is less than 0.5[tf] weight. After both truss surfaces were put on the concrete abutment, upper and lower cross beams which have 1.2[m] length were attached and stainless wire ropes were put together as lateral bracings.

According to the result of the shop assembly, portals were set on the end diagonal members as shown in Photo 8. These portals are made of plywood. It took four hours and a half to the completion as the same at the indoor test.

Photo 8 : Timber truss bridge at its completion

5 CONCLUSIONS

A timber truss bridge appropriate to cascade use and to temporary use was developed and constructed practically. Although the bridge does not have preservative treatment because of temporary use, it is enough strong to carry the full pedestrian load. It takes not so long time to construct this bridge by human power because the construction working is easy. Also, unified size of timber can make easy to assemble the members into the bridge, and can be convenient for the next stage in cascade use.

In addition to cascade use, this bridge is appropriate to emergency use as earthquake disaster because an easiness of construction and a short time construction are achieved.

This study covers only two cases, one is an indoor erection test, the other is an on-site test. Many tests of assembling and disassembling cyclically should be carried out to inspect the easiness of construction working after a long time. During the storage of timber, from one stage to the next stage, distortion or twist of timber should be prevented.

REFERENCES

GRADING RESEARCH ON STRUCTURAL LARCH DIMENSION LUMBER

Wanli Lou, Haiqing Ren*, Zhaohui Wang, Haibin Zhou

ABSTRACT: In this study, we selected dahurian larch (Larix gmelinii) dimension lumber whose cross-section size was 40mm×90mm as the research object species. The dimension lumber was visually graded according to Chinese Standard GB50005-2003, and then full-size bending test was carried out to achieve the relationship between MOE and MOR, also grade boundaries were set by mechanical properties. The results showed: Major grade-reducing defects of larch dimension lumber were knots, shake/checks and decay. More than 40% Ⅰc grade lumbers could be got according to visual grading rules in GB50005-2003. With modulus of elasticity as indicator property, larch dimension lumber could be sort into three mechanical grades: M14, M30 and M40. The grade strength of larch dimension lumber in sequence was as: M40, Ⅰc, M30, Ⅱc, Ⅲc, M14. The fact that more than 80% higher grade lumber could be got by mechanical grading technology indicate that Chinese commercial forest of larch is more suitable for mechanical grading.

KEYWORDS: Dimension lumber, Visual grading, Mechanical grading, Full-size bending test

1 INTRODUCTION

As an important structural material in light wood structure building, dimension lumber must be graded before using in the market in order to make sure that lumber satisfies the limits and requirements of material mechanical properties and appearance quality in building structure. Generally speaking, there are two ways of grading: visual grading and mechanical grading [1]. Visual grading which based on a long-term productive practice is evaluated according to grading rules by the naked eyes observing the defects, which influenced strength and appearance of timber. While, mechanical grading, usually conducting non-destructive testing and based on the visual grading, is grading the lumber in the light of its flexural strength and modulus of elasticity to determine the strength grade of timber. The abroad research on lumber grading started earlier than domestic [2-5], many countries have established their own grading standards and systems in conformity with their actual conditions, which are still being adjusted and improved. In contrast, our researches on dimension lumber grading are relatively backward [6-12]. Dahurian larch is a major commercial forest resource in Northeastern China. It is an important timber species for Chinese wood structural materials application, as it is relatively fast growing, and has good strength and decay resistance. In order to use the excellent building material and to best exert its service performance, it is necessary to graded lumbers especially when they are used in the construction. In this paper, dahurian larch dimension lumber was conducted researches on visual grading and mechanical grading, in the last, the article come up with a relatively reasonable grading method for Chinese larch dimension lumber, and optimized the service performance of the structural dimension lumber.

2 MATERIALS AND METHODS

Dahurian larch (Larix gmelinii), produced in Northeastern China were collected from the Forestry Bureau in Heilongjiang. Logs of age 35 year and higher, and with diameters ranging in size from 16 ~ 34 cm were used to manufacture dimension lumber by the cant sawing method. Dimension lumber specimen cross-section was 40mm×90mm and 4000mm long. The lumber sample was then kiln-dried and air-dried to about 12% moisture content before testing.
First of all, larch dimension lumber was visually graded based on its defects of natural growth, drying and processing, visual grading was conducted according to GB 50005-2003 "Code for Design of Timber Structures" and National Lumber Classification Authority (NLGA). After visual grading, all specimens were full-size tested in edgewise bending under third point loading and over a span corresponding to 18 times the standard height of the specimen in accordance with GB 50329-2002 "Standard Test Methods for Timber Structure". The tension side was selected randomly and the Maximum Strength Reducing Defect (referred to MSRD), determined prior to testing, was randomly located in the test span. After testing, full cross-section samples approximately 40 mm long were cut from near point of failure to determine the moisture content of specimens.

3 RESULTS AND DISCUSSIONS

3.1 VISUAL GRADING STATISTICAL AND ANALYSIS RESULTS

According to the rules of visual grading and based on the evaluation of various types of defects, larch dimension lumbers were sorted into Ⅰc, Ⅱc, Ⅲc, Ⅳc and reject grades. Visual grading statistical results of larch dimension lumber are shown in Table 1.

Table 1: Visual grading statistical results of larch dimension lumber

<table>
<thead>
<tr>
<th>Visual grades</th>
<th>Number</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ⅰc</td>
<td>397</td>
<td>37</td>
</tr>
<tr>
<td>Ⅱc</td>
<td>177</td>
<td>16</td>
</tr>
<tr>
<td>Ⅲc</td>
<td>243</td>
<td>22</td>
</tr>
<tr>
<td>Ⅳc</td>
<td>137</td>
<td>13</td>
</tr>
<tr>
<td>reject</td>
<td>131</td>
<td>12</td>
</tr>
</tbody>
</table>

From Table 1, we can see that the proportion of Ⅰc grade is around 40%, which higher than other grades, the second goes to Ⅲc grade and the ratios of Ⅳc and reject grade are almost equal. The results showed that we could get more high-level dimension lumbers through visual grading and this method could improve the utilization rate of dimension lumbers.

Visual grading is mainly based on the various defects of dimension lumber. For structural lumber visual grading, the major defects which involve in the grading rules are knot, wane, decay, slope of grain, shake/checks and so on. In this paper, we studied the major grade-reducing defects of each visual grade of larch dimension lumber, considering the defect size of Ⅰc grade is relatively small and the grading rule has not strict requirements for this grade, in addition, lumbers of reject grade have little value in actual structure, therefore, this research focus on three grades: Ⅱc, Ⅲc and Ⅳc. Figure 1 shows the major grade-reducing defects of various grades of larch dimension lumber.

It can be seen from Figure 1, the major reducing-grade defects are knot, shake/checks, decay and skips. Among them, knot is the main reducing-grade defect, its proportion is more than 95%, while shake/checks and knot are main reducing-grade defect for Ⅲc grade, for Ⅳc grade, the ratio of knot is highest, the next are decay and shake/checks, which have the almost same ratio. From the above conclusion, we know that knot and shake/checks are major reducing-grade defects, this result can provide reference for larch sawn timber processing and drying technology.

3.2 MECHANICAL GRADING OF STRUCTURAL LARCH DIMENSION LUMBER

Typically, there has a good positive correlation between the stiffness and strength of wood, most mechanical grading systems and the standard grades of mechanically graded lumber make use of this relationship [13]. Although methods vary between machines, most mechanical graders sort dimension lumber on the basis of the modulus of elasticity (MOE), with the assumption that grade categories with higher MOE will have higher modulus of rupture (MOR) and other strength properties. MOE plays an important role in evaluating the performance of dimension lumber; it is the basis of other mechanical properties of wood. The bending mechanical properties of larch dimension lumber were obtained with the lateral loading method through the full-size bending tests, and then went on related analyzing and processing for larch dimension lumber mechanical properties, as shown in Figure 2.
Figure 2 shows, the MOE of larch dimension lumber was normal distribution, data distribution was concentrated, mainly in the 11~17GPa; Therefore, in the properties assessment system of larch dimension lumber, the MOE can be used as predictive index to assess other related mechanical properties, which is attributed to the stability of MOE distribution.

In addition, the relationship between strength properties of dimension lumber is the basis of building evaluation system of intensity. The elastic modulus of elasticity and flexural strength which were got from the static bending tests were one to one, Figure 3 shows the relationship between MOE and MOR of larch dimension lumber.

It can be seen from Figure 3, the MOE and MOR of larch dimension lumber has a good correlation, the coefficient of determination was 0.552, and this result is similar to the findings from other scholars about larch dimension lumber [14]. According to the known MOE value, through the use of regression, we can predict the MOR value of larch dimension lumber, which can reduce test work and save cost.

This study tested about 900 larch dimension lumber and obtained the appropriate physical and mechanical properties, and then analyzed the mechanical properties.

With the forecasting analysis, larch dimension lumber MOE was normal distribution and the MOE and MOR has a good positive correlation, therefore, MOE was selected in this study as predict indicator to evaluate other mechanical properties of larch dimension lumber. This study got each larch dimension lumber grade characteristic values under three grade boundaries restrictions and statistics of each grade dimension lumber and proportion, as presented in Table 2.

### Table 2: Mechanical grading statistical results of larch dimension lumber

<table>
<thead>
<tr>
<th>Grades</th>
<th>MOE /GPa</th>
<th>MOR /MPa</th>
<th>Total Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>CV</td>
<td>Mean</td>
</tr>
<tr>
<td>M14</td>
<td>9.6</td>
<td>7.6</td>
<td>10.2</td>
</tr>
<tr>
<td>M30</td>
<td>13.9</td>
<td>11.4</td>
<td>11.5</td>
</tr>
<tr>
<td>M40</td>
<td>18.9</td>
<td>17.1</td>
<td>8.2</td>
</tr>
</tbody>
</table>

Note: Grade boundaries are set which based on the static MOE values.

In the mechanical grading of dimension lumber, the principal of grading is to raise the proportion of higher grade as much as possible, in order to make full use of resources and to enhance the value of products. Table 2 shows that, the larch dimension lumber could be sorted into M14, M30 and M40 three grades through divided the different MOE interval, the ratio of M30 and M40 accounted for 83.8%; MOE coefficient of variation was lower, at about 10%, further illustrated the low variability of MOE; as larch dimension lumber mechanical grades increase, the characteristic values of MOR obviously improved, thus the coefficient of variation decreased with the increase of mechanical grades, indicating that the strength variability of lumber decline.

### 3.3 COMPARISON OF VISUAL GRADING AND MECHANICAL GRADING

The strength characteristic values of each visual grade of larch dimension lumber was obtained by full-size bending test. According to the bending properties, the grade boundary condition of larch dimension lumber was set to mechanical graded larch dimension lumber, meanwhile got the strength characteristic values of each mechanical grades (Table 3).

### Table 3: Statistical results of larch dimension lumber visual grades and mechanical grades

<table>
<thead>
<tr>
<th>Grading methods</th>
<th>Grades</th>
<th>Size</th>
<th>MOE /GPa</th>
<th>MOR /MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
<td>CV</td>
</tr>
<tr>
<td>Visual grading</td>
<td>l c</td>
<td>389</td>
<td>15.2</td>
<td>19.3</td>
</tr>
<tr>
<td></td>
<td>ll c</td>
<td>169</td>
<td>13.0</td>
<td>19.3</td>
</tr>
<tr>
<td></td>
<td>lll c</td>
<td>234</td>
<td>13.6</td>
<td>22.6</td>
</tr>
<tr>
<td>Mechanical grading</td>
<td>M14</td>
<td>150</td>
<td>10.1</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>M30</td>
<td>664</td>
<td>14.4</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>M40</td>
<td>80</td>
<td>19.3</td>
<td>11.7</td>
</tr>
</tbody>
</table>
Through comparing, mechanical grading can significantly reduce the MOE and MOR coefficient of variation of dimension lumber, such as MOE and MOR coefficient of variation of M40 were reduced to 10% and 25%, while in the visual grading of dimension lumber, the MOR coefficient of variation was relatively higher, basically was 35% above, it explains that mechanical grading can improve the high grade lumber strength characteristic value and the safety coefficient, the strength properties of lumber which mechanically graded presented more stable.

According to the results which presented by Table 3, the grade strength of larch dimension lumber can be sorted as sequence as M40, Ⅰc, M30, Ⅱc, Ⅲc and M14. Dimension lumber as a commodity, the lumber manufactures usually pay more attention to its market value, which related to its grade and yield. On the basis of referencing the prices of import larch dimension lumber and comprehensive analyzing the market price of log and timber of larch forest, we preliminary evaluate the market price and output value of larch dimension lumber.

**Table 4: Each grade economic value assess of Chinese larch dimension lumber**

<table>
<thead>
<tr>
<th>Grades</th>
<th>Number of dimension lumber</th>
<th>M40</th>
<th>Ⅰc</th>
<th>M30</th>
<th>Ⅱc</th>
<th>Ⅲc</th>
<th>M14</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>68</td>
<td>378</td>
<td>578</td>
<td>164</td>
<td>225</td>
<td>121</td>
</tr>
<tr>
<td>Price(RM B)/(Yuan·m-3)</td>
<td>3800</td>
<td>3600</td>
<td>3500</td>
<td>3000</td>
<td>2500</td>
<td>2300</td>
<td></td>
</tr>
<tr>
<td>Output value(RM B)/Yuan</td>
<td>3721</td>
<td>19596</td>
<td>29131</td>
<td>7085</td>
<td>8100</td>
<td>4008</td>
<td></td>
</tr>
</tbody>
</table>

Table 4 displays the values of larch dimension lumber which obtained by visual grading and mechanical grading, they were respectively RMB 34781 and RMB 36860, from this we can see that the unit value of larch dimension lumber got by mechanical grading is RMB 188.23 higher than visual grading. Therefore, mechanical grading can greatly improve dimension lumber value and increase economic benefit, especially for mass production of dimension lumber.

Relative speaking, the efficiency of mechanical grading is improved greatly than visual grading, in addition, mechanical grading can be used in industrial production, the speed of mechanical grading machine is higher, the highest grade speed can reach 10000 lumbers per day, which greatly saves time and cost.

4 CONCLUSIONS

In visual grades of larch dimension lumber, the proportion of Ⅰc grade was higher than other grades, knot and shake/checks were the major reducing-grade defects for larch dimension lumber.

The MOE of larch dimension lumber obeyed normal distribution, MOE was selected as predict indicator and was used to set grade boundary conditions, and then sorted larch dimension lumber into M14, M30 and M40 three grades.

The grade strength of larch dimension lumber was sorted as sequence as M40, Ⅰc, M30, Ⅱc, Ⅲc and M14. Mechanical grading greatly enhanced the grading efficiency and improved the strength characteristic values of high-grade dimension lumber; besides, mechanical grading also decreased the coefficient of variation of characteristic values for dimension lumber. Mechanical grading is more suitable for Chinese larch dimension lumber.

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REFERENCES


THE SECULAR CHANGE OF THE TIMBER BRIDGE “KINTAI-KYO”

Mikio Koshihara1

ABSTRACT: The timber bridge, “KINTAI-Kyo”, is located in Iwakuni City, Yamaguchi Prefecture, JAPAN and was built in 1673. After rebuilt in 2002, the shape of bridge, temperature and humidity are measured. The following results were provided by these measurements. In the first year, the deformation at the center of bridge move 20mm downward for gaps between the structural elements. And there is a seasonal change of the deformation at the center of the bridge. At the center of bridge it moves upward in summer and downward in winter. The range of seasonal change is about 20mm. These changes are influences by temperature and humidity. In 2005 the irregular change of shape was observed in the autumn because no.1 bridge was washed away by the typhoon. These measurements have a possibility of safety diagnosis for timber bridge.

KEYWORDS: Timber bridge, Deterioration, Health monitoring

1 INTRODUCTION

The timber bridge, “KINTAI-Kyo”, is located in Iwakuni City, Yamaguchi Prefecture, JAPAN. Figure 1 shows an elevation of “KINTAI-Kyo” bridge. The bridge consists of five timber ones and is supported on each stone pillar. The bridge is 193.3m long and 5m wide. Three bridges of the middle part, from No.2 to No.4, are 35.1m long and the rise is about 5m. It was constructed first in 1673 of Edo era by a feudal lord of the time, Hiroyoshi Kikkawa, to meet the transportation problems which the people faced every time when the Nishiki River flooded. Unfortunately, the entire bridge was washed away by a flood in May, 1674. But it was reconstructed in October, 1674. After that the bridges were often repaired and reconstructed partially. Each bridge was repaired about every 12–15 years and reconstructed about every 20 years.

In 2001 “KINTAI-Kyo” was rebuilt as same shape in 1953. It is marked by the beauty of its five arches. The bridge length is 193m, every arch length is about 35m, its rise is about 5m and the wide is 5m. This bridge was made by some structural elements, KYOROKU(timber beam), KURA-GI(timber bracing), TASUKE-GI(timber parts), TAGA(iron hoop), KASUGAI(iron cramp), DABO(timber shear connector), nails and MAKIGANE (hoops), as shown in Figure 2.

There are past researches about the secular changes of “KINTAI-KYO”. The properties of vibration are measured and static loading tests were conducted by Waseda university[1]-[3] from 1953 with the interval of 5 years.

On the reconstruction of these bridges in 2002, investigation about deterioration was conducted. In these investigation two greatly deterioration were found. One is the decay of the bridges especially in the joint between
the floor board and top of HEIKINGI as shown in figure 3. This decay of timber was caused by the puddles around the pegs. Other deterioration is the deformation of dowels between the beams as shown in figure 4. The horizontal deformations of the dowels were about 2-4 mm.

Figure 2: Detail of “KINTAI-Kyo”

Figure 3: A decay of the top of “HEIKINGI”

Figure 4: Deformation of dowel between beams

2 SECULAR CHANGE OF TIMBER BRIDGE, “KINTAI-Kyo”

2.1 MEASUREMENT METHOD

The bridge shape and factors of secular change are being measured from 2003. In this year the no.4 and no.5 bridges were completed and just one year was passed from the no.3 bridge was completed. The no.2 and no.1 bridges were reconstructed in 2004 and there are the no.2 bridge passed about 50 years in 2001.

2.2 SEAPE

The shapes of the middle three bridges (no.2, no.3 and no.4) are being measured with theodolite. 13 targets were measured on every one side of bridge and 26 targets were measured on every bridge. Target points are the rivets bottom of the balustrades (figure 5). Measurements were started on March 14, 2003. In the first month the shape was measured with the interval of one week and in next two months it was measured with the interval of two weeks. After three month it was measured with the interval of one month and it was measured every 3 month from August 1, 2005.

2.3 TEMPERATURE AND HUMIDITY

Temperature and humidity influence the water content and the strain of wood. Especially wooden bridge is located in hard environment, above the river.

Figure 5: Target point of measurement

Figure 6: Thermo recorder
Temperature and humidity under the bridge are being measured with thermo recorder, T & D’s TR72S, every one hour (figure 6).

2.4 LIVE LOAD

For timber structure the creep deformation is one of the largest influences to the secular change. In case of this bridge the continuous or repeating loads and is dead load, its weight, and live load, crossing person. For estimating the live load, the number of sold tickets to cross the bridge was counted as the number of persons crossing bridge. Actually this load is underestimating because there are many not counting people with coupon tickets and group tickets.

3 RESULTS

3.1 SECULAR CHANGE OF THE BRIDGE SHAPE

Figure 7 shows the deformation of the center of bridge. The horizontal axis shows how many days passed from the completion of each bridge. No.3 bridge is completed in 2002, no.4 bridge is from 2003 and no.2 bridge is from 2004. In the first year, from 0 to 365 days, the deformation of center of all bridges went down about 20mm. But the deformation of no.2 was a little small. It is because this no.2 bridge has completed lastly and there is no construction after completion. After the first year they go up in summer and go down in winter. The width of change is about 15mm. The deformation of each point of no.4 bridge in the summer (Aug. 1, 2006) and in the winter (Feb. 1, 2007) are shown in figure 8. The maximum deformation of bridge is at the point of a third part of bridge.

The deference of the deformation between at the upper part and lower part of the river in bridge is shown in figure 9. This figure shows the deformation at the point 405 shown in figure 8. The deformation at the upper part of the river in bridge is larger than at the lower part of the river and its difference grows larger. The deference is about 10mm in 5m width.
3.2 FACTOR OF CHANGE

3.2.1 Temperature and humidity

Temperature and humidity were measured by thermo recorder every one hour. The results are shown in figure 10 and figure 11. Both temperature and humidity value are 7-day average before measuring the shape of bridges. The horizontal axis shows how many days from every January 1.

Temperature changes cyclically. The highest temperature, about 30 degrees Celsius, was recorded in August and the lowest temperature, about 5 degrees Celsius, was recorded in February. Considering the change of relative humidity, it is high because this bridge is located above the river. Especially it is over 90% in every night and in every morning. But the change in the year is not cyclically and it is different from every year.

3.2.2 Live load

The number of sold tickets to cross the bridge was counted. This ticket is round-trip ticket, so the number of crossing the bridge is twice as this number. The number of sold tickets in month is shown in figure 12. About 80-240 thousand persons cross the bridge every month. There are two peaks and over 100 thousand persons cross the bridge in April and November. In April they enjoy the cherry blossom on the bank of river and in November they enjoy autumn leaves. There is small peak in August and this reason is summer vacation. There are two off-peak in July and December. In June and July is the rainy season in Japan and in December it is cold in Iwakuni-city.

These numbers are reference value and maybe lower limit because there were the crossing persons who did not buy the tickets.

3.2.3 Gap of connectors

“KINTAI-KYO” bridge is mainly composed of beams and these beams are connected to the other beam with dowels. These dowels are deformed by the shear force between beams. This deformation influences the shape of the bridges. By the field experiments in 2003[4]-[5] it is cleared that the vertical deformation of the center of bridge is 14.3mm if the gap between beams is 0.28mm and that deformation is 27.39mm if the gap is 0.67mm.

3.2.4 Typhoon

On September 6, 2005, the column of the no.1 bridge was washed away by a flood. The damage of no.1 bridge after typhoon no.14 is shown in figure 13.
4 DISCUSSION

4.1 THE DEGREE OF THE INFLUENCE TO SECULAR CHANGE

Timber bridge, “KINTAI-KYO” is changing its shape secularly and cyclically. Considering the cause of secular change of wooden bridge, there is many factor described in 3.2. In this chapter the degree of influence is considered.

4.2 TEMPERATURE AND HUMIDITY

Temperature and humidity influence to the moisture contents of wooden members. Figure 14 shows the relation between the deformation of center of bridge and the moisture content. The moisture content of wooden member is calculated according to the theory of Hawley, Keywerth, Kollmann et.al. In the no.3 and no.4 bridge the correlation between the deformation and moisture content, but in the no.2 bridge there is no correlation. In this figure the moisture content is changing from 10% to 15%. If the moisture content of wooden member is change from 10% to 15%, the change of length parallel to grain is about 0.15% and length perpendicular to grain is about 1.5%.

4.3 TYPHOON

The no.1 bridge was washed away by a flood. The other bridge were damaged by washing away of no.1 bridge. The deformation of center of bridge was changed irregularly as shown in figure 15. The no.4 bridge did not change as same as the no.2 and no.3 bridge. If these change can be monitoring, some damages can be found. These measurements have a possibility of safety diagnosis for timber bridge.

5 CONCLUSION

After rebuilt in 2002, the shape of bridge, temperature and humidity are measured. The following results were provided by these measurements. In the first year, the deformation at the center of bridge move 20mm downward for gaps between the structural elements. And there is a seasonal change of the deformation at the center of the bridge. At the center of bridge it moves upward in summer and downward in winter. The range of seasonal change is about 20mm. These changes are influences by temperature and humidity. In 2005 the irregular change of shape was observed in the autumn because no.1 bridge was washed away by the typhoon. These measurements have a possibility of safety diagnosis for timber bridge.

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REFERENCES

CHEMICAL PROPERTIES AND DECAY RESISTANCE OF *Eucalyptus grandis* WOOD FROM STEAMED LOGS

Cinthia Dias Rocha¹, Elias Taylor Durgante Severo², Fred Williams Calonego³, Cláudio Angeli Sansigolo⁴

**ABSTRACT:** The objective of this study was to evaluate the effect of the log steaming on the chemical properties and decay resistance of *Eucalyptus grandis* wood. Logs with diameter between 20 and 22 cm were studied. Half of the logs were kept in its original condition, and the other half was steamed at 90°C for 13 hours. The holocellulose, Klason lignin, total extractives content and the weight loss caused by the decay fungus *Pycnoporus sanguineus* were characterized. The results showed that the log steaming of *E. grandis* wood caused: (1) a significantly decreased in holocellulose content; (2) an increase of 4.8% and 4.4% in total extractives and lignin content, respectively; and (3) a decrease in its durability against the decay fungus *P. sanguineus* in order of 13.03%.

**KEYWORDS:** Log steaming, Chemical properties, *Pycnoporus sanguineus*, *Eucalyptus grandis*.

1 INTRODUCTION

Recently, the lumber industry in Brazil has passed large changes. The native woods of tropical forests have presented use restrictions due to legislation and environmental friendliness. Moreover, Brazil is among the ten countries with largest reforested area, and the *Eucalyptus* is the most important kind, with about 4.5 million ha (ABRAF, 2010) [1]. However, the *Eucalyptus* wood shows adverse conditions when it is used in timber and furniture industry. The presence of high levels of growth stress cause cracks and distortions in logs and boards, resulting in low yield of the material. Techniques that have been used to relieve the tensions from growth stress include heating logs at high humidity during certain time. This process is called steaming and leads to a relaxation of tensions in the material through plasticization of lignin. Thus, the steamed wood presents a lower percentage of defects, and better quality during and after the sawing process. Studies have shown about 50% reductions in growth stress of *Eucalyptus saligna* logs after 24 hours of treatment in hot water (SKOLMEN, 1967) [2], significant reductions of 44% and 53% in the length and width of cracks in *Eucalyptus dunnii* after 20 hours of steaming (SEVERO and TOMASELLI, 2000) [3], and reductions of 34% and 48% in the length and width of cracks in *Eucalyptus grandis* logs after 24 hours of steaming at 90°C (CALONEGO and SEVERO, 2005) [4]. However, the heating of wood at high temperature cause degradation of hemicelluloses and at amorphous region of cellulose, modifying the original properties of material, especially when used high humidity or steam. Under these conditions the wood degraded by acid hydrolysis (FENGEL and WEGENER, 1984; STAMM, 1956) [5, 6].

This wood degradation caused by thermal treatment can significantly modify some properties of the material, such as chemical composition and biological durability. Momohara et al. (2003) [7] showed that *Cryptomeria japonica* heat-treated with steam at 105°C for 24h and exposed to fungus *Fomitopsis palustris* had weight loss of 27%, while the untreated wood had 31%. The authors concluded that 135°C after 24 h was the minimum conditions to promote significant improvements in a wood’s biological resistance to the fungus. According to Ishiguri et al. (2005) [8], the log steaming at 75°C of *Cryptomeria japonica* for 100 hours caused an decrease of 79 to 76% in holocellulose due to...
degradation of hemicelluloses and an increase of 8 to 10% in the solubility at 1% NaOH content.
The objective of this study was to evaluate the effect of log steaming on the chemical properties and biological resistance to decay fungus *Pycnoporus sanguineous* of *Eucalyptus grandis* wood.

2 MATERIAL AND METHODS

2.1 COLLECTION OF MATERIAL

The present study utilized wood from 5 years and 11 months old *Eucalyptus grandis* trees of the Rio Claro Farm managed by the Duratex SA Company located at coordinates 22°46'S and 48°52'W. Five trees were randomly selected from inside the 2.2 ha stand. Subsequent to felling, the trees were sectioned into 6.0-m logs. The first log from each tree with a diameter ranging between 20 and 22 cm was utilized.

2.2 THERMAL TREATMENT OF WOOD

The treatments to relieve growth stress included making grooves in the logs. Two groves were machined to a depth equal to one third of the radius of each log at 10 cm from the crosscut section of the log, according to Severo et al. (2010) [9].

Another procedure used to relieve growth stress was steaming of the logs. This treatment was applied to half-logs and the other half was kept as a control. The 3 m logs were steamed for a time determined from equations obtained by Steinhagen et al. (1980) [10] and Calonego and Severo (2006) [11]. According these authors, green logs of *Eucalyptus grandis* wood steamed for 13 hours at a temperature of 90°C and 100% relative humidity was sufficient for the center of the material reached 82°C which according by Quirino and Vale (2002) [12] is the temperature of glass transition of the lignin.

Then the logs (untreated and steamed) were cut into flat saw boards 40 mm thick containing the pith.

2.3 CHEMICAL PROPERTIES OF WOOD

Five boards from untreated and steamed wood were sawed into samples measuring 30 by 20 by 50 mm, which were transformed into chips and then processed into sawdust. This material was used for chemical analysis and was classified between 40 and 60 mesh. The procedures were implemented according to the standards presented in TAPPI (1999) [13]. Holocellulose, Kason lignin and total extractives content were characterized.

2.4 DECAY ACCELERATED TEST

The test conducted to evaluate the effect of logs steaming on the biological durability of *E. grandis* wood was the method known as the soil-block. Twelve boards from untreated and steamed logs were sawed into test blocks measuring 25 by 25 by 9 mm in size, with the 9 mm dimension in the grain direction. The procedures were implemented according to the standards presented in ASTM D-2017 (1994) [14]. In preparation, the samples were dried in a drying oven at 103 ± 2°C, until they reach a constant weight. As recommended by ASTM D-1413 (1994) [15], the initial oven-dry weight (WI) of each test block was determined.

The soil-block test was prepared in 725 mL cylindrical culture bottles using 300 g of soil with a pH of 5.50 and a water holding capacity of 29%. After filling the bottles with distilled water, a feeder strip of *Pinus* sp. was added. Subsequently, the bottles were sterilized at 121 ± 1°C for 1 h. After sterilization, the test blocks were placed in the culture bottles, with the cross-section face of the feeder strip facing down.

The culture bottles were incubated in an incubation chamber in the dark to promote the growth of the fungus and at 26.7 ± 1°C and 70 ± 4% relative humidity for 12 weeks. At the end of exposure period, the test blocks were removed from the culture bottles. They were then dried in the drying oven at 103 ± 2°C once again until they reached a constant weight. This weight was determined as the final oven-dry weight (WF) of each test block. The percent weight losses in the individual test blocks from before and after exposure to the decay fungus was then calculated.

The percent weight losses in the test blocks provide a measure of the relative decay susceptibility or, inversely, of the decay resistance of the steamed *E. grandis* wood. Chemical properties and decay resistance data were normally distributed and paired t test at 5% significance were used for comparison of the means.

3 RESULTS AND DISCUSSION

3.1 CHEMICAL PROPERTIES

The holocellulose, Kason lignin and total extractives contents of untreated wood from *Eucalyptus grandis* were 81.37%, 23.26% and 1.26%, respectively. These results are presented in Table 1 and are similar to those cited by Andrade et al. (2010) [16].

The effect of log steaming of *Eucalyptus grandis* in chemical composition is also showed in Table 1. This study shows that steaming logs causes increases in Kason lignin and in total extractives content of 4.4% and 4.8%, respectively. Already, the holocellulose content presented a significant decrease of 3.1% when compared to untreated wood.

Table 1: Chemical composition of steamed and untreated *Eucalyptus grandis* wood.

<table>
<thead>
<tr>
<th>Chemical Composition (%)</th>
<th>Holocellulose</th>
<th>Kason Lignin</th>
<th>Total Extractives</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>C.V.</td>
<td>Mean</td>
<td>C.V.</td>
</tr>
<tr>
<td>Untreated</td>
<td>5</td>
<td>0.5</td>
<td>81.38a</td>
</tr>
<tr>
<td>Steamed</td>
<td>5</td>
<td>1.7</td>
<td>78.86b</td>
</tr>
<tr>
<td>Increase or (reduction) (%)</td>
<td>--</td>
<td>(3.1)</td>
<td>--</td>
</tr>
</tbody>
</table>

Where N - repeat number of samples; C.V.- coefficient of variation; Different letters - significant difference by t-test at 95% probability.
The decrease of holocellulose is similar to those found by Ishiguri et al. (2005) [8]. Ishiguri et al. (2003) [17] also reported a reduction of 1.6% in holocellulose content when Cryptomeria japonica wood was exposed for 70 hours in the presence of steam. The decrease of holocellulose content is explain by the degradation of hemicelluloses. Thus, this results were similar with those reported by Fengel and Wegener (1984) [5] and Stamm (1956) [6] that under high humidity, there is an increase in the degradation of the hemicelluloses by acid hydrolysis.

The increase of Klassen lignin is the result of a proportional increment caused by the degradation of hemicelluloses. Similar behavior has also been observed by Yilgor et al. (2001) [18], who studied the effect of steaming of Fagus orientalis at 80°C and concluded that there was an increase of 3.9% in lignin content. The increase of lignin was also reported by Brito et al. (2008) [19] due a decrease in hemicelluloses content after a thermal treatment of Eucalyptus saligna at 120°C.

Already, the increase of total extractives was explained by Yilgor et al. (2001) [18] and Ishiguri et al. (2005) [8] who emphasize that steam promoted a massive mobilization of the wood components and that the new compounds formed by thermal degradation may have been extracted by solvents as a fraction of extractives content.

### 3.2 DECAY TEST

The weight loss caused by decay fungus P. sanguineus in untreated and steamed Eucalyptus grandis is found in Table 2.

The value for the amount of weight loss in untreated E. grandis wood was 25.94% for the samples tested in the soil-block test. Based on the result presented, E. grandis wood can be classified according to ASTM D-2017 (1994) [14] in the class of moderate resistance, one in which the weight loss due to decay varies between 25% and 44%. This result is similar to that cited by ALEXIOU et al., 1990 [21]. That caused a greater micelial growth, and therefore, a higher weight loss of treated wood.

Similar behavior was reported by Doi et al. (2005) [22] when Larix leptolepis was steaming for 8h and inoculated with fungus Fomitopsis palustris during 90 days. The untreated wood showed 5% of weight loss while the steamed wood presented 14% of weight loss.

### 4 CONCLUSIONS

This study shows that log steaming of Eucalyptus grandis showed: (1) a significantly decreased in holocellulose content; (2) an increase of 4.8% and 4.4% in extractives and lignin content, respectively; and (3) a decrease in its durability against the decay fungus P. sanguineus in order of 13.03%.

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### REFERENCES


MECHANICAL PERFORMANCE OF HARDWOOD (ZELKova SERRATA) FRAME ASSEMBLIES IN FIRE

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KEYWORDS: zelkova serrate, hardwood, fire resistance test, charring rate, young’s modulus

1 INTRODUCTION

In spite of the wide use of hardwood (Zelkova serrata) for the load-bearing members of large traditional buildings such as main buildings of temples (see Fig.1) and castles in Japan for centuries, little effort has been made for the promotion of the engineering design of its seismic and fire protection performance. The stagnant engineering development in the hardwood construction has caused significant difficulty in its use during the recent strengthening of the building regulation in Japan.

This study intends to make a breakthrough in the contemporary use of hardwood for buildings under the cooperation of forestry, construction firm, structural engineers and fire safety engineers. The study consists of the measurements of important properties for the assessment of the mechanical and fire resistance performance, development of the predictive method of the mechanical fire safety performance and validation of the predictive method by large-scale loaded fire test on hardwood columns and beams.

The predictive method is based on the estimation of the buckling load for the geometric moment of inertia with the Young’s modulus at the internal temperature and the reduced section area due to charring both predicted for a cross section of the load bearing member. Since little was known on the Young’s modulus of Zelkova at elevated temperature, charring rate and formation of temperature profile within a Zelkova column exposed to the standard fire resistance test exposure, the study was initiated with the measurement of these properties in the laboratory small – medium scale experiments.

2 MEASUREMENTS OF IMPORTANT PROPERTIES OF HARDWOOD

Two experimental procedures have been conducted to measure important properties of hardwood for the assessment of the mechanical and fire resistance performance.

2.1 UN-LOADED FIRE RESISTANCE TEST

To investigate charring rate and formation of temperature profile within a Zelkova column exposed to fire, un-loaded 1-hour fire resistance tests have been conducted with Zelkova columns of about 350mm in diameter and 1m in height (see Fig.2). Japanese Cedar and Cypress columns were also tested to compare the performance of hardwood with that of

Fig.1 Buddhism temple with hardwood for building component (Daijoji in Tama City, Tokyo)
softwood. And to examine the influence of season crack, which is specific to solid wood, to charring behavior, the specimens with quasi-crack were also tested (see Fig.4). The specimens were exposed to the ISO834 standard fire curve. The temperature inside columns was measured with thermocouples inserted in columns.

The charring rate of every tree species (non-cracked) calculated from experimental result was 0.7 to 0.8mm/minute (see Table1). It suggests that the charring behavior of *Zelkova*, Japanese Cedar and Cypress are almost equal. Comparing the charring rates of specimens, which are affected by the difference of width and depth of crack on the specimens, the wider crack the specimen had, the higher its charring rate was. However, the depth of crack had little influence on charring rate of the specimen. The charring depth of the non-cracked parts of cracked specimen was about 40mm (see Fig.6), and it was as deep as that of non-cracked specimen (see Fig.5). Then, it is considered that small crack have little influence on the charring rate of large section wooden member.

The formation of temperature profile within specimens were shown in Fig.7. It shows the relationship between the depth from the char layer and the highest achieving temperature from the experimental result. The formation of temperature profile within *Zelkova* was almost equal to that within Cypress and lower than that within Japanese Cedar. In Fig.7, the dot-line shows the formation of temperature profile within column used in the predictive method of mechanical fire resistance of softwood. The formation of temperature profile within *Zelkova* was equal to or lower than that used in the predictive method of mechanical fire resistance of softwood.

In light of these result, the numerical values were determined that were used in the predictive method of mechanical fire resistance of hardwood. The following numeral values were used in the method.

- Charring rate : 0.8mm/minute
- Formation of temperature profiles inside the column: the same value used in the predictive method of mechanical fire resistance of softwood

### Table 1: Specification and result of the test

<table>
<thead>
<tr>
<th>No.</th>
<th>Species</th>
<th>Size</th>
<th>Moisture content (%)</th>
<th>Depth and width of crack(mm)</th>
<th>Depth of char layer(mm)</th>
<th>Charring rate (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>zelkova</td>
<td>φ354mm</td>
<td>13.7</td>
<td>40.1 45.8 0.7</td>
<td>50 30 0.7</td>
<td>300 200 80 50</td>
</tr>
<tr>
<td>2</td>
<td>cypress</td>
<td>φ360mm</td>
<td>14.4</td>
<td>31.6 46.5 0.5</td>
<td>45 38 0.5</td>
<td>300 200 80 50</td>
</tr>
<tr>
<td>3</td>
<td>cedar</td>
<td>φ360mm</td>
<td>18.3</td>
<td>29.1 44.5 0.5</td>
<td>38 32 0.5</td>
<td>300 200 80 50</td>
</tr>
<tr>
<td>4</td>
<td>zelkova</td>
<td>φ356mm</td>
<td>27.5</td>
<td>33.1 39.0 0.6</td>
<td>42 35 0.5</td>
<td>300 200 80 50</td>
</tr>
<tr>
<td>5</td>
<td>cypress</td>
<td>φ357mm</td>
<td>19.8</td>
<td>31.8 42.4 0.5</td>
<td>44 37 0.5</td>
<td>300 200 80 50</td>
</tr>
<tr>
<td>6</td>
<td>zelkova (cracked)</td>
<td>φ358mm</td>
<td>13.1</td>
<td>W2.5 D:50 44.1 0.7</td>
<td>W2.5 D:30 52.7 0.9</td>
<td>300 200 80 50</td>
</tr>
</tbody>
</table>
2.2 TEST FOR MEASUREMENT OF YOUNG’S MODULUS OF ZELKOVA AT ELEVATED TEMPERATURE

Although the performance of Young’s modulus of soft wood at elevated temperature had been already revealed by bending tests with Japanese Cedar specimens, that of hardwood haven’t been known yet. Then to investigate the remaining ratio of Young’s modulus of Zelkova at elevated temperature, the three-point bending tests have been conducted by means specified in JIS (see Fig.9). About one hundred specimens (20mm × 20mm × 320mm) made of the same log were buckled (see Fig.8). To eliminate the influence of decrease in water content to Young’s modulus, moisture content of specimens were decreased to about 2 percent before the test with drying machine.

At first, Young’ modulus at normal temperature of every specimens were measured by non-destructive bending tests. The measured value was from 70tf/cm² to 134tf/cm² and the average was 105tf/cm², almost equal to the widely used Young’s modulus of Zelkova. Next, Young’ modulus at elevated temperature (180°C, 150°C, 100°C and 50°C) were measured by destructive bending tests. Twenty specimens were tested at each temperature.

The remaining ratio of Young’s modulus was worked out by calculating ratio between Young’ modulus at normal temperature and that at elevated temperature condition. The average value of the remaining ratio of Young’s modulus of Zelkova at 180°C, 150°C, 100°C and 50°C were each 0.73, 0.84, 0.92 and 0.97 (see Fig.10).

The result shows that Young’s modulus of Zelkova decreases as the temperature gets higher, and it is also observed in the case of softwood. Although they showed similar tendency, the Young’s modulus of hardwood decreased at slower rate than that of softwood. Therefore, in this paper, the remaining ratio of Young’s modulus used in the predictive method of mechanical fire resistance of softwood was determined to be used in that of hardwood.

3 LOADED FIRE RESISTANCE TESTS ON ACTUAL-SIZE COLUMNS OF ZELKOVA

To develop the large-section hardwood column with 1-hour quasi-fireproof construction, loaded fire resistance tests were conducted with three Zelkova...
columns, and it was examined how moisture content and Young’s modulus influence to mechanical fire resistance of hardwood column. And the predictive method of mechanical fire resistance of hardwood was proposed in this chapter and verified by comparing the predicted performance with the result of the test.

The specimens were three Zelkova columns of 343mm in diameter and 3500mm in height (see Fig.11). The test was continued until the column was buckled.

The predictive method of mechanical fire resistance of hardwood is proposed in Fig.16. This method is based on the result of the tests discussed in the preceding chapter. Predicted time for each specimen to get buckled was calculated. The predicted time of specimen A was 74 minutes, that of specimen B was 79 minutes and that of specimen C was 94 minutes (see Table 2).

As the result of test, the vision was received of development of large section hardwood columns with 1-hour quasi-fireproof construction and it was considered that the predictive method of mechanical fire resistance is applicable to hardwood.

The following conclusions are drawn.

1. Charring rate of Zelkova serrata under ISO834 standard fire resistance test exposure is roughly 0.8mm/min.
2. The internal temperature profile within a large column of Zelkova serrata is nearly identical to that of Japanese Cedar, more widely used and studied wood in Japan, irrespective of notable difference in the density.
3. Reduction of the Young’s modulus at elevated temperature is less significant with of Zelkova serrata than with Japanese Japanese Cedar.
4. The buckling load of a column or a beam of Zelkova serrata can be predicted by using the temperature dependence of the Young’s modulus and the charring rate data obtained from smaller specimens.

**Table 2: Specification and result of the test**

<table>
<thead>
<tr>
<th>No.</th>
<th>Species</th>
<th>Size (mm)</th>
<th>Moisture content (%)</th>
<th>Young modulus (df/cm²)</th>
<th>Load (kN)</th>
<th>Predicted time column buckled (min)</th>
<th>Termination time of test (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Zelkova</td>
<td>343 x 3500</td>
<td>28.9%</td>
<td>95</td>
<td>640</td>
<td>74 &lt; 82</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Zelkova</td>
<td>343 x 3500</td>
<td>26.3%</td>
<td>111</td>
<td>79</td>
<td>79 &lt; 94.5</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td></td>
<td>343 x 3500</td>
<td>27.3%</td>
<td>94</td>
<td>320</td>
<td>94 &lt; 111</td>
<td></td>
</tr>
</tbody>
</table>
INVESTIGATION OF THE TENSILE STRENGTH OF KEYED MORTISE AND TENON JOINTS IN TIMBER FRAME STRUCTURES

Daniel P. Hindman¹, Lance D. Shields², Joseph R. Loferski³

ABSTRACT: Timber frame joinery requires special attention to joinery, particularly in connections exposed to tensile loads. The keyed through-tenon joint has been used in timber frame to connect interior members in tension, but has received little attention in terms of experimental strength evaluation. The purpose of this paper was to investigate the tension capacity of keyed through-tenon joints examining effects of wood species, tenon length and number of keys. Experimental methods loaded the timber frame joint in tension. Failure of the joint was largely governed by tenon length, with short length tenons producing tenon tear-out failures, while longer tenons produced key bending failures. Ductile failures were produced by long tenons (28 cm) and were accurately predicted using the model results.

KEYWORDS: Keyed through-tenon joints, wood-to-wood connections

1 INTRODUCTION

A timber frame is a structural building system composed of heavy timber members connected using carpentry-style joinery such as dovetails, mortise-and-tenon, etc. A common type of mortise-and-tenon joint is a keyed (or wedged) through-tenon joint (Figure 1) used in historic structures and by the modern timber frame industry in the United States.

Figure 1: Keyed Through-Tenon Joint

Creating tension resisting joints is a difficult aspect of timber construction due to the equivalent strength of traditional cut joinery and the timbers connected. Cutting tension joinery limits the tensile capacity remaining. This capacity is then further reduced by shear components such as fasteners used to connect the members [1].

Keyed through tenon joints generate withdrawal resistance due to the keys applying load on the back of the mortise member without reducing the net section of the mortise member with peg holes [1]. The only known research pertaining to keys includes studies on bending members such as scarf joints and keyed beams. Current design practices ascribe little to no strength to keyed through-tenon joints causing the overdesign of timber frame members. By establishing the tensile capacity of keyed through-tenon joints, a better understanding of the load capacity and more efficient design of structures is created.

2 BACKGROUND

Timber frame engineering has been practiced by a small group of specialized structural engineers and has always been challenging accordingly. The National Design Specification for Wood Construction (NDS) [2] uses design procedures for sizing timbers and provides design provisions for timber connections using steel fasteners but does not mention timber frame joinery. The Timber Frame Engineering Council (TFEC) is an organization of structural engineers that specialize in timber frame engineering who developed the Standard for Design of Timber Frame Structures and Commentary [3]. TFEC 1-2010 serves as a supplement to NDS provisions. Design provisions include seasoning effects and notching of structural members, mortise and tenon

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connections loaded in shear and tension, and lateral load carrying systems. Connection design provisions include yield limit equations, dowel bearing strength, peg diameter and bending yield strength, seasoning and creep effects, spacing requirements, adjustment factors, tenon size and quality, and mortise placement for pegged mortise and tenon joints loaded in tension [3]. Connection capacity is equal to the minimum design strength of one peg multiplied by the number of pegs in the connection [3].

3 METHODS AND MATERIALS

3.1 JOINT TESTING

Tensile load (including proportional limit, 5% offset and ultimate capacity) and stiffness of white oak (Quercus alba) and Douglas-fir (Pseudotsuga menziesii) joints with 10 cm and 28 cm long tenons with one and two keys were measured. Experimental results were compared to mathematical models developed from the NDS [1], the General Dowel Equations for Calculating Lateral Connection Values Technical Report 12 (TR-12) [3], and engineering mechanics.

Table 1 shows the test plan including test variables and number of specimens used in testing. Test variables included joint species (white oak or Douglas-fir), protruding tenon length (10 or 28 cm), and number of keys (one or two). An initial set of 40 joints were tested with white oak keys. Six of these joints with little to no tenon damage were retested with ipe (Tabebuia spp.) keys. Specimen labelling used English length units of the tenons – 10 cm (4 in) and 28 cm (11 in). Based on the information in Table 1, a typical specimen label was WO-4-2-1, or a white oak sample with a 10 cm tenon, 2 keys, and specimen number 1 of this group. After joint testing, specimens were cut from tested joints and additional key stock to generate mechanical properties for use in model predictions. Figure 2 shows the test setup used to load the keyed through-tenon joints in tension.

Table 1. Test Protocol of Joists with White Oak Keys

<table>
<thead>
<tr>
<th>Joint Species (Mortise and Tenon)</th>
<th>Tenon Length, cm (in)</th>
<th>Keys per Joint</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>White Oak</td>
<td>10 (4)</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>28 (11)</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>Douglas-fir</td>
<td>10 (4)</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>28 (11)</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>46</strong></td>
</tr>
</tbody>
</table>

Joint slip was taken as the average of the LVDT readings. Load-deformation data measured by the load cell and LVDTs was processed through MTS FlexTest 40 data acquisition software to generate load-deformation curves. Initial joint stiffness, proportional-limit load, 5% offset yield load, and ultimate load were recorded from the curves.

Joints were tested on an MTS Servo-Hydraulic Test Machine with a load cell with a range of 222 kN (50 kips) and an error less than 1% of the load. Two linear variable differential transducers (LVDTs), with a range of two inches and sensitivity of 0.025 mm (0.001 in), were attached at opposite sides of the joint to measure the tenon member slip relative to the mortise member.

Figure 2: Test setup for Keyed Through-Tenon Joints

Testing concluded when the load dropped at least 20% of the ultimate load with no sign of recovery or when wedging action of the keys occurred at the shear planes. Expected time to reach the maximum load was approximately ten minutes, ranging from five to 20 minutes based on ASTM D 5652 [5]. Maximum load for the joints with 28 cm tenons was obtained between five and 20 minutes, except for two Douglas-fir joints with 28 cm tenons with one key that displayed brittle failures. Joints with 10 cm tenons obtained maximum load in less than five minutes due to brittle failures, except for one white oak joint with a 10 cm tenon with one key and two white oak joints with two keys. All joints retested with ipe keys failed within the five to 20 minute time range. Joints within and out of the time range were treated identically for analysis. The displacement rate was initially 0.51 mm per minute and then increased to 0.76 mm per minute after the third joint, in an attempt to achieve the time range, which was used to accommodate the majority of the joints.

Proportional-limit load, 5% offset yield load, ultimate load, and stiffness were found. Proportional-limit load was found by scaling all curves and selecting the initial longest and straightest portion of each curve by eye. A set of parallel lines were drawn on either side of the selected curve portion and the first deviation from the lines was taken as the proportional-limit. This visual analysis was performed separately by two researchers.
Five percent offset yield load was found by offsetting a line, parallel to the initial linear portion of the load-deformation curve, 5% of the key width, and then finding the point of intersection with the load-deformation curve. If the 5% offset line intersected the load curve after ultimate load or not at all, the joint was not considered to have a 5% offset yield load. Ultimate load was the maximum load prior to a post-failure event, such as key wedging. Stiffness was the slope of initial linear portion of the load-deformation curve.

3.2 ULTIMATE LOAD PREDICTIONS

The ultimate joint load predictions from the models using the inputs from Shields [6] along with geometric measurements of the joints taken prior to testing are listed below. All equations are discussed in greater detail in [6]. The load prediction for any joint was the minimum value predicted by the individual models and was termed 'governing ultimate load' for ultimate load predictions and 'design load' for allowable load predictions. Governing ultimate and allowable predictions were consistent in the models for joints with four-inch tenons and key horizontal shearing (ZKv) in joints with 11-inch tenons. The models are shown with abbreviations including:

- Tenon Net-section Tension (at keyholes) (ZNT)
- Tenon Parallel-to-grain Keyhole Bearing (ZIm)
- Tenon Row Tear-out/ Tenon Relish (ZRT)
- Tenon Group Tear-out/ Block Shear (ZGT)
- Mortise Bearing (Zm)
- Key Bending (ZKv)
- Key Bearing (ZIm-K and ZIs-K)
- Key Horizontal Shear (ZKv)

4 RESULTS AND DISCUSSION

4.1 ULTIMATE LOAD EXPERIMENTAL RESULTS

The average ultimate load for joints with white oak keys including COV values and percent differences with respect to species, tenon length, and number of keys is shown in Table 2. The average ultimate load ranged from 14.8 kN to 69.4 kN. Two white oak joints with 28 cm tenons with two keys (WO-11-2-2 and WO-11-2-3) experienced key wedging which was considered a post-failure event. A deformation limit of 8.9 mm was established, to eliminate the selection of key wedging for ultimate load of these joints, based on similar deflection at ultimate load within the same joint group since distinguishing between ultimate load and key wedging was difficult between the two joints. The COV values ranged from 6.4% to 29.7%. Joints with four-inch tenons produced greater COV values (13.6% to 29.7%) than joints with 11-inch tenons (6.4% to 10.1%) for each joint species indicating that joints with key failures produce more consistent ultimate load values than joints with tenon failures.

Comparisons of average ultimate joint load between joint species, with similar tenon length and number of keys, showed white oak joints with greater ultimate load than Douglas-fir joints, other than joints with 28 cm tenons with two keys where Douglas-fir joints had 1.8% greater average ultimate load than white oak joints. Douglas-fir joints with 11 cm tenons with one key had 29% less average ultimate load than white oak joints. Douglas-fir joints with 11 cm tenons with two keys had 39% less average ultimate load than white oak joints. Douglas-fir joints with 28 cm tenons with one key had 8.3% less average ultimate load than white oak joints. The greatest differences between white oak and Douglas-fir joints occurred in joints with 11 cm tenons indicating that the ultimate load of joints with tenon failures are more dependent on joint member species than joints with key failures.

Joints with 28 cm tenons produced greater average ultimate load than joints with 10 cm tenons. White oak joints with 28 cm tenons were 66 and 27% greater in average ultimate load than white oak joints with 11 cm tenons, for joints with one and two keys respectively. Douglas-fir joints with 28 cm tenons produced more than twice the average ultimate load than Douglas-fir joints with 11 cm tenons, for joints with one and two keys. The smaller difference of average ultimate load between white oak joints with 11 cm and 28 cm tenons with two keys indicated that 11 cm white oak tenons with two keys were close to the balanced tenon length and key size that would cause simultaneous tenon and key failure for the species used.

The greatest differences in average ultimate load were between joints with one and two keys per species and tenon length. The least difference in average ultimate joint load, regarding the number of keys, was between the white oak joints with 28 cm tenons where the joints with two keys produced an average ultimate load nearly twice that of joints with one key. The greatest difference in average ultimate joint load, regarding the number of keys, was among the white oak joints with 11 cm tenons where the joints with two keys produced an average ultimate load 158% greater than with one key. Douglas-fir joints with two keys produced over twice the average ultimate load than Douglas-fir joints with one key, for both 11 cm and 28 cm tenons. Greater average ultimate load for joints with 11 cm tenons with two keys than with one key was likely due to the double number of tenon shear planes. Greater average ultimate load for joints with 28 cm tenons with two keys than with one was likely due to greater total key width which provided more key bending, bearing, and shear resistance.

4.2 DISCUSSION OF FAILURES

A brittle joint was defined by brittle behavior at ultimate load, such as tenon failure. Tenon failures included tenon splitting, tenon split spreading, single plane shearing, and relish failure. All joints with 11 cm tenons were brittle as well as one white oak and one Douglas-fir joint with 28 cm tenons with one ipe key (WO-11-1-4-IPE and DF-11-1-4-IPE), and two Douglas-fir joints with 28 cm tenons with one white oak key (DF-11-1-1 and DF-11-1-3). One Douglas-fir joint with a 28 cm tenon with one white oak key experienced a tenon split prior to a key bending and crushing failure due to an initial tenon defect (DF-11-1-2).
Table 2: Ultimate Load of Joints with White Oak Keys

<table>
<thead>
<tr>
<th>Species</th>
<th>Tenon Length, cm</th>
<th>Number of Keys</th>
<th>Average Ultimate Load, kN, (lbs) [COV,%]</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>WO</td>
<td>10</td>
<td>1</td>
<td>20.9 (4,700) [22.4]</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>53.8 (12,100) [13.6]</td>
<td>158</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>1</td>
<td>34.7 (7,810) [10.1]</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>68.5 (15,400) [6.4]</td>
<td>27 97</td>
</tr>
<tr>
<td>DF</td>
<td>10</td>
<td>1</td>
<td>14.8 (3,340) [27.4]</td>
<td>-29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>32.8 (7,370) [29.7]</td>
<td>121</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>1</td>
<td>31.8 (7,160) [8.9]</td>
<td>-8.3 115</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>69.4 (15,600) [8.0]</td>
<td>1.8 112 118</td>
</tr>
</tbody>
</table>

Figure 3 is a load-deformation plot of typical brittle joints and Figure 4 is a photograph of the corresponding failure. A relish failure occurred in a Douglas-fir joint with an 28 cm tenon with one key (DF-11-1-1). This brittle failure may have been due to a tenon split observed prior to testing.

Figure 3. Load-Displacement Plot showing Relish

Figure 4. Relish Failure of DF 11-1-1

The only 11 cm tenon joint that produced ductile behavior, but was still brittle in accordance with the definition above, was a white oak joint with two keys (WO-4-2-3) (Figures 5 and 6). This joint was termed brittle because a relish failure occurred at one of the keys, establishing ultimate load, prior to the intersection of the 5% offset line and the load-deformation curve. The other key experienced failure and the loading was stopped at approximately one-inch of deformation when the tenon split at the failed key.

Figure 5. Load-Displacement Plot showing Relish and Key Bending and Crushing

Figure 6. Key Bending and Crushing of WO-4-2-3

The load-deformation curves of a typical ductile joint is shown in Figure 7 with a photograph of the corresponding failure in Figure 8. Two criteria had to be met when defining a ductile joint: (1) a ductile behavior must occur at ultimate load, such as a key failure, and (2) the joint in consideration must have had a 5% offset line intersection with the load curve prior to ultimate load.
Most joints with 28 cm tenons were ductile by producing key failures. A good example of ductile behavior was a white oak joint with an 28 cm tenon with one key (WO-11-1-2) that showed a key bending/crushing failure that occurred after the intersection of the 5% offset line and load-deformation curve. Figure 8 is a photograph of a key bending and crushing failure that occurred in a Douglas-fir joint with an 28 cm tenon with two keys (DF-11-2-5).

**Figure 7. Load-Displacement Showing Key Bending and Crushing**

**Figure 8. Failure of Key Bending and Crushing**

Model were developed to predict the yield and capacity values for the following yield/failure modes: net section tension, parallel-to-grain bearing, tenon tear-out, tenon group tear-out, mortise bearing, key bending, key bearing and key horizontal shear. The models predicted row tear-out in the tenon for all joints with 10 cm long tenons, and key horizontal shear for all joints with 28 cm long tenons. Ratios of predicted versus experimentally tested ultimate joint loads (predicted/tested) or P/T ratios were used to compare the model results.

### 4.3 COMPARISON OF ULTIMATE LOAD

The average ultimate load predictions and COVs for each joint group and model for joints with white oak keys are shown in Table 3. Ultimate load predictions for the mortise and tenon members used matched specimens cut from the members. Ultimate load predictions for keys used the average strength of the additional key stock due to the limited size of the keys. Governing load predictions are bold-faced and shaded for easy identification. White oak joints are listed as "WO" and Douglas-fir joints are listed as "DF." The first number after species is the protruding tenon length, in inches, and the second number represents the number of keys per joint in a group.

Predicted ultimate joint load for all white oak and Douglas-fir joints with four-inch tenons was governed by tenon relish failure ($Z_{RT}$). Average governing ultimate joint load was 32.9 kN (10.5% COV) and 68.5 kN (12.1% COV) for white oak joints with 11 cm tenons with one and two keys, respectively, and 28.0 kN (6.7% COV) and 54.3 kN (9.3% COV) for Douglas-fir joints with 11 cm tenons with one and two keys, respectively. Average governing ultimate load of white oak and Douglas-fir joints with 28 cm tenons was governed by horizontal key shearing ($Z_{Kv}$). Average governing ultimate joint load was 73.0 kN (6.5% COV) and 107 kN (2.4% COV) for white oak joints with 28 cm tenons with one and two keys, respectively, and 61.0 kN (3.5% COV) and 85.9 kN (2.9% COV) for Douglas-fir joints with 28 cm tenons with one and two keys, respectively. Average governing ultimate load of white oak and Douglas-fir joints with 28 cm tenons with two keys was approximately one-and-a-half that of the same joints with one key since tenons with two keys had twice the number of tenon shear planes than tenons with one key, where tenon relish failure assumed simultaneous failures of each shear plane. White oak joints were predicted to have greater ultimate load than the Douglas-fir joints, considering tenon relish failure, due to the higher parallel-to-grain shear strength of white oak than Douglas-fir.

Predicted ultimate joint load for all white oak and Douglas-fir joints with 28 cm tenons was governed by horizontal key shearing ($Z_{Kv}$). Average governing ultimate joint load was 73.0 kN (6.5% COV) and 107 kN (2.4% COV) for white oak joints with 28 cm tenons with one and two keys, respectively, and 61.0 kN (3.5% COV) and 85.9 kN (2.9% COV) for Douglas-fir joints with 28 cm tenons with one and two keys, respectively. Average governing ultimate load of white oak and Douglas-fir joints with 28 cm tenons with two keys was approximately one-and-a-half that of the same joints with one key since keys in joints with two keys were 38.1 mm wide, totaling a width of 76.2 mm, and keys in joints with one key were 50.8 mm wide. White oak joints were predicted to have greater ultimate load than the Douglas-fir joints, considering horizontal key shear, due to the higher perpendicular-to-grain bearing strength of white oak mortise members than Douglas-fir mortise members.

Ultimate joint load prediction COV values are dependent upon COV values from material input tests which explains the high COV values of the tenon net-section tension results ($Z_{NT}$) which were calculated using the tension specimens with high COV values. Tension specimens of white oak joints with 11-inch tenons with two keys had greater net-section tension strength than the same joints with four-inch tenons, while both groups had the same tenon net-section area. Measured geometries of the joints had a small influence on the COV values due to dimensional tolerances. According to ultimate load predictions, white oak and Douglas-fir joints with four-inch tenons were governed by tenon relish failure with no key failures, of the size
Table 4-13: Average Predicted Ultimate Joint Load with White Oak Keys

<table>
<thead>
<tr>
<th>Model</th>
<th>WO-4-1</th>
<th>WO-4-2</th>
<th>WO-11-1</th>
<th>WO-11-2</th>
<th>DF-4-1</th>
<th>DF-4-2</th>
<th>DF-11-1</th>
<th>DF-11-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Z_{NT}$</td>
<td>774 (31.9%)</td>
<td>363 (18.4%)</td>
<td>773 (33.0%)</td>
<td>746 (24.1%)</td>
<td>736 (25.6%)</td>
<td>534 (30.5%)</td>
<td>647 (33.6%)</td>
<td>443 (15.4%)</td>
</tr>
<tr>
<td>$Z_{lim}$</td>
<td>94.7 (12.7%)</td>
<td>140.1 (7.9%)</td>
<td>106 (8.5%)</td>
<td>138 (11.8%)</td>
<td>102 (2.4%)</td>
<td>186 (21.1%)</td>
<td>127 (12.5%)</td>
<td>143 (13.1%)</td>
</tr>
<tr>
<td>$Z_{RT}$</td>
<td>32.9 (10.5%)</td>
<td>68.5 (12.1%)</td>
<td>136 (6.4%)</td>
<td>263 (13.1%)</td>
<td>28.0 (6.7%)</td>
<td>54.3 (9.3%)</td>
<td>103 (18.6%)</td>
<td>230 (9.4%)</td>
</tr>
<tr>
<td>$Z_{GT}$ (a)</td>
<td>403 (30.8%)</td>
<td>N/A</td>
<td>454 (28.3%)</td>
<td>N/A</td>
<td>382 (24.7%)</td>
<td>N/A</td>
<td>375 (29.9%)</td>
<td>N/A</td>
</tr>
<tr>
<td>$Z_{GT}$ (b)</td>
<td>N/A</td>
<td>149 (15.1%)</td>
<td>N/A</td>
<td>366 (17.7%)</td>
<td>N/A</td>
<td>198 (26.1%)</td>
<td>N/A</td>
<td>253 (9.7%)</td>
</tr>
<tr>
<td>$Z_{GT}$ (c)</td>
<td>N/A</td>
<td>283 (16.6%)</td>
<td>N/A</td>
<td>643 (20.3%)</td>
<td>N/A</td>
<td>390 (27.9%)</td>
<td>N/A</td>
<td>420 (11.3%)</td>
</tr>
<tr>
<td>$Z_{GT}$ (d)</td>
<td>N/A</td>
<td>256 (17.4%)</td>
<td>N/A</td>
<td>556 (21.8%)</td>
<td>N/A</td>
<td>365 (29.3%)</td>
<td>N/A</td>
<td>348 (13.0%)</td>
</tr>
<tr>
<td>$Z_{Ic}$</td>
<td>220 (5.5%)</td>
<td>380 (17.6%)</td>
<td>245 (19.6%)</td>
<td>383 (21.8%)</td>
<td>132 (29.8%)</td>
<td>161 (7.9%)</td>
<td>137 (12.2%)</td>
<td>172 (11.5%)</td>
</tr>
<tr>
<td>$Z_{Ii,m,T}$</td>
<td>77.8 (3.5%)</td>
<td>117 (2.4%)</td>
<td>82.3 (8.8%)</td>
<td>118 (4.8%)</td>
<td>65.9 (9.4%)</td>
<td>95.7 (2.6%)</td>
<td>68.1 (5.6%)</td>
<td>93.9 (3.1%)</td>
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<tr>
<td>$Z_{Ii,K}$</td>
<td>77.4 (2.8%)</td>
<td>116 (3.0%)</td>
<td>79.2 (6.9%)</td>
<td>116 (2.0%)</td>
<td>65.0 (9.0%)</td>
<td>92.6 (2.5%)</td>
<td>65.0 (3.7%)</td>
<td>93.0 (2.5%)</td>
</tr>
<tr>
<td>$Z_{Ii-K}$</td>
<td>96.5 (1.9%)</td>
<td>144 (0.2%)</td>
<td>97.0 (1.2%)</td>
<td>144 (1.8%)</td>
<td>95.7 (0.4%)</td>
<td>142 (1.2%)</td>
<td>96.1 (0.3%)</td>
<td>143 (0.4%)</td>
</tr>
<tr>
<td>$Z_{Ii-K}$</td>
<td>260 (0.9%)</td>
<td>384 (0.7%)</td>
<td>261 (1.5%)</td>
<td>384 (1.5%)</td>
<td>254 (0.5%)</td>
<td>380 (0.8%)</td>
<td>255 (0.3%)</td>
<td>379 (0.8%)</td>
</tr>
<tr>
<td>$Z_{Kc}$</td>
<td>712 (3.2%)</td>
<td>107 (2.6%)</td>
<td>72.9 (6.5%)</td>
<td>107 (2.4%)</td>
<td>59.6 (8.8%)</td>
<td>85.4 (2.6%)</td>
<td>61.0 (3.5%)</td>
<td>85.9 (2.9%)</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

This paper demonstrated the tension capacity of keyed through-tenon joints used in timber frame construction. After testing a variety of joints, ductile failures were observed in connections with long tenons (28 cm), which are recommended for use. Prediction of ductile failures focused on key bending properties and produced good estimates to predict strength of these connections. This work will allow the improved design of timber frame connections in the future.

High values for tenon net-section tension at keyholes ($Z_{NT}$), tenon group tear-out ($Z_{GT}$), and bearing predictions were due to high bearing strength (considering maximum bearing strength within 12.7 mm of deformation) and high tension strengths from tension tests. Tension strength was a factor in tenon net-section tension at keyholes ($Z_{NT}$) and tenon group tear-out ($Z_{GT}$) which produced the highest predicted ultimate loads, except for the group of white oak joints with 11 cm tenons with two white oak keys, where key bearing strength against the mortise member was the greatest prediction. Load optimization (balancing) of these joints can be achieved by increasing the number of keys (greater number of shear planes) and total bearing area on the keys (increased key width), while reducing tenon net-section area. Increasing total key bearing area (width), for a given tenon thickness and key depth, would increase key bearing, bending, and shear strength since the combined key width would increase. Increased number of tenon shear planes (greater shear area) may also allow a reduction in tenon length, if individual key width is also reduced to avoid increasing key resistance relative to relish resistance, constructing a brittle joint.

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EFFECTS OF SIZE RATIOS ON DOVETAIL JOINTS IN KOREAN TRADITIONAL WOODEN BUILDING

Sung-Jun Pang 1, Jung-Kwon Oh 2, Chun-Young Park 3, Se-Jong Kim 4, Jun-Jae Lee5

ABSTRACT: In this study, the moment resistance was investigated according to the size ratio of dovetail joints and the similitude was checked among the different sizes of joints. The average maximum and yield moment resistance was increased as the scale ratio was increased. However, when both of the moment values were divided by the cube of scale ratio, the values were similar among the different scale ratio joints. It means that the moment resistance conformed to the similitude theory.

KEYWORDS: Size Ratio, Dovetail Joint, Moment-carrying Capacity, Traditional Post-beam Joint

1 INTRODUCTION

Dovetail joints are commonly used at post-beam joints in traditional Korean wooden buildings constructed with solid sawn timber. It is necessary to identify factors that have significant effects on the performance of dovetail joints. Testing full-scale models is necessary to develop rational methods for structural analysis and design of joints, but it is very complicated and costly. Therefore, developing an innovative small-scale modeling method and testing small-scale models using similitude for predicting the behavior of the full-scale model should be considered. In this study, the moment resistance was investigated according to the size ratio of dovetail joints and the similitude was checked among the different sizes of joints.

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2 Materials and Methods

2.1 Specimens

To determine the similitude on moment-carrying capacity in dovetail post-beam joints, three groups of dovetail joints (Type I, Type II, and Type III) with three replications each were manufactured in three different sizes, as shown in Figure 2. The species used was Japanese cedar (Cryptomeria japonica) in green condition (density: 270kg/m³, moisture content: 30%-40%), and the shape of the post, beam, and Dori were uniformly controlled. The dimensions among the three groups were controlled using scale ratios, as shown in Figure 3. The scale ratios for the three groups were 1:1.25:1.5, such that Type II and Type III specimens were 1.25 and 1.5 times larger, respectively, than Type I specimens. The configurations of the specimens are shown in Figure 3, and the unknown dimensions (A, B, C, and D) are explained in Table 1. For all specimens, the dimensions
B, C, and D were 1.5, 3, and 4 times larger, respectively, than the dimension of A. Thus, the shapes and scale ratios were uniformly controlled among the three groups.

<table>
<thead>
<tr>
<th>Type</th>
<th>Dimension (mm)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Type I</td>
<td>40</td>
<td>60</td>
</tr>
<tr>
<td>Type II</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>Type III</td>
<td>60</td>
<td>90</td>
</tr>
</tbody>
</table>

2.3 Measurement of internal joint rotation

When the joint rotation occurred at the position between the post and either the center or the bottom of the Dori, the real displacement of LVDT (X₁) was X₁, as shown in Fig. 5, and it was the same for X₁’. The real displacement of LVDT (X₂) was X₂, and the distance between LVDT (X₁) and LVDT (X₂) was 15+d+15. Therefore, the internal joint rotation angle (θ) was calculated as the displacement of both LVDT (X₁) and LVDT (X₂) and the distance between them. Using the geometric relationship between the joint rotation angle and the displacements shown in Fig. 5, Eq. (1) can be obtained:

\[ \theta = \arctan \left( \frac{X_1 + X_2}{15 + d + 15} \right) \]  

where

- X₁ = the displacement of LVDT(X₁) measured on the top side of the Dori (mm)
- X₂ = the displacement of LVDT(X₂) measured on the bottom side of the Dori (mm)
- 15+d+15 = the distance between the two LVDTs (mm)
- the depth of the Dori is d

Figure 5: Geometric relationship between the joint rotation angle and the displacements: (a) when the joint rotation is at the center of the Dori, and (b) when the joint rotation is at the bottom of the Dori.
rotation occurred at the position between the post and the center of the Dori; and (b) when the joint rotation occurred at the position between the post and the bottom of the Dori.

2.4 The similitude and dimensional analysis

In a small-scale model test for a full-scale structure, if the variables involved in moment \((M)\), length \((L)\), and modulus of elasticity \((E)\), using the dimensionless parameter for \(M\) can be found with the following equation; Where, \(f\) is the full-scale structure, \(s\) is the small-scale model.

\[
\frac{M}{E'L'} = \left(\frac{M}{E'L'}\right)_s
\]

\[
\frac{M_s}{M_f} = \left(\frac{L_s}{L_f}\right)^3
\]

Therefore, it can be concluded that the moment of the small-scale model is the moment of full-scale structure divided by the triple ratio.

3 Results and discussions

In the experimental test results, the larger specimens demonstrated greater moment-carrying capacity (Fig. 6 and Table 2), and all of the specimens had a rotation capacity consistent with the deformations. Thus, this type of joints has ductility capacity and could be considered as a semi-rigid joint. Meanwhile, the failure mode of the Type I specimens differed from that of the Type II and Type III specimens (Fig. 7). Because of the different failure modes, it was difficult to compare Type I specimens with Type II and Type III specimens, but it was possible to compare between Types II and III.

The average maximum moment \((M_{\text{max}})\) of the joint types increased in relation to an increase in the scale ratio. However, \(M_{\text{max}}\) divided by the cube of the scale ratio was similar among the different scale joints (Fig. 8). This relationship was also present in the graph of the joint types and the average yield moment (Fig. 9), and it indicates that the moment resistance conformed to similitude theory.

Table 2. Summary of the test results for the various size ratios

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>(M_{\text{max}}) (kN・m)</th>
<th>(M_y) (kN・m)</th>
<th>(R_s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>1.25</td>
<td>0.70</td>
<td>0.40</td>
</tr>
<tr>
<td>Type II</td>
<td>1.5</td>
<td>0.90</td>
<td>0.45</td>
</tr>
<tr>
<td>Type III</td>
<td>1.5</td>
<td>0.90</td>
<td>0.45</td>
</tr>
</tbody>
</table>

\(M_{\text{max}}\), maximum moment; \(M_y\), yield moment; \(R_s\), scale ratio of each specimen in relation to the Type I specimens; \(\text{COV}\) (%), coefficient of variation.

![Figure 6: Moment-rotation curves](image)

![Figure 7: Failure modes of the dovetail joints in the different scale specimens: (a) Split failure that occurred at the post; (b) and (c) Split failure that occurred at the beam](image)
4 Conclusions

The average maximum and yield moment resistance was increased as the scale ratio was increased. However, when both of the moment values were divided by the cube of scale ratio, the values were similar among the different scale ratio joints. It means that the moment resistance conformed to the similitude theory.

REFERENCES


COMPRESSIVE BEHAVIOUR OF WOOD-BASED FIBERBOARDS

H. Tran1*, C. Delisée1, P. Doumalin2, J. C. Dupre2, A. Germaneau2, J. Malvestio1

ABSTRACT: The compressive behaviour of wood-based fibrous insulators depends significantly on their complex microstructure, which requires a multi-scale approach in order to have a full comprehension. This approach, based on X-ray microtomography combined with Digital Volume Correlation method, allows us not only to extract a large amount of intrinsic characteristics (fiber diameter, porosity etc.) but also confront these data to heterogeneous local strain field. Van Wyk’s experimental model is proposed to validate macroscopic law of material.

KEYWORDS: Wood fibers, heterogeneity, compression test, non-destructive imaging, Digital Volume Correlation.

1 INTRODUCTION

Nonwoven wood/thermoplastic fiberboards have potential applications as thermal and acoustic insulators and as non-polluted and recyclable materials due to their good environment impact. During their life (transport, storage, handling…), these materials are compressed at different levels, the following question is raised: until which compression ratio the material can return completely to its initial shape after the unloading and preserve all its mechanical and thermal characteristics? This material usually exhibits a complex and time-dependant mechanical behaviour, characterized by a nonlinear stress-strain curve with residual deformation and hysteresis, due to its complex microstructure. Several numerical and experimental studies usually based on Van Wyk’s model [1, 2, 3], have devoted to describe compressive response of fibrous materials through fiber-to-fiber contacts, mean distance between contacts, friction between fibers, while others employ the discrete simulation, originated from molecular dynamics techniques [4, 5].

We present here a new multi-scale methodological approach which allows us to characterize the mechanic behaviour at different scales of the structure through the unidirectional compression tests. The global behaviour is determined at the scale of the structure. The local response is studied from strain field measuring by Digital Volume Correlation in the bulk of material by X-ray microtomography.

2 STUDIED MATERIAL

The material sample is extracted from a low density fiberboard, containing maritime pine fibers and bi-components thermo-fused polyester fibers. The cohesion is assured by the thermo-bonding of polyester fibers. The combination between a family of variable isolated wood fibers and fiber bundles and a family of textile fibers (mean diameter of 20μm and mean length of 55mm) creates a 3D random assembly (Figure 1) with low density (45kg.m⁻³) and very high porosity (>95%).

Figure 1: SEM image of Wood/PES fiberboard (940μm)

3 METHODS

3.1 UNIAXIAL COMPRESSION TEST

In macroscopic test, the 100x100x65mm³ samples are subjected with or without confinement to compression-release cycles at the same displacement rate 3mm/min. Some samples are totally compressed. Several cycles are then imposed for the other samples at different level of strain 20%, 25%, 30%, 35%, 40% and three cycles per level.
In microscopic test, a cylindrical sample, 9mm of thickness and 9mm of diameter is compressed at strain rate from 5% to 80% under X-ray microtomography (Nanotom Phoenix X-ray). The sample is constrained in PMMA tube guaranteeing macroscopic 1D displacement without giving any extra lateral load on the sample. The load is transferred perpendicularly to the surface of the sample through a Teflon support which is used to limit friction with the tube. The microtomographic scans are obtained for each compression states after the applied force becomes stable.

![Figure 2: Microtomographic test set-up](image)

### 3.2 DEFORMATION OBSERVATION

For macroscopic tests, a camera allows to observe the evolution of the deformation on the sample surface during the compression tests. For 3D microscopic tests, high-resolution X-Ray microtomography (6μm/voxel) is employed to rebuild in grey levels (Figure ) the different phases (fiber/porosity) of sample microstructure at each compression step [6, 7].

![Figure 3: Microtomographic image of a slice and crop of the zone of interest](image)

### 3.3 FULL-FIELD STRAIN MEASUREMENT

In this framework, digital volume correlation is performed on microtomographic images between two consecutive images or two consecutive compression states in order to obtain 3D full field strain of sample. This technique consists in finding, in the deformed image, a small domain that is the most similar to the chosen domain in the reference image, with the hypothesis that the strain is homogeneous in this domain (Figure ), which is a set of adjacent voxels for 3D images. Correlation process gives a discrete displacement field.

![Figure 4: Principle of Digital Volume Correlation](image)

**Considering the initial configuration** \( \mathbf{X}(X,Y,Z) \) **in the current configuration** \( \mathbf{x}(X,Y,Z) \) **the deformation gradient tensor is describes as:**

\[
\mathbf{F} = \frac{\partial \mathbf{x}}{\partial \mathbf{X}}
\]

**Green-Lagrange strain field is calculated by finite differences as** [8, 9]:

\[
\varepsilon = \frac{1}{2} \left( \mathbf{F}^T \mathbf{F} - \mathbf{I} \right)
\]

In this study, DVC is performed on a grid of 4950 points. The distance between grid points is 60 pixels and the initial size of correlated domain is 90×90×90 voxels.

### 4 RESULTS AND DISCUSSION

#### 4.1 MACROSCOPIC BEHAVIOUR

Figure 4 presents the relation between the stress and the engineering strain rate in destructive test. The stress increases slowly within 50% of strain level before increasing quite suddenly and reaching the maximum strain level of 85% at 4200N of applied force. The behaviour of material can be distinguished into two phases: elastic phase until 50% of compression and non-elastic phase. The Young modulus is found respectively 0.2MPa and 4MPa for elastic and non-elastic phase. One may be assumed that because of the important existence of porosity, the applied force is firstly used to push these porosities out. Next, the more the fibers receive the pressure, the more the applied force increases. So we can suppose that when the load reaches the maximum, there is no porosity in the sample, as the internal porosities do not exist anymore because of the collapse phenomenon.

![Figure 5: Load-Strain curve of destructive test](image)
residual strain and hysteresis are detected for all the cycles, most likely due to the rearrangement, the densification of the fiber mat and the friction at fiber contacts. Although the curves change from loading, unloading and reloading, they become stable after the first loading curve which can be explained by the influence of compression machine on the first loading and this influence is then self-adjusted for successive cycles. The hysteresis is dependent on the maximum load reached.

\[ \varepsilon = \ln \left( \frac{e}{e_0} \right) ; \quad \sigma = \frac{F}{S} \]  

(3)

where \( e \) is the sample length, \( e_0 \) the initial sample length and \( F \) the applied load on the section \( S \) of the sample. The global experiment results show a good agreement with Van Wyk’s experimental model (Figure 6) where the stress is related to the strain following Equation (2):

\[ \sigma = a \exp(b\varepsilon) \]  

(4)

Figure 6 gives a comparison between experiment data and Van Wyk’s model which is well-established for mechanical behaviour of random fiber network. The results are fitted better to Van Wyk’s curve with \( a = 1500 \text{Pa} \) and \( b = -3 \). The fact that \( b = -3 \) indicates clearly that the wood/thermoplastic fiberboard, in spite of its different populations of fibers, follows well the macroscopic law for 3D random assemblies in compression.

4.2 FULL FIELD STRAIN

Figure 7 presents the total Ezz strain field in the loading direction at different compression states.
At microscopic scale, the material exhibits a strong heterogeneous local strain due to not only the porosity distribution but also the arrangement of fibers in the material. This heterogeneity decreases with the compression rate as shown Figure 8 which gives histograms of local strains.

![Figure 8: Histogram of local strain at different compression states](image)

The 3D strain field is superposed with the microstructure (Figure 9). As we can observe, the more porous the zone is, the more it is compressed.

![Figure 9: 3D strain field in the compression direction](image)

Van Wyk’s experimental model, originally developed for wool fibers seems to be also applicable for this kind of material at macroscopic scale.

### REFERENCES

THE USE OF ARTIFICIAL NEURAL NETWORKS IN TIMBER GRADING

Nataša Zavrtanik¹, Jan Dobnikar², Goran Turk³

ABSTRACT: Structural material should have satisfactory strength that enables a proper load carrying capacity. To determine the strength characteristics of wooden elements, non-destructive experiments are used to estimate strength properties which can realistically be determined by destructive experiments only. Artificial neural networks have been used in this work to estimate the strength characteristics of structural timber by two different approaches, via approximation and by direct classification.

KEYWORDS: artificial neural networks, strength of structural timber, grade determining parameters, indicating properties

1 INTRODUCTION

The main problem in strength grading of wooden elements is the fact that strength can only be measured by a destructive test. Therefore, the results of non-destructive tests, which are more or less correlated to the strength, have to be used. Usually a linear or exponential relationship is presumed. However there is no theoretical background to support either of these assumptions. An alternative approach of using artificial neural network is presented here.

Several thousands wooden specimens were taken into consideration. For all specimens the density, three elastic modules, strength, dimensions of specimens, weight, moisture content, total KAR and some grading machine indicating properties were measured. Eight out of ten of specimens were randomly chosen for the training of artificial networks and the remaining were left for testing procedure. Several examples, in which different input and output data is used, were taken into consideration. For generating and learning of neural networks we’ve used our own fortran code NTR2003 as well as publically available software WEKA Toolkit[1]. The efficiency of various neural networks is compared through several statistical quantities and also the effect of data dissipation is considered.

2 LITERATURE REVIEW

There are many indications of the potential use of artificial neural networks in approximation and classification problems. A synthesis of the published researches in the area of the use of neural networks in the field of classification is presented in [2]. In more general terms, a review of the use of artificial neural networks in several engineering applications is given in [3].


Researchers in [5] used feed forward multilayer networks for obtaining the modulus of elasticity of Abies pinsapo Boiss from the data of test specimen dimension, moisture content, density and ultrasonic wave propagation velocity. The classification of wood species by neural network has been performed and published in [6], similar work gas been presented in [7].

An interesting contribution on the use of ANN in automated visual inspection was published in [8].

The use of ANN in timber strength estimation was given in [9]. The prediction of fracture toughness of pine samples by ANN was done by [10]. An attempt to timber strength grading by ANN on oak was presented in one of the earlier WCTE conferences [11], too.

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3 FEED-FORWARD NEURAL NETWORKS

Artificial neural networks are smaller and less complicated than biological networks. Components of artificial networks are neurons which are connected with connections described by their weights (Figure 1). The particularity of artificial networks is that we don't program them but we train them. Training of networks needs two parts of data. The first part includes training sets of data and the second part of data is testing sets, which is needed to establish how effective the training is. In the following one specific type of neural networks is presented and used, i.e. feed-forward neural network or multi-layer perceptron.

The scheme of artificial neural networks is shown in Figure 1. For example: neuron $u_i$ is connected with a few neurons, which send it their output signal $o_i$. Output signals are multiplied by weights $w_{ij}$. A threshold signal $b_i$ is added to the sum of weighted signals. This gives the value of the signal of neuron $u_i$. Then the output function $f$ copies the value to the output signal $o_i$ of neuron $u_i$. Connections between neurons and output function may be regulated in advance.

![Figure 1: Scheme of operation of artificial neural networks](image)

The symbols in Fig. 1 have the following meaning:

- $u_i$ . . . neuron $i$,
- $u_j$ . . . neuron $j$,
- $u_k$ . . . neuron $k$,
- $u_l$ . . . neuron $l$,
- $w_{ij}$ . . . weight of interaction between neurons $i$ and $j$,
- $w_{ik}$ . . . weight of interaction between neurons $i$ and $k$,
- $w_{il}$ . . . weight of interaction between neurons $i$ and $l$,
- $b_i$ . . . threshold of neuron $i$,
- $f_i$ . . . output function,
- $o_i$ . . . output signal of neuron $i$,
- $o_j$ . . . output signal of neuron $j$,
- $o_k$ . . . output signal of neuron $k$,
- $o_l$ . . . output signal of neuron $l$.

An important phase of neural network analysis is determining the weights, which are determined during training of network. There are several methods, but the most spread is generalized delta rule or error back-propagation algorithm.

3.1 DESCRIPTION OF PARAMETERS THAT AFFECT TRAINING OF ARTIFICIAL NEURAL NETWORKS

Maximum permitted error is the maximum difference between target and with neural networks calculated output values of data.

Learning step size

In general, smaller learning step requires a large number of iterations and vice versa. However, there is a possibility that neural networks with a large learning step don't learn well, because they don't find the minimum error.

Momentum

This parameter determines the influence of previous iteration on the present changes of weights. The use of momentum helps to avoid local minima.

Maximum number of iterations

Maximum number of iterations limits the training of neural networks.

Network geometry refers to the number of input and output neurons, to the number of hidden layers and to the number of neurons in each hidden layer. The optimal geometry of networks is usually not known in advance.

Output functions copy value of output signal. There exist different sorts of output functions like linear function, sigmoid function, tangential sigmoid function, exponential output function, etc.

4 REGRESSION TREES

When all non-destructive testing data is acquired, timber grading can also be performed by the use of algorithms that generate decision trees. This algorithm divides the population into two parts by choosing a certain dividing value of one of the input parameters. This value is a criterion, which divides the population to specimens with lower, and to those with higher value. It is chosen propositionally in such a way, that each of the two parts of the population can be further divided into two parts that are as different as possible. Each parameter can be used as a criteria more than one time. Each leaf contains a value of the output parameter, as well as the number of training instances that matched the criteria of that leaf. The difference between the value on the leaf and the value of the output parameter of the specimens on that leaf is a base for the evaluation of correlation.

If the dividing process is never stopped, the number of leaves match the number of specimens. Division can be stopped by limiting the minimum number of instances on a leaf, defining the minimum variance of the parameter on a leaf, or by limiting the depth of the tree. After the tree is generated, pruning can be performed on the branches which don’t improve the accuracy to the model.
5 TWO APPROACHES TO THE USE OF ANN IN TIMBER GRADING

There are two possible approaches with the use of the artificial neural networks in timber grading: approximation of grade determining parameters and classification. In the first approach artificial neural networks are used for approximation of grade determining properties with respect to indicating properties. Indicating properties obtained by ANN is then used in an ordinary way of timber grading. In the second approach neural networks are used for direct timber classification into strength grades on the base of indicating properties. In this case the optimal grades are used as output parameter and neural network needs to directly learn the right or optimal grades.

6 RESULTS

Several different combinations of input data have been tried. Linear regression has been used in all combinations, too. The analysis of pine and spruce loaded in bending and tension from ten European countries has been done. The experiments were performed in the framework of international project Gradewood [12]. The combined sample included 5302 specimens. The population was divided in four parts as shown in Table 1.

<table>
<thead>
<tr>
<th>Loading type</th>
<th>Pine</th>
<th>Spruce</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>1085</td>
<td>1478</td>
</tr>
<tr>
<td>Bending</td>
<td>428</td>
<td>2311</td>
</tr>
</tbody>
</table>

The training of each subsample has been performed separately. The subsample has been divided in two sets: training/validation set and test set. Approximately 20 % of specimens have been assigned to testing set, the remaining 80 % were used in training and validation. This set was further divided into training and validation sets. The method of cross-validation has been used to optimally determine the ANN parameters such as stopping criteria, geometry of neural networks, etc. Since the comparison between the different subsamples shows similar results, only the results of subsample for spruce in bending is shown here. In Tables 2 and 3 the efficiency of different ANN for global MOE and strength is presented. The comparison with the linear regression is given, too. These results show that the ANN give slightly better approximation than linear approximation, since the coefficient of determination $R^2$ is higher for all variations of ANN modeling whereas the average relative error is lower.

<table>
<thead>
<tr>
<th>Case</th>
<th>Step size</th>
<th>Mom. Number of iter.</th>
<th>Average rel. error</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2</td>
<td>0.3</td>
<td>1000</td>
<td>5.7%</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.4</td>
<td>1000</td>
<td>5.7%</td>
</tr>
<tr>
<td>3</td>
<td>0.15</td>
<td>0.2</td>
<td>1500</td>
<td>5.4%</td>
</tr>
<tr>
<td>4</td>
<td>0.15</td>
<td>0.2</td>
<td>300</td>
<td>5.9%</td>
</tr>
<tr>
<td>Linear regression</td>
<td></td>
<td></td>
<td></td>
<td>6.3%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case</th>
<th>Step size</th>
<th>Mom. Number of iter.</th>
<th>Average rel. error</th>
<th>$R^2$</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2</td>
<td>0.30</td>
<td>1000</td>
<td>16.3%</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>0.30</td>
<td>300</td>
<td>16.3%</td>
</tr>
<tr>
<td>3</td>
<td>0.4</td>
<td>0.30</td>
<td>300</td>
<td>18.4%</td>
</tr>
<tr>
<td>4</td>
<td>0.45</td>
<td>0.30</td>
<td>300</td>
<td>16.3%</td>
</tr>
<tr>
<td>Linear regression</td>
<td></td>
<td></td>
<td></td>
<td>16.8%</td>
</tr>
</tbody>
</table>

In Tables 4 and 5 the results of regression trees are given. The comparison with ANN and linear regression shows that regression trees are considerably less efficient in modeling the strength. The average relative error is consistently higher whereas the coefficient of determination is lower.

<table>
<thead>
<tr>
<th>Case</th>
<th>Max. depth</th>
<th>Min. number on leaf</th>
<th>Tree size</th>
<th>Average rel. error</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>501</td>
<td>8.8%</td>
<td>0.672</td>
</tr>
<tr>
<td>2</td>
<td>-1</td>
<td>1</td>
<td>41</td>
<td>9.0%</td>
<td>0.674</td>
</tr>
<tr>
<td>3</td>
<td>-1</td>
<td>20</td>
<td>29</td>
<td>9.2%</td>
<td>0.658</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>20</td>
<td>13</td>
<td>10.4%</td>
<td>0.569</td>
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<table>
<thead>
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<th>Case</th>
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<td>95</td>
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<td>0.453</td>
</tr>
<tr>
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<tr>
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<tr>
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<td>3</td>
<td>30</td>
<td>11</td>
<td>19.4%</td>
<td>0.423</td>
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</table>

7 CONCLUSIONS

The global MOE and strength were modeled by three methods: traditional linear regression, feed forward artificial neural network and regression trees. The results
obtained by all three approaches are given and compared for the case of spruce in bending. It is shown that feed forward ANN give slightly better results than linear regression. Better approximation of grade determining parameters may lead to more accurate machine settings which will be expressed in higher yield of higher strength classes as well as more reliable grading. Unfortunately, the regression trees are less efficient. It is our aim to try regression trees with another approach in which we will not approximate the grade determining properties, but directly use the regression trees for the determination of grades. The hypothesis that such use of regression trees can be very efficient will be tested in future.

ACKNOWLEDGEMENT

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THE EFFECT OF Changing AMBIENT HUMIDITY ON MOISTURE CONDITION IN TIMBER ELEMENTS

Tomaž Hozjan¹, Goran Turk², Stanislav Srpčič³, Staffan Svensson⁴

ABSTRACT: This paper deals with the effect of the changing ambient humidity on moisture conditions in timber elements. The naturally varying humidity is possible to model as a relative combination of different harmonic cycles, with different periods and amplitudes. For the determination of the moisture field a fully coupled transport model including a model for the influential sorption hysteresis of wood is used. The coupled model accounts for both vapor transport in pores and bound water transport in wood tissue. Moisture state history influences relationship between moisture state of wood and air humidity, it must therefore be taken into account. In order to include history dependency, a hysteresis model is used here. Results from numerical calculations for timber specimen exposed to combined daily and annually cyclic variation of outside humidity are presented.

KEYWORDS: Coupled moisture transport, Sorption hysteresis, Varying natural humidity conditions

1 INTRODUCTION

Wood is a hygroscopic material, which means it naturally absorbs and desorbs water to balance the potential of surrounding environment. Absorption of water in wood occurs when the relative humidity is less than the equivalent timber relative humidity. During the natural cycle of changing humidity the changes of outside temperature may increase or decrease the moisture transfer in the wood, too. Natural cycle of changing the moisture content causes timber elements to swell and shrink. In timber elements this effect causes additional stresses and strains and thus influences the whole stress-strain field in wood elements. As it is known these additional stresses cannot be neglected in a design of timber elements, especially if one considers the bearing elements of structures such as columns or beams. Therefore the moisture content in timber structures is essential.

The constantly varying outdoor and indoor conditions show typically two cycles, daily and annual. One way to model the naturally varying climate is therefore to superimpose two harmonic cycles, one with the daily period and one with the annual. The initial moisture condition in timber member is very influential parameter for predictions of moisture state. In the present study the harmonic process is presented.

In order to predict the moisture state of a volume of seasoned wood in natural varying humidity an efficient moisture transport model has to be developed. In the literature few such models can be found. In general, in so-called multiphase models for describing the transfer of water in capillary porous materials each of the substances is taken into account. For the modelling of moisture state in wood two-phase models are most commonly used. This model describes the combined transport of separate water vapour and bound water diffusion and their interaction through sorption. At relative low humidity, moisture transport is governed by water-vapor diffusion where bound-water diffusion is relatively slow. However, at higher relative humidity bound-water diffusion becomes more significant and causes the process of sorption to slow down. In this model, the moisture content depends on so called sorption curves, sorption hysteresis. Actually, sorption is a phase change between water vapour and bound water, which occurs when the driving potentials in the two phases are not in equilibrium. This is unique for this type of models, but necessary when considering the different climatic influences on wood. It is evident that accurate models for description of the transfer of moisture in wood are very complex.

Analytical solutions to these kinds of problems are not possible. Therefore the calculation of an arbitrary state
of humidity and temperature field in the timber element that is exposed to variable surrounding humidity can be achieved only by numerical calculations.

2 METHOD

2.1 Model

The model for moisture transport used here is a coupled Fickian model presented by Svensson and Frandsen [1, 2]. In this model vapor transport in pores and bound water transport in wood tissue are modelled by individual transport equations both following Fick’s law. The transport equations are fully coupled by process of phase change from vapor to bound water or vice versa, i.e. sorption. The hysteresis effect found for sorption is also included in the model. Mathematically the model is written as the two transport equations coupled through the sorption.

\[ \frac{\partial (\varphi p_v)}{\partial t} = \frac{\partial}{\partial x} \left( D \frac{\partial p_v}{\partial x} \right) - \frac{RT}{M_{\text{H}_2\text{O}}} \frac{\partial \hat{c}}{\partial t} \]  

(1)

\[ \frac{\partial \hat{c}_b}{\partial t} = \frac{\partial}{\partial x} \left( D_b \frac{\partial \hat{c}_b}{\partial x} \right) + \frac{\partial \hat{c}}{\partial t} \]

The driving potentials are (1) written as vapor pressure \( p_v \) for the water vapor transport in the lumens and other pores and concentration \( c_b \), for the bound water transport in the cell wall. \( \varphi \) is the porosity, \( D \) is the diffusivity, \( R \) is the universal gas constant, \( T \) is the temperature, \( M_{\text{H}_2\text{O}} \) is the molecular mass of water and \( \hat{c} \) is the sorption.

The diffusivity of water vapor in the wood pores is assumed to follow the empirical relation established by Schirmer [3] for water vapor diffusion in air.

\[ D_p = \left( 2.31 \cdot 10^{-5} \frac{p_{\text{am}}}{p_{\text{am}} + p_v} \left( \frac{T}{273} \right)^{1.81} \right) \xi \]  

(2)

Where \( p_{\text{am}} \) is the atmospheric pressure and \( T \) is the temperature. \( \xi \) is a material parameter accounting for the influence of the porous structure of wood on the diffusivity; \( \xi \) is a positive value smaller than unity.

The diffusivity of bound water is assumed to follow Arrhenius law [4].

\[ D_b = D_0 e^{-\frac{E_b}{RT}} \]  

(3)

Where \( D_0 \) is the material directional diffusivity and \( E_b \) the activation energy [5]. The activation energy is assumed to be described by:

\[ E_b = (a - b \frac{c_b}{\rho}) \]  

(4)

where \( a \) and \( b \) are constants and \( \rho \) is the wood dry density.

Sorption is driven by the out of balance of the moisture in the cell wall and the moisture in the pores.

\[ \frac{\partial \hat{c}}{\partial t} = H(p_{\text{ev}} - p_v) \]  

(5)

where \( H \) is the reaction rate function for sorption with the following expression:

\[ H = C_0 + C_1 e^{-\frac{p_{\text{ev}} - p_v}{p_v}} \]  

(6)

where \( C_0 \) to \( C_1 \) are model parameters.

The out of balance term \( (p_{\text{ev}} - p_v) \) is based on the equilibrium states between wood and vapor described by the sorption isotherm. \( p_{\text{ev}} \) is the equivalent vapor pressure for a known cell wall moisture concentration, \( c_b \) at equilibrium moisture state, see Figure 1.

The equilibrium moisture states for the two extremes of solely absorption from dry state and solely desorption from saturated state of wood and surrounding air humidity at constant temperature is assumed to follow the Hailwood and Horrobin [6] sorption isotherm model:

\[ mc = \frac{h}{k_1 + h \cdot k_2 + h^2 \cdot k_3} \]  

(7)

where equilibrium moisture content \( mc \) is the ratio of bound water concentration \( c_b \) and dry wood density \( \rho \), humidity \( h \) is the ratio of vapor pressure \( p_v \) and saturated vapor pressure \( p_{\text{sat}} \), at prevailing temperature and \( k_1 \) to \( k_3 \) are model parameters. All possible moisture states for the wood at a constant temperature are enclosed by the sorption isotherm model, illustrated in Figure 1.

The hysteresis of the sorption is modeled by the exploited sorption site model [1]. The model uses a normalized moisture state variable \( s(h) \) at equilibrium defined as:

\[ s(h) = \frac{mc(h) - mc_a(h)}{mc_j(h) - mc_a(h)} \]  

(8)

where \( mc_a \) is the equilibrium moisture content on the absorption boundary curve and \( mc_j \) is the equilibrium moisture content on the desorption boundary curve for the same prevailing humidity. The state variable, \( s \), has the following expression:

\[ s_1 = s(h_1, s_0, h_0) = \begin{cases} 
0 & h_1 - h_0 > 0 \land s_0 = 0 \\
1 & h_1 - h_0 < 0 \land s_0 = 1 \\
-1 + 2 \left( 1 - h \frac{\ln(1/(1-h_0))}{\ln(1/(1-h_0))} \right) & h_1 - h_0 > 0 \land s_0 > 0 \\
2 - 2 \left( h \frac{\ln(1/(1-h_0))}{\ln(1/(1-h_0))} \right) & h_1 - h_0 < 0 \land s_0 < 1 
\end{cases} \]  

(9)

where index 1 indicates present equilibrium state and index 0 previous equilibrium state. The history state variables, \( h_{d0} \) and \( h_{d1} \) are defined as:

\[ h_{d0} = h_0 \left( \beta h_0 \right) \left( \frac{\ln(1/(2-x_1))}{\ln(1/(2-x_1)) + \ln(1/(2-x_2))} \right) \]

\[ h_{d1} = 1 - \left( 1 - h_0 \right) \left( \frac{\ln(1/(2-x_1))}{\ln(1/(2-x_1)) + \ln(1/(2-x_2))} \right) \]

where \( \alpha \) and \( \beta \) are model parameters.

The equilibrium moisture state determined for the boundary curves, Equation (7), or for the hysteresis case on a scanning curve, Equations (8) and (9), are connected to the moisture transport model through
Equation (5). The constitutive relation functions, $D_p$, $D_b$, and $H$ described by Equations (2), (3) and (6) are employed in the governing Equations (1). The equation system for moisture transport has the two initial unknown model variable; bound water concentration, $c_b$, and partial vapor pressure, $p_v$. All material parameters are given in appendix.

### 2.2 Boundary conditions

The extreme but also possible situation of full convection, i.e. when the convection in the air surrounding the wood is capable of instantaneously establishing and maintaining the ambient humidity on the macroscopic wood surface, is assumed. Consequently partial vapor pressure at the surface is assumed to be identical to the partial vapor pressure in the ambient air. The Dirichlet boundary condition is therefore applied:

$$p_v^s = p_v^a$$

where $p_v^s$ is the partial vapor pressure at the (imaginary) boundary between surrounding air and pores at the macroscopic surface and $p_v^a$ is the partial vapor pressure of ambient air.

The vapour pressure of the ambient air was modelled as the simple harmonic equation:

$$p_v^a = p_{v,0} + \sum_{i=1}^{N} A_i \sin(2\pi \cdot t / P_i)$$

where $p_{v,0}$ is the vapour level, $A$ is the half amplitude, $P$ the period and $t$ is time.

Bound water in the cell wall can only be interchanged with the air, surrounding the wood and in the wood pores, through sorption. Hence, the phase bound water is confined in the cell wall and the Neumann boundary condition, fully restricted, is applied:

$$n \cdot J_b = 0$$

where $J_b$ is the bound water flux and $n$ the normal vector to the cell wall surface.

### 2.3 Numerical solution

The solution of the coupled boundary value problem is calculated numerically with the help of partial differential equation solver in MatLab environment. Original code is appropriately modified to be used with hysteresis model (9) and solve the problem of moisture transport in wood for isothermal condition and assuming the effect of sorption heat insignificant.

### 3 NUMERICAL STUDY

In the first part of the study the aim is to present the influence of daily variation of humidity in combination with annual variation of humidity on moisture field in timber element. For modeling this type of naturally varying climate two harmonic cycles were combined together, i.e. one with a daily period and one with an annual period c.f. (10). Of course this is only a rough simplification of real humidity variation which, as it is know, can be much more stochastic. This way the timber element is exposed to the combined harmonic boundary conditions (Fig 1.).

The response to the harmonic load is for the early stages in the process dependent on the initial moisture state of the member. The offset of the initial moisture state in relation to the, so-called, quasi-static or oscillating moisture state equilibrium [8] is always influencing and may, if large, dominate the moisture behaviour initially. In the following numerical studies the harmonic cycle is always cycling around the quasi-static moisture equilibrium which is also the initial condition for the moisture content.

For the case with a combined input the amplitude for the annual sinusoid was equivalent to 30% relative humidity and daily sinusoid had an amplitude equivalent to 60% relative humidity. The calculated moisture states for a wood specimen in the combined climate at the macroscopic surface inward to a depth of 150 mm is shown in figure 2 as two enveloping curves. The enveloping curves are calculated as the maximum and minimum moisture content at a given depth over a time period of four years. The quasi-static moisture equilibrium is also illustrated as the horizontal line in figure 2 marked as QS,C.
variation. The two cases with single sinusoid investigated are the terms of the combined sinusoid with the exception that the daily variation’s vapor level is equivalent to a relative humidity of 65%. This level is equal to the sum of annual sinusoids vapor level and half its amplitude. Compared to combined sinusoid humidity variation the numerical simulation of single sinusoid is less time consuming. In order to make the simulation short as possible the initial moisture state should coincide with quasi-static state for single sinusoid humidity variation, where the quasi-static state for single sinusoid humidity variation was determined with help of the results presented in [8]. The calculated envelope curves for these two cases are illustrated in figure 3 with the daily case on top in red color. With straight dashed lines quasi-static state for daily (QS,D) and annually (QS,A) humid variation are presented (figure 3).

By superimposing the envelope curves in figure 3 the maximum enveloping moisture content, mc, for the combined humidity variation is here estimated as:

\[ mc = (1-w) \cdot mc_{QS,A} + w \cdot mc_{QS,D} + \Delta mc_{A+} + \Delta mc_{D+} \]

where \( w \) is a weight factor, \( mc_{QS,A} \) and \( mc_{QS,D} \) are the quasi static moisture states for the annual and daily case, respectively. The positive moisture fluctuations from the quasi static moisture states for annual and daily case are \( \Delta mc_{A+} \) and \( \Delta mc_{D+} \).

By choosing the weight function \( w \) to 0.5 for \( L < 13 \text{ mm} \) and else 0 the result will be as the upper blue curve together with black diamonds in figure 4. The position \( L = 13 \text{ mm} \) is the depth at which the humidity variations with a daily period has just lost influence on the moisture state. In other words the daily harmonic humidity cycle will not give moisture fluctuation at depth in wood larger than 13 mm if initial moisture state coincides with the quasi static moisture state. By accounting for that the quasi static moisture state reached with harmonic humidity with daily period gradually loses influence on the moisture state from \( L = 5 \text{ mm} \) (\( w = 0.5 \)), when the estimated moisture content is lower than the quasi static moisture state for the daily period, to \( L = 13 \text{ mm} \) (\( w = 0 \)), when the daily cycle is insignificant for moisture fluctuation, the enveloping moisture curve calculated with the combined humidity cycle can be reproduced. In figure 4 the dashed curves are the calculated values for the combined case, also depicted in figure 2, and the blue curve when \( mc > 0.14 \) together with red and black diamonds are the superimposed estimation.

By repeating the above exercise for minimum values of the annual case, figure 3, and a daily case with a vapor level equivalent to 35% relative humidity, the lower envelop curve is reproduced with same accuracy as the for the maximum values shown here.

4. CONCLUSION

Analysis of timber element exposed to natural variation of surrounding humidity was presented. Stochastic oscillation of outside humidity was exemplified with combination of two harmonic cycles. For the calculation of moisture distribution in the timber element coupled multi-Fickian model including sorption hysteresis model was used. The results from the numerical study of timber specimen exposed to daily, annually and combined cyclic variation of ambient humidity were studied. It is confirmed [7, 8] that the quasi-static moisture equilibrium is independent amplitude and period of harmonic humidity cycles. The enveloping curve of the possible moisture content within a specimen as a consequence of combined harmonic humidity cycles can be reproduced by superimposing the responses from the harmonic cycles composing the combined cycle. In order to verify the superposition technique future research should contain different combination of load combinations, for example extreme pulse load. It is known [7] that sorption is connected to heat flux.
Therefore for the influence of heat transfer on the moisture state is the field that needs further study.

**ACKNOWLEDGEMENT**

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**REFERENCES**


**APPENDIX**

**Table A1: Parameters used in numerical simulations.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
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<tr>
<td>$\varphi$</td>
<td>0.67</td>
<td>-</td>
</tr>
<tr>
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<td>-</td>
</tr>
<tr>
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<td>$a$</td>
<td>$38.5 \times 10^3$</td>
<td>J·mol$^{-1}$</td>
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<td>$b$</td>
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<td>s$^2$·m$^{-2}$</td>
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EXPERIMENTAL AND NUMERICAL ANALYSISYS OF GLULAM BEAMS IN NATURAL CLIMATIC CONDITIONS

Tomaž Hozjan¹, Tomaž Pazlar², Stane Srpčič³

ABSTRACT: The paper presents some experimental results of the long term behaviour of glued laminated timber in sheltered outdoor environment and some new results in numerical modelling of experiment. The tests on six loaded and five unloaded glulam beams were initiated in the summer of 2003 and finished in autumn 2006. The temperature, relative humidity, displacements and strains were measured in hourly intervals and gathered in a measurement database. The coupled problem of moisture and heat transfer over the beam was solved numerically by an enhanced FEM based calculating procedure. The numerical values of MC were compared to the measured ones and to the average values obtained by non-destructive gravimetric method. In the second stage of numerical analysis the mechanical response of the loaded beam was determined taking into account the temperature and moisture dependent elastic, thermomechanical, normal creep and mechano-sorptive strains as well as the shrinkage and swelling strains. The calculated longitudinal and transversal strains and displacements were compared to those measured during the experiment.

KEYWORDS: Long term tests, glulam beams, heat and mass transfer, mechanical response

1 INTRODUCTION

The main aim of the presented investigation was to acquire more profound knowledge about the long-term behaviour of load bearing glulam beams in changing climatic conditions as appearing during seasons in natural environment. Many similar investigations of rheological behaviour of real sized wood and wood based products have already been performed around the globe ([1], [2]). Therefore the aim of experiment was to obtain datasets on shrinkage and swelling, viscous creep and mechano-sorptive effect also for glue laminated timber (glulam) made of Slovenian spruce. The testing methods providing the essential material parameters for the computational analysis of timber structures mostly deal with relatively small specimens although the model similarity with real sized structural elements is not always self-evident. Therefore, the verification or even calibration of material parameters which govern the development of particular strains in loaded wooden beams seem to be an interesting topic when investigating the mechanical behaviour of timber structures.

2 EXPERIMENT: DESCRIPTION AND RESULTS

Testing equipment was set up at the outdoor testing facilities at Slovenian National Building and Civil Engineering Institute (ZAG, Ljubljana). The glued laminated beams were exposed to outdoor climate, except direct insulation – the testing area was covered and according to the sunlight analysis also shaded from the south and east side. Consequently, the tested beams were protected against the direct impact of rain and snow, as well. The installed protection did not affect the air circulation around the specimens. The test set-up is presented in Figure 1.

Six loaded ("L1 – L6") and five non-loaded (“N1 – N5”) spruce glulam beams with the dimensions 0.05 x 0.10 x 1.80 m were installed as simply supported beams on the wooden substructure. The masses of glulam elements indicate that some beams may have lower stiffness – this was also confirmed with the MOE measurements and destructive tests performed after long term tests (Table 1). Two additional (shorter) elements with three isolated sides were installed for the purpose of implementation of non destructive gravimetric method for moisture content measurements on relatively small specimens [3]. All beams were made from non-treated spruce and glued

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with RFF glue. Steel frames in combination with concrete blocks were used to impose load on beams. The loaded beams were equally loaded with $P = 3.04$ kN. The loading schema is presented in Figure 2. The measurements were recorded at hourly intervals from June 9th 2003 to October 12th 2006.

Table 1: Characteristics of beams

<table>
<thead>
<tr>
<th>beam</th>
<th>$\rho_{gluam}$ [kg/m³]</th>
<th>MOE$_{bend.}$ [N/mm²]</th>
<th>MOR$_{byp}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>477</td>
<td>13781</td>
<td>54.9</td>
</tr>
<tr>
<td>L2</td>
<td>436</td>
<td>10220</td>
<td>39.3</td>
</tr>
<tr>
<td>L3</td>
<td>528</td>
<td>14549</td>
<td>50.5</td>
</tr>
<tr>
<td>L4</td>
<td>516</td>
<td>15514</td>
<td>62.3</td>
</tr>
<tr>
<td>L5</td>
<td>525</td>
<td>15813</td>
<td>69.5</td>
</tr>
<tr>
<td>L6</td>
<td>416</td>
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<td>36.7</td>
</tr>
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<td>N1</td>
<td>490</td>
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<td>509</td>
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<td>N4</td>
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<tr>
<td>N5</td>
<td>436</td>
<td>11854</td>
<td>47.9</td>
</tr>
</tbody>
</table>

The following quantities were measured:

- Loaded beams: vertical displacements in the middle of span (LVDTs), horizontal displacements at the free end (LVDTs), longitudinal strains – top and bottom lamella (strain gauges) – all beams, longitudinal strains (beam “L6”) in second top, middle and second bottom lamella (strain gauges) and transverse strains (extensometers).
- Unloaded beams: horizontal displacements (LVDTs) at the free end, longitudinal strains (beam “N1”) in the top, middle and bottom lamella (strain gauges) and transverse strains (extensometers).
- Moisture sensors at three different depths (10, 25 and 40 mm) were installed in additional element.
- Temperature and relative air humidity.

All measured quantities were evaluated and filtered from quantitative point of view, gathered in an extensive database and represented graphically. The results of the experiment were comprehensively already presented in [4]. Therefore only the relevant conclusions obtained in experiment are presented in this paper.

The temperature and relative air humidity were measured at two locations inside the test setup, but the results practically did not differ. The lowest measured temperature was $-12.8^\circ$ C and the maximum temperature was $36.5^\circ$ C. The relative air humidity ranged from 36.1% to about 100%. From the temperature diagrams clear yearly cycles can be observed, whereas differences in relative humidity are not so pronounced. The shapes of diagrams of vertical displacements of all tested beams are similar, but due to differences in stiffness (Table 1) there is a significant difference between the absolute values of displacements (Figure 4).

The diagrams of vertical displacements also confirm that the majority of creep deformations occur in the first two months after imposing a load. Although the displacements generally increase, local aberrations can also be observed: displacements increase in summer and decrease in winter. Measurements in certain periods are missing due to problems with the central data acquisition computer.
The shape of horizontal displacement diagram also closely follows the temperature and relative air humidity diagrams (Figure 5). However the differences in mechanical characteristics of beams are not evident.

Figure 5: Horizontal displacements at the free end loaded beams

Similar conclusions about the resemble of displacement/strain curves for both loaded and unloaded beams with the temperature/relative air humidity curves can also be made for the other measured quantities (longitudinal strains, transverse strains). See reference [4] for details.

3 NUMERICAL SOLUTION

When analyzing the mechanical behaviour of timber elements in a changing climate conditions, the temporal and spatial distribution of temperature and water content of wood play a decisive role. Assuming that the deformation of the structure does not significantly affect the moisture and heat transfer in the wood, the numerical procedure can be divided into two physically distinct but closely related phases. In the first phase the time development of the moisture and temperature state of the timber element has to be calculated and the results are employed as the input data for the second phase in which the mechanical response of glulam elements on the mechanical load as well as on actual humidity and temperature conditions is determined.

3.1 Moisture and temperature state of timber

The heat and moisture transport over the beam is described by coupled multi-Fickian model presented in [5]. Governing equations of the vapor transport in pores and the bound water transport in wood tissue are modelled by a system of two individual transport equations both following Fick’s law as:

\[
\frac{\partial}{\partial t} \left( \nabla \cdot \left( D_b \nabla c_b + D_v \nabla T \right) \right) + \dot{c} = 0,
\]  

(1)

\[
\nabla \cdot \left( D_v \nabla \left( p_v \frac{\varphi M_{H_2O}}{RT} \right) + D_c \nabla T \right) - \dot{c},
\]  

(2)

where the driving potentials are concentration \(c_v\) for the bound water transport in the cell wall and vapor pressure \(p_v\) for the water vapor transport in the lumens and other pores. \(D_b\) and \(D_v\) are the corresponding diffusion matrices of bound water and water vapour, respectively \(D_{cT}\) and \(D_{vT}\) are corresponding thermal coupling diffusion matrices, \(T\) is the temperature, \(\varphi\) is the porosity, \(M_{H_2O}\) molecular mass of water, \(R\) the universal gas constant and \(\dot{c}\) is the sorption rate. All the diffusion coefficients in Eqs. (1) and (2) are temperature dependent. Their relationships are given in [5]. Transport equations (1) and (2) are fully coupled by process of phase change from vapor to bound water or vice versa also known as sorption. Process of sorption is a time-dependent exothermal reaction. According to Le Chatelier’s principle more water can be bound in the cell wall if the temperatures are lower. In the present analysis temperature dependence of the boundary sorption curves (adsorption and desorption curve) is considered according to the [5], while sorption process is described with sorption hysteresis model give in [6].

The third equation is the energy conservation equation where the effect of accumulation, conduction and diffusion is taken into account:

\[
\left( C_v \dot{c}_v + c_v C_v + c_b C_b \right) \frac{\partial T}{\partial t} + \left( h_b - h_v \right) \dot{c} + \left( C_v J_v + c_v J_v \right) \nabla T = \nabla \left( k_{mix} \nabla T \right),
\]  

(3)

There \(T\) is the temperature, \(C_v\) is the specific heat capacity of the \(\alpha\)-component of the mixture, \(h_b\) and \(h_v\) are the enthalpy of the phases (bound water and water vapour), \(J_v\) and \(J_c\) mass fluxes of the phases, \(k_{mix}\) the thermal conductivity of the timber. Furthermore, the homogeneity of material properties across the cross-section is assumed.

The equation system for moisture and heat transport (1)–(3) has three initial unknown model variables; bound water concentration, \(c_b\), partial vapor pressure, \(p_v\) and temperature, \(T\). Equations (1)–(3) with corresponding initial and boundary conditions are generally non-linear and can be rarely solved analytically. Therefore, numerical methods have to be employed. In our case the computer program based on finite element method, presented in [7] was used and extended to include heat transfer. The system (1)–(3) has to be solved taking into account corresponding boundary conditions. It is assumed that the partial vapor pressure at the (imaginary) boundary between surrounding air and pores (lumens) at a macroscopic surface, \(p_v^a\), is identical to the partial vapour pressure of ambient air, \(p_v^s\). The Dirichlet boundary condition is therefore applied:
\[ p_1' = p_1'' . \] (4)

Bound water in the cell wall can only be interchanged with the air (surrounding the wood and in the wood lumen) through sorption. Hence, the phase bound water is confined in the cell wall and the Neumann boundary condition, fully restricted, is applied:

\[ \mathbf{n} \cdot \mathbf{J}_b = 0, \] (5)

where \( \mathbf{J}_b \) is the bound water flux and \( \mathbf{n} \) is the normal vector to the cell wall surface. Heat transfer at the boundary is prescribed with the Neumann boundary condition:

\[ \mathbf{n} \cdot \mathbf{J}_r = -k_{\text{mix}} \nabla T \] (6)

where \( \mathbf{J}_r \) is the heat flux at the surface.

### 3.2 Mechanical analysis

The basic assumption at the mechanical analysis is that imaginary filaments of an element are exposed to uniaxial stress state. To consider the non-linear material behaviour the relations between strain \( \varepsilon \), stress \( \sigma \), moisture \( w \) and time \( t \) are expressed in an incremental form:

\[ d\sigma = d\sigma(\varepsilon_0, \varepsilon_0, c_{10}, d\varepsilon, d\varepsilon, dt). \] (7)

In this work, the additive principle is adopted where the total geometrical strain increment \( d\varepsilon \) is expressed as a sum of shrinkage/swelling \( d\varepsilon_s \), normal creep \( d\varepsilon_c \), mechano-sorptive \( d\varepsilon_{ms} \) and mechanical strain increment \( d\varepsilon_m \). Due to incremental approach and assuming that all the values involved are sufficiently small, in our numerical evaluation, the infinitesimal stress and strain increments as well as the increments of water content and time are replaced by the finite ones and additive principle can be written as:

\[ \Delta\varepsilon = \Delta\varepsilon_s + \Delta\varepsilon_c + \Delta\varepsilon_{ms} + \Delta\varepsilon_m. \] (8)

Shrinkage and swelling deformation is assumed to be a linear function of water content. Thus, the increment of shrinkage/swelling deformation is:

\[ \Delta\varepsilon_s = \alpha_s \Delta c_b, \] (9)

where \( \alpha_s \) is a constant shrinkage coefficient parallel to the grain at the actual constant temperature. Normal creep is assumed to be temperature independent; that means that it depends on time and actual stress only. Various creep models for wood can be found in the literature. In this study next model is used, where the increment of normal creep is:

\[ \Delta\varepsilon_c = \sigma_0 \alpha_1 \left( e^{-\alpha_2 \Delta t} - e^{-\alpha_1 \Delta t} \right) . \] (10)

where \( \sigma_0 \) is the stress at the beginning \( t_0 \) of the time step \( \Delta t = t_1 - t_0 \), \( \alpha_1 \) and \( \alpha_2 \) are model parameters. The increment of mechano-sorptive deformation is expressed by

\[ \Delta\varepsilon_{ms} = \sigma_0 \Phi \left( 1 - e^{-\alpha_3 \Delta w} \right), \] (11)

where \( c \) is generally different for sorption and desorption and \( \Phi \) is the reference compliance.

The mechanical part of the deformation consists of an elastic part \( \Delta\varepsilon_e \) only, i.e., the plasticity effect is negligible and does not explicitly depend on time and water content it is obtained from Eq. (8) as:

\[ \Delta\varepsilon_e = \Delta\varepsilon_c = \Delta\varepsilon_e - \Delta\varepsilon_e - \Delta\varepsilon_{ms}. \] (12)

Based on Hooke’s law, the increment of elastic strain can also be expressed as

\[ \Delta\varepsilon_e = \frac{1}{E_1} \left( \Delta\sigma - \Delta E\varepsilon_{e0} \right). \] (13)

Here, subscripts 0 and 1 denote the quantities at the beginning and at the end of the time step, respectively. By comparing the Eqs. (12) and (13), the stress increment \( \Delta\sigma \) can be expressed in a simple form

\[ \Delta\sigma = E_1\Delta\varepsilon_c + \Delta E\varepsilon_{e0}. \] (14)

Stress \( \sigma_1 \) at the end of the time step is

\[ \sigma_1 = \sigma_0 + \Delta\sigma. \] (15)

Eqs. (8) – (15) represent a specific constitutive model which was incorporated into the self-developed computer program which works in MatLab environment. The program is based on the finite element method, which enables geometrically and materially non-linear analysis of planar beams and frames. Kinematic equations used in the formulation of this element allow consideration of large displacements and rotations and moderate deformations. The basic equations were developed by the mixed variational principle where, besides the transversal displacements and rotations, the axial force is taken as an independent parameter. This element improves the convergence of numerical procedures involved in mechanical analysis.

### 4 NUMERICAL EXAMPLE

The aim of this study was to present the spatial and time development of temperature and moisture content and also the mechanical behaviour of glulam beam exposed to varying outside ambient conditions. Results are compared with those measured during experiment. In this analysis we consider first 192 days of the experiment period to present the capability of the before presented numerical model, while whole experiment lasted for 2 years.

#### 4.1 Results of moisture and temperature state of timber

The numerical values of the material parameters, used in the numerical calculations, for the constitutive relation and the governing equations are taken from the literature [5], [6], [7]. Material parameters that differ from the literature are given in the Table 2.
Table 2: Material parameters used in numerical simulation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>reduction factor</td>
<td>( \zeta_r ) 0.4</td>
</tr>
<tr>
<td>diffusion coefficient</td>
<td>( D_T ) 17.5 ( \times 10^{-6} ) m(^2) s(^{-1} )</td>
</tr>
</tbody>
</table>

The beam consists of seven laminations. We assume that the glue between laminations is an impermeable barrier for the moisture flow which represents an extreme assumption about the glue permeability. In reality certain degree of permeability may be detected for the resorcinol glue, frequently used in manufacturing glulam beams. This way in numerical simulation we can model only one lamination of the cross-section that is exposed to the outside change of relative humidity (RH) and temperature. The time development of the moisture content in the point at the distance of 10 mm from the outside boundary is presented in Figure 6. The comparison between measured and calculated values shows locally more pronounced differences which indicates that the assessment of diffusion parameters is not ideal. Yet we can observe very good local daily variation of the moisture content in the first 50 days. At around 50 days quite big sudden increase of the measured moisture content can be noticed up to around 30%. Afterwards measured moisture content drops again down quite rapidly to around 18%. This measured values leaves out doubts on the accuracy of the moisture measurements around this period. Anyway the calculated daily amplitude variation is on average around 3% and very well agrees with one from the experiments.

Figure 7: Moisture content development in the centre point of the middle lamination of the glulam beam for 192 days.

Similar behaviour of the moisture content distribution can be noticed on Fig. 7 where the time development of the moisture content in the centre point is presented. Here slightly bigger difference in average value of the moisture content from 130 days onward can be noticed. Figure 8 shows that measured and calculated values of temperature in the centre point of the middle cross-section of the beam are in good agreement.

Figure 8: Temperature development in the centre point of the middle lamination of the glulam beam for 192 days.

4.2 Results of the mechanical analysis of the glulam beam

The geometry and the mechanical load are shown in Fig. 1. The water content distribution presented in previous section was used as input data for mechanical analysis. Mechanical analysis is performed for the glulam beam mark L3 in the experiment. The material parameters are shown in Table 3.

Table 3: Material parameters used in numerical simulation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
<td>( E_{ref} = 1450 ) kN/cm(^2)</td>
</tr>
<tr>
<td>Shrinkage parameter</td>
<td>( \alpha_s = 6.25 \times 10^{-5} ) %/cm</td>
</tr>
<tr>
<td>Mechno-sorptive parameter</td>
<td>( \Phi^\infty = 0.00003 )</td>
</tr>
<tr>
<td>(sorption)</td>
<td>( c^+ = 1.6 )</td>
</tr>
<tr>
<td>(desorption)</td>
<td>( c^- = 2.4 )</td>
</tr>
</tbody>
</table>

In Table 4, the values of the parameters of the creep model used for the mechanical analysis are shown.

Table 4: Creep parameters for the creep model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension ( \alpha_1 \times 10^{-6} )</td>
<td>2.719</td>
</tr>
<tr>
<td>Compression ( \alpha_2 \times 10^{-6} )</td>
<td>1.975</td>
</tr>
<tr>
<td></td>
<td>2.017</td>
</tr>
</tbody>
</table>

Displacements at the mid-span of the beam obtained by numerical simulation were compared to displacements obtained by the experiment (Fig. 9). At the beginning up to 80 days the numerical displacement is lower than the one from experiment. Afterwards the experimental displacement somehow stabilizes and the global increase of the mid-span displacement is relatively small, while the calculated values of the mid-span displacement still
rise up to 130 days before they start to stabilize and at the end they are bigger than experimental values. If we look back at the results of the moisture content we can notice that after 80 days numerical values of moisture content are bigger than experimental one. Therefore such numerical results of the mid-span displacement are somehow expected. However, numerical and experimental results in general prove that the influence of shrinkage, creep and mechano-sorptive effect on real timber structures is significant in the early phase of loading. It was found that mechano-sorptive and creep parameters have big impact on the numerical result. Values given in Table 3 and 4 were obtained by parametric study.

![Figure 9: Vertical displacements in the middle of span, loaded beams L3.](image)

5 CONCLUSIONS

Although the number of specimens in the presented experiment was relatively low, the test results in general confirm that at the stress level applied the influence of shrinkage, creep and mechano-sorptive effect on real timber structures is significant in the early phase of loading. However, listed effects become negligible after relatively short period of time. For the case of beam L3 this was already around 50 days. The dependence of elastic modulus on water content and temperature remains an important parameter which determines the deformability of beams. The main problem in mechanical analysis remains the experimental evaluation and verification of parameters involved in numerical procedures. It turns out that mechano-sorptive and creep parameters have major impact on the numerical results.

ACKNOWLEDGEMENT

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REFERENCES

TIMBER ENGINEERING AS A TOOL FOR SPECIES CONSERVATION IN TROPICAL RAIN FORESTS: THE CASE OF THE CONGO BASIN FOREST.

René Oum Lissouck¹, Régis Pommier², Patrick Castéra³, Louis Max Ayina Ohandja⁴, Frank Taillandier⁴, Emmanuel Njungab⁵, Gérard Elbez⁶.

ABSTRACT: Timber exploitation in the Congo basin forest is highly intensive and restricted to a low wood species number. As consequence, rarefaction and extinction risk of some species are increasing. The first engineering solution set up to mitigate the wood overexploitation is the mechanical grading (similar to Eurocode 5) of structural timber in four classes. The paper focuses on Engineered Tropical Wood Products (ETWP), namely glulam as a new approach. Wood species are chosen according to a multicriteria analysis. The selected criteria are: extinction risk, harvesting potential, traditional gluing ability, mechanical strength, durability and wood machining. Single and mixed glulam beams are realized with specific polyurethane glue. Delamination (EN 302-2) and shear tests (EN 392) are fulfilled (in tropical conditions) in order to assess glue joints performance. The results show a good respect of all standards requirements. Such ETWP, which can be manufactured and used in tropical conditions, are wood engineering tools to conserve and highlight Congo basin forest richness.

KEYWORDS: ETWP, multicriteria analysis, extinction risk, forest biodiversity, Congo basin.

1. INTRODUCTION

The Congo basin forest is the second tropical forest in the world. Cameroon country is a representative ecosystem of this area. At least three hundred wood species have been inventoried [1]. More than a hundred can be used as structural wood. But timber exploitation in this geographic area is highly selective. Among the hundred structural wood species, only six represent more than 80 % of the total exploitations [2]: sapelli (entandrophragma cylindricum), azobé (lophira alata), iroko (milicia excelsa), ayous (triplochiton scleroxylon), tali (erythrophleum ivorensis), fraké (terminalia superba). The direct consequences are the increasing rarefaction and a high extinction risk of these species. More than three of them are classified as endangered, like all the entandrophragma trees species [3, 4]. Another reasons contributing to the decline of these species are:

- the lack of technological knowledge (gluing, drying, durability and mechanical properties) on less exploited secondary woods;
- the international and local markets practices;
- the poor implementation at a national level of the international agreements on biodiversity conservation (ratified by many countries in the world).

In this paper, a new approach based on Engineered Tropical Wood Products (ETWP) technology (namely glulam), is proposed. After a reasoned wood species selection, we present the first tests results of single and mixed green gluing of ayous (triplochiton scleroxylon) and frake (terminalia superba).
2. STRATEGIES BASED ON ENGINEERING CONCEPTS

The mechanical grading of structural timber in the Congo basin [5] has been set up to mitigate the overexploitation of tropical wood and promote the subsistence of over-consumed species in our biodiversity. 19 species of the cameroonian part of the Congo basin have been organized in four resistant groups led by ayous, sapelli, tali and azobé. The assignment of a given species to one of the four groups takes place in return of a homogeneity test comparison of the random variables of species to the random variable of the group. The main interest of this grouping technique, similar to the Eurocode 5 grading, is that mechanical properties of some tropical woods are now guaranteed. But, this knowledge improvement is not enough to come over companies and consumers interest on tropical referenced woods.

The topic of this paper is based on ETWP manufactured by using gluing techniques. ETWP are more reliable and efficient than traditional wood products. Nowadays, their design can be performed by using one or two woods species. ETWP can be locally manufactured without significant industrial resources [7].

Due to some specificities (dense wood species, high shrinkage coefficients, presence of removals), gluing tropical woods is difficult, especially in tropical conditions characterised by high temperature and high relative humidity. Therefore, a correct and rigorous experimental gluing study is required to assess the bondability of tropical single and mixed glued woods.

Table 1: Mechanical grading and physical characteristics of Congo basin wood species [5]

<table>
<thead>
<tr>
<th>Groups</th>
<th>T38</th>
<th>T66</th>
<th>T93</th>
<th>T136</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic breaking stress (MPa)</td>
<td>38</td>
<td>66</td>
<td>93</td>
<td>136</td>
</tr>
<tr>
<td>Characteristic elastic modulus (MPa)</td>
<td>5,205</td>
<td>9,386</td>
<td>13,280</td>
<td>19,611</td>
</tr>
<tr>
<td>Characteristic volumic mass (kg/m³)</td>
<td>245</td>
<td>453</td>
<td>648</td>
<td>954</td>
</tr>
</tbody>
</table>

Wood species

- Fromager
- Ayous
- Sapelli
- Iroko
- Bibolo
- Padouk
- Fraké
- Eyong
- Moabi
- Ebene
- Moringui
- Bilinga
- Bele
- Wenge
- Difou
- Doussié
- Bubinga

3. EXPERIMENTAL WORK

Experimental design has been performed in three steps: wood species selection, gluing tests and results analysis.

3.1 WOOD SPECIES SELECTION

Experimental works are based on a technological database of 77 species of potential interest, produced by CIRAD [6] and Cameroonian Ministry of Forest Affairs. The data have been successfully compared to other literature sources (Tropical Timber database and ProtaBase). In order to obtain an optimal choice of species that takes into account the main concerns of key stakeholders in the tropical timber industry, six criteria among the most important steps of eco-designed wood products have been selected. These criteria are: extinction risk, harvesting potential, gluing ability, natural or artificial durability, mechanical grading and lamellae machining. Every criterion is linked to a main objective assigned to ETWP. The 77 wood species have been classified according to every criterion in order to divide them in, more or less, homogeneous categories that have close similarity.

3.1.1 Extinction threat

More than 7300 wood species (about 10% of wood known species) are subjected to extinction risk in the world over [3]. The status of some tropical rainforest wood species in the Congo basin has deteriorated rapidly over recent decades. Heavy exploitation for the timber is the main cause of the decline. The intensity and the felling of the most commercialized species exceed the natural regeneration capacity [8]. Moabi (baillonella toxisperma) for instance is barely present in felling gaps in the Cameroonian South West part of the natural rain forest [9]. We have organised the 77 species in five status conservation groups: critically endangered (CR), endangered (EN), vulnerable (VU), least concern (LC) and No concern (NC). (CR), (EN), and (VU) are the various levels of endangered species. Mukulungu (autranella congolensis) is classified as critically endangered [10].

3.1.2 Harvesting potential

Harvesting potential is a good indicator of wood species rarefaction and sustainable forest management. In the cameroonian part of the Congo basin forest, 31 species represent 90% of the total harvesting potential, which varies from 0.1 up to 2 m³/ha [11]. It affects the financial viability of ETWP manufacturing. This variation gap allows us to classify our species in five groups: “very low”, “low”, “average”, “high” and “very high”.

3.1.3 Traditional gluing ability

Traditional gluing refers to structural bonding of wood with low moisture content. Tropical woods are generally known as difficult to bond. Gluing is an important process of glulam. Difficulties which affect ETWP bondability are correlated to wood density [12]. A good wood ability to bond is a key element of the continuity of glulam mechanical properties. According to TROPIX 7.0 database [6], there are three groups of traditional gluing ability of tropical woods: “good”, “fair” and “bad”. The 77 wood species have been grouped according to the 3 classes.
Table 2: Gluing ability of some tropical woods

<table>
<thead>
<tr>
<th>Gluing ability</th>
<th>Specie botanic names</th>
<th>Family</th>
<th>Vernacular names</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bad</td>
<td>klainedoxa gabonensis</td>
<td>irvingiaceae</td>
<td>Eveuss</td>
</tr>
<tr>
<td></td>
<td>petersianthus macrocarpus</td>
<td>lecythidaceae</td>
<td>Essia</td>
</tr>
<tr>
<td></td>
<td>desbordesia glaucescens</td>
<td>irvingiaceae</td>
<td>Alep</td>
</tr>
<tr>
<td></td>
<td>staudtia kamerunensis</td>
<td>myristicaceae</td>
<td>Niowe</td>
</tr>
<tr>
<td></td>
<td>Enbrao Oblonga</td>
<td>sterculiaceae</td>
<td>Eyong</td>
</tr>
<tr>
<td>Good</td>
<td>Dictemoranthus benthamianus</td>
<td>caesalpiniaceae</td>
<td>Mowingui</td>
</tr>
</tbody>
</table>

3.1.4 Mechanical strength

The mechanical strength is an important parameter of structures reliability and security. The bending breaking stress is our mechanical strength indicator. Glulam strength in bending is highly linked to the strength grade of external lamellae [13]. 19 species have been organized in four mechanical strength groups: T38, T66, T93 and T136 (table 1). In order to set up the mechanical grading of our 77 species, the grading methodology proposed by Mvogo and al. [5] has been used.

3.1.5 Durability

Durability means the ability of wood to withstand biodegrading organisms (fungi and insects) attacks. A wood moisture content of 20% is usually enough to initiate decay [14], which can have dramatic consequences. Indeed, the degradation of wood causes a decrease in mechanical resistance [15]. In the case of glulam beams, replacing a degraded lamella is quite impossible. Tropical climate conditions are characterised by the highest intensity of biodegraded organisms attacks [16]. By considering the tropical woods resistance against the major biodegrading organisms (fungi and insects), the 77 tropical wood species have been organised in 3 durability classes: “Good”, “Average” and “Bad”.

3.1.6 Wood machining

Sawing and machining operations are 4 times more difficult in the case of tropical woods [17]. As a consequence, the use of less known species in timber structures decreases [18]. In this study, we have enumerated and counted all the major constraints that may affect wood processing before bonding like toxic odor, low drying speed, important strain risks, surface planning difficulty, changes in wood color, internal and external cracking during drying. The number of constraints allows us to classify our species in five groups: “very low”, “low”, “medium”, “high” and “very high”. There is a good correlation between the number of constraints and wood density (figure 1).

Figure 1: Correlation between wood machining difficulty and density.

The correlation between wood machining difficulty and density is in agreement with authors like Simson and Tenwalde [19]. An example of processing difficulty grading is given by table 3.

Table 3: Machining difficulty of some tropical woods

<table>
<thead>
<tr>
<th>Group</th>
<th>Specie botanic names</th>
<th>Family</th>
<th>Vernacular names</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>Triplochiton soleroxylon</td>
<td>sterculiaceae</td>
<td>Ayous</td>
</tr>
<tr>
<td>Low</td>
<td>terminalia superba</td>
<td>combretaceae</td>
<td>Frake</td>
</tr>
<tr>
<td>Medium</td>
<td>aningera altissima</td>
<td>sapotaceae</td>
<td>Aningre</td>
</tr>
<tr>
<td>High</td>
<td>dacyodes buethoven</td>
<td>burseraceae</td>
<td>Ozgo</td>
</tr>
<tr>
<td>Very high</td>
<td>cylcodiscus gabunensis</td>
<td>mimosaceae</td>
<td>Okan</td>
</tr>
</tbody>
</table>

3.1.7 Multicriteria analysis

The usual techniques of optimization, based on a single parameter, are not appropriate for selecting the best wood species needed in ETWP design. Multicriteria analysis is a more interesting tool in case of managing many criteria before making an optimal choice [20]. In this study, we use two ELECTRE (Elimination et Choix Traduisant la Réalité) methods: ELECTRE III and ELECTRE TRI. The family of ELECTRE methods belongs to the French School, which uses the rule of majority in an outranking relation, in opposition with American School which uses the rule of unanimity of criteria in the idea of dominance. ELECTRE III is used to compare alternatives (wood species) from the best to worst. ELECTRE TRI categorizes alternatives in hierarchical groups. The different categorization proposed consists of three levels of interest: high, medium and low. Inside a criterion, a score between 0 and 5 is given to each type of wood according to the homogeneous categorization which has been defined. Some examples of scoring, according to extinction risk and wood durability criteria are presented in tables 4 and 5. The relative importance of one criterion over another has been characterised by a score between 1 and 5. Priority has been given to extinction risk and harvesting potential with a score of 5. Gluing ability, mechanical grading and durability have respectively a score of 4, 3, 2 and 1. A margin of 30% error has been taken into account in the scoring repartition.
3.1.8 Wood species selection results

Results of the two ELECTRE techniques are presented according to the mechanical strength grading in table 6. 20 species are interesting for manufacturing ETWP, especially glulam. *Dabema* is an example.

### Table 6: Results of multicriteria analysis

<table>
<thead>
<tr>
<th>Groups</th>
<th>T38</th>
<th>T66</th>
<th>T93</th>
<th>T136</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Level Interest</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fraké</td>
<td>Tali</td>
<td>Œkan</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dabema</td>
<td></td>
<td></td>
<td>Bete</td>
<td></td>
</tr>
<tr>
<td>Medium Level Interest</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Emien</td>
<td></td>
<td></td>
<td>Awoera</td>
<td>Alep</td>
</tr>
<tr>
<td>Iomba</td>
<td></td>
<td></td>
<td>Bubinga</td>
<td>Pao Rosa</td>
</tr>
<tr>
<td>Pomager</td>
<td></td>
<td></td>
<td>Difou</td>
<td></td>
</tr>
<tr>
<td>Ayous</td>
<td></td>
<td></td>
<td>Limbali</td>
<td>Longhí</td>
</tr>
<tr>
<td>Low Level Interest</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kosiço</td>
<td></td>
<td></td>
<td>Afromosia</td>
<td></td>
</tr>
<tr>
<td>Akossika</td>
<td></td>
<td></td>
<td>Kolbe</td>
<td></td>
</tr>
<tr>
<td>Andoung</td>
<td></td>
<td></td>
<td>Iatandza</td>
<td></td>
</tr>
<tr>
<td>N'gongre</td>
<td></td>
<td></td>
<td>Essia</td>
<td></td>
</tr>
<tr>
<td>Kekele</td>
<td></td>
<td></td>
<td>Oziço</td>
<td></td>
</tr>
<tr>
<td>Nsikibila</td>
<td></td>
<td></td>
<td>Acajou</td>
<td></td>
</tr>
<tr>
<td>Ailek</td>
<td></td>
<td></td>
<td>Tiama</td>
<td></td>
</tr>
<tr>
<td>Koto</td>
<td></td>
<td></td>
<td>Framine</td>
<td></td>
</tr>
<tr>
<td>Eyong</td>
<td></td>
<td></td>
<td>Tola</td>
<td></td>
</tr>
</tbody>
</table>

### Table 5: Scores of some species according to durability criteria

<table>
<thead>
<tr>
<th>Classes</th>
<th>Specific Botanic names</th>
<th>Vernacular names</th>
<th>Scores</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good</td>
<td>Triplochiton scleroxylon</td>
<td>Tali</td>
<td>5</td>
</tr>
<tr>
<td>Average</td>
<td>Fraké</td>
<td>Fraké</td>
<td>4</td>
</tr>
<tr>
<td>Bad</td>
<td>Daniella klariei</td>
<td>Faro</td>
<td>1</td>
</tr>
</tbody>
</table>

### Table 4: Scores of some species according to extinction risk criteria

<table>
<thead>
<tr>
<th>Conservation status</th>
<th>Specific Botanic names</th>
<th>Vernacular names</th>
<th>Scores</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critically endangered (CR)</td>
<td>Autanella congolensis</td>
<td>Mukulungu</td>
<td>0</td>
</tr>
<tr>
<td>Endangered (EN)</td>
<td>Diospyros crassiflora</td>
<td>Ebene</td>
<td>0</td>
</tr>
<tr>
<td>Vulnerable (VU)</td>
<td>Lophira alata</td>
<td>Azobé</td>
<td>2</td>
</tr>
<tr>
<td>Least Concern (LC)</td>
<td>Triplochiton scleroxylon</td>
<td>Ayous</td>
<td>4</td>
</tr>
<tr>
<td>No concern (NC)</td>
<td>Entandrophragma cylindricum</td>
<td>Téfafi</td>
<td></td>
</tr>
</tbody>
</table>

### 1.2 GLUING TESTS

In this part of the study, we present the first results of tropical green gluing, using glulam technique. The aim of gluing tests is to assess the structural wood gluing assembly. *Ayous* (*Triplochiton scleroxylon*) and *frake* (*Terminalia superba*) are selected according to their interest level after multicriteria analysis. For each bonding configuration (*ayous*, *fraké* and *ayous-fraké*), two beams are realised. Each beam consists of six individual lamellae with the following dimensions: 20 mm thick, 100 mm wide and 400 mm long. Specific polyurethane glue for gluing subtracts has been used. The wood moisture content is 48 ± 6 % (*ayous*) and 65 ± 16 % (*fraké*). The open assembly time is 2 min, and the closed assembly time 10 min. Approximately 1 hour after lamellae cutting and planning, a weighed amount of adhesive is spread on each surface at a spread rate of 400 g/m². Both adhesive-spread surfaces are immediately assembled to ensure contact between surfaces. Pressure is applied at 0.96 N/mm². Constant pressure is maintained for 20 hours. After pressing, beams were conditioned at 12°C and 65 % relative humidity for 20 days before cutting into specimens. Shear and delamination tests are realised.

#### 3.2.1 Delamination test (EN 302-2)

Delamination test consists in the appreciation of the glued joint resistance after 3 ageing cycles. In each beam, 3 delamination specimens of 76 mm thickness are cut on every beam. According to the EN 302-2 standard, moisture content of a given specimen is about 12% at the beginning of the first cycle. This condition is not respected. Due to the important drying time of these specimens, three scenarios were considered. In the first one, specimens are dried at 65°C for 22 hours. In the second one, the delamination test starts with the pressure-soak. In the last one, specimens are conditioned at 12°C and 65 percent of relative humidity for three months. In these conditions, the equilibrium moisture content in the wood is 12%. 2 specimens are used for each bonding configuration.

#### 3.2.2 Shear test (EN 302-2)

A compression shear stress, parallel to the grain direction, is applied in each bond line. 2 specimens of 45 mm thickness were cut in each beam. Every specimen has 5 bond lines. The ultimate shear resistance and wood failure ratio are measured for each broken glue line joint. The moisture content of specimens is 15±2% (equilibrium value in tropical climate conditions).

#### 1.3 RESULTS

First results, concerning *fraké* and *ayous* are represented in table 7. According to EN 302-2 standard, any delamination rate greater than 10% is considered failure of the specimen. The maximum value of delamination rate is 2, 4%. The delamination requirement is respected. The main wood removals are observed at the end of the drying cycle of delamination.
Table 7: Delamination results according to different scenarios and standards requirements

<table>
<thead>
<tr>
<th>Bond lines</th>
<th>Specimens</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ayous</td>
<td>N°1</td>
<td>0.0%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>≤10% for outdoor use;</td>
</tr>
<tr>
<td>Ayous</td>
<td>N°2</td>
<td>1.5%</td>
<td>2.4%</td>
<td>0.6%</td>
<td>≤5% for indoor use</td>
</tr>
<tr>
<td>Frake</td>
<td>N°1</td>
<td>0.0%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>≤10% for outdoor use;</td>
</tr>
<tr>
<td>Frake</td>
<td>N°2</td>
<td>2.1%</td>
<td>2.0%</td>
<td>0.3%</td>
<td>≤5% for indoor use</td>
</tr>
<tr>
<td>Ayous</td>
<td>N°1</td>
<td>0.0%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>≤10% for outdoor use;</td>
</tr>
<tr>
<td>Ayous</td>
<td>N°2</td>
<td>0.6%</td>
<td>0.0%</td>
<td>0.3%</td>
<td>≤5% for indoor use</td>
</tr>
</tbody>
</table>

Figure 2 shows delamination and tannin removals on a fraké specimen.

Figure 2: A vue of frake specimen at the end of a drying cycle (specimen n°2, scenario 1)

The thickness of the bond line, measured after microscope scanning using is about 0.2 mm (figure 3).

Figure 3: Electronic microscope scanning of a bond line (ayous)

The respect of delamination requirements allows us to perform shear tests (EN 392). Results are shown in table 8 and figures 4 and 5.

Figure 4: Cumulative frequencies of bond lines shear resistance

Figure 5: Failure in a mixed bond line

Table 8: Average shear resistance of single and mixed bond lines and glulam shear requirements

<table>
<thead>
<tr>
<th>Bond lines</th>
<th>Average Shear resistance (MPa)</th>
<th>Wood failure rate (%)</th>
<th>Average density at 15% moisture content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ayous - ayous</td>
<td>8.75 ± 0.75</td>
<td>78 ± 27</td>
<td>0.36</td>
</tr>
<tr>
<td>Fraké – fraké</td>
<td>9.98 ± 0.60</td>
<td>82 ± 14</td>
<td>0.45</td>
</tr>
<tr>
<td>Ayous - fraké</td>
<td>9.22 ± 0.88</td>
<td>86 ± 25</td>
<td>0.40</td>
</tr>
<tr>
<td>Glulam</td>
<td>8</td>
<td>≥ 72</td>
<td>-</td>
</tr>
<tr>
<td>Requirements</td>
<td>9</td>
<td>≥ 63</td>
<td>-</td>
</tr>
<tr>
<td>(EN 386)</td>
<td>11</td>
<td>≥ 45</td>
<td>-</td>
</tr>
</tbody>
</table>

4. DISCUSSIONS

The high number of interesting species for ETWP manufacturing is a serious opportunity given to the industrial actors, to decrease the pressure on the most wanted extinguishable species. Table 6 also shows an order of ranking inside a mechanical group. Some species like moabi (baillonella toxisperma), essessang (ricinodendron heudelotii) and aïlé (canarium schweinfurthii) have an important value...
concerning health, food and fruits trade for local populations [21]. Their low interest in our study may help industrials to reduce conflicts against local populations.

There is a good respect of standards requirements concerning delamination and shear resistance. A justification is that, green gluing was not influenced by wood removals. Removals have appeared only during drying. The ultimate shear strength of the different glue joints is close. Ratios shear/density varies between 22 and 24 MPa. The shear strength of mixed glued woods is practically equivalent to the highest resistance of homogeneous glued wood joint.

5. CONCLUSION

Conclusive gluing tests confirm the durability of glued woods and the aptitude to perform glulam for the homogenous and mixed wood species in tropical climate conditions. Green gluing is an interesting process of ETWP promotion and development. ETWP (especially glulam) can be manufactured in tropical conditions while conserving the wood diversity species of the Congo basin forest.

REFERENCES

PARAMETRICAL COMPARISON OF GLUED LAMINATED BEAMS WITH VARIABLE HEIGHT

Srečko Vratuša¹, Manja Kitek Kuzman², Vojko Kilar³

ABSTRACT: The paper describes some Eurocode 5 code requirements for glued laminated beams and presents a parametrical comparison of two most typical forms of double beams with variable height with levelled and saddled lower edges. The stress conditions have been compared for a number of differently-shaped double-tapered and pitch-cambered beams. The results obtained for a refined mesh of finite elements in the SAP2000 computer program have been compared with the results obtained by simplified formulae given in Eurocode 5. It was shown that in general the correctly modelled finite element model can account for all structural particulars of beams with variable height.

KEYWORDS: Glued laminated timber, Double-tapered and pitch-cambered beams, Stress perpendicular to the grain

1 INTRODUCTION

Modern glued laminated structural timber is a product of the most advanced technologies, which include the most recent findings on materials, design and the theory of structures. In the modern sustainability oriented architectural practice the usage of various forms of beams and frames with variable height made of glued laminated wood is becoming more and more popular [1], [2]. The simplified formulae included in the codes can be used only for simply supported beams with predetermined shapes [3]. In all other cases the FEM models are widely used by structural engineers, while a number of modern computer programs have enabled more accurate modelling, analysis and more economical use of materials.

The aim of the paper is to present the appropriateness of modern general purpose structural analysis FEM computer program (e.g. SAP2000) [4] for the analysis of glulam beams with variable height. The main quantities observed were stresses in the apex area as well as maximal bending stress and its location. The results obtained for a very refined mesh of finite elements in the SAP2000 structural analysis program have been compared with the formulae for simple beams given in EN 1995-1-1:2004 (Eurocode 5) [4]. It was shown that in general a cautiously modelled finite element model with appropriate grid density and orientation of local axes in the direction of grains, using the appropriate wood orthotropic material can account for all structural particulars of beams with variable height. The main advantages of the finite element approach however remain the general freedom of used forms and shapes, various grain orientations, and different material characteristics in the mathematical model.

2 PARAMETRIC STUDY

2.1 GENERAL

The procedures for design, fabrication, quality control and construction of glued laminated beams have been standardized and included in Eurocode standards. The glulam timber enables the usage of structural beams of different shapes and forms. Curved members of virtually any practical radius are possible simply by forming individual pieces into the desired shape prior to gluing them. Glulam members are available in much longer lengths and sizes than standard sawn timbers. Besides straight beams with constant height, the Eurocode 5 (Part 1-1) also includes beams of variable height with three different typical shapes: Single or double-Tapered beams, Curved beams and Pitched cambered beams (Saddeled beams).

The calculation of stresses for these elements is usually more complicated than for the elements made of other materials [5]. The main reason is the variable geometry of the cross sections, inclinations of element edge regarding the grain direction in laminations and the curvature of element axis. For double-tapered beams with varying cross-section, curved and pitched-cambered

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beams, we should account for special stress conditions in the apex area. In addition to normal bending stresses, we should also expect transversal-radial stresses, acting in the direction perpendicular to the longitudinal direction of beams. These radial stresses are in many cases the crucial parameter that determines the size of the beam in the apex area, because the design strength in the direction perpendicular to the grains is much smaller as the strength in the direction of the grains. Different researchers, especially Möhler and Blumer, Gopu and Goodman, Gutkowski et al, have performed elaborate analyses of such beams and studied the effects of the afore-mentioned parameters on beam stress conditions [7]. They have also proposed some expressions which enable the simplified analytical calculation of such beams, which were later included in many national codes and standards (also in Eurocode 5). In codes, these specific stress conditions are usually considered by different correction factors for the calculation of bending and radial stresses. The most accurate analyses were made by solving the differential equation of wall made of orthotropic material. Several experimental research studies have confirmed the results [8]. Today we can access the more complicated stress conditions of such beams also by using a finite element computer program (e.g. SAP2000 or other). In this case, the mathematical model should be prepared with care in order to account for adequate numerical accuracy as well as for different material properties in different directions corresponding to actual grain directions in glulam elements.

2.2 DATA AND MATHEMATICAL MODELLING

This paper presents a comparative analysis of two most typical shape representatives of beams with variable height, e.g. double-tapered beams and pitched-cambered beams (called also saddled beams). It is assumed that the beams are supported only by vertical reactions and that the overturning or buckling out of their plane is prevented. The geometry data were derived from a straight beam made of GL28h with a constant rectangular cross section designed for a given span (16 m) and assumed design uniform load (21 kN/m). According to Eurocode 5, the height of such beam according to bending criteria should be exactly \( h = 100 \) cm, if beam width is fixed to \( b = 20 \) cm. Seven variants of double-tapered and seven variants of saddled beams (all with \( 2c = 0.25 \) \( L = 4 \) m, see Fig. 2b) considered in the study have the same volume as the original straight beam. For this reason, we have modified their basic geometry data \((h_{ap}, \alpha_{ap}, \text{and } r_{in}) \) individually for each case as shown in Table 1. To ease the comparisons, the individual beams are labelled with one letter (T for tapered and S for saddled) and the roof slope in percent \((100\cdot \tan \alpha_{ap})\). Because we have used the same wood volume to produce the beams, the slopes of saddled beams are higher as the slopes of tapered beams. The glued laminated beams are modelled as orthotropic 4-node final elements with appropriate orientation of material characteristics in different directions. The mesh of used finite elements together with direction of local axes for analyzed shape variants are presented in Fig. 1:

<table>
<thead>
<tr>
<th>Table 1: Modified geometry data for considered beams</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Double-Tapered beams</strong></td>
</tr>
<tr>
<td>SLOPE [%]</td>
</tr>
<tr>
<td>T-0</td>
</tr>
<tr>
<td>T-2.5</td>
</tr>
<tr>
<td>T-5.0</td>
</tr>
<tr>
<td>T-7.5</td>
</tr>
<tr>
<td>T-10.0</td>
</tr>
<tr>
<td>T-12.5</td>
</tr>
<tr>
<td>T-15.0</td>
</tr>
</tbody>
</table>
The elastic modulus $E$ amounted to 1260 kN/cm² for Direction 1 and to 42 kN/cm² for Direction 2. The corresponding shear modulus amounted to 78 kN/cm² for all directions. In tapered beams, Direction 1 is always parallel to global X direction and the modelling is not problematic. More demanding is the modelling of pitch-cambered beam where the Direction 1 local axis is always perpendicular to radius of curvature of each element. In this case, we had to use cylindrical coordinates to ease the modelling of the finite elements mesh.

3 RESULTS

3.1 STRESSES IN APEX AREA

The computer analysis of glued laminated beams enables the calculation of the complete stress distribution for any cross section. This is an advantage over simplified expressions from the standards, which enable only stress calculations at a certain point where the maximum values are expected. Fig. 2 shows the selected results for analysed double-tapered beams (left) and saddled beams (right). Figs. 2a and 2b show the geometry of the analysed beams. The following four figures show the design bending stresses. Figs. 2c and 2d show the contour lines of the same bending stresses and the Figs. 2e and 2f the distribution of normalized bending stresses over the height of the beams in the apex area. The last four figures show the design transverse - radial stresses. Figs. 2g and 2h show the contour lines of the same radial stresses and the Figs. 2i and 2j the distribution of normalized radial stresses over the height of the beams in the apex area. Because the beam heights change, they are presented as relative values in the way that the zero coordinate means the lower edge and 1,0 coordinate the upper edge of the beam. Also, the design values of stresses are normalized corresponding to their design value of strength, which is also shown as reference value among results in Fig. 2. The positive values mean tension and the negative values compression. The design bending stresses ($\sigma_{t,90}$) are normalized with design bending strength ($f_{m,d}$). According to Eurocode 5, this value is obtained for a reference height amounting to 60 cm for glued laminated beams. For lower beams, the bending strength is slightly higher (maximum up to 10%). Another important factor that should be considered is the strength reduction due to bending of the laminates during production. For these elements, the reduction of bending strength depends on the ratio of the radius of curvature and the thickness of laminates. For selected geometry of beams however, these modification factors do not apply and the design bending strength for material GL28, short-term load duration class, service class 2 ($k_{mod}=0.90$) and material safety factor $\gamma_m=1.25$ amounts to $f_{m,d}=2.016$ kN/cm².

Also the design transverse - radial stresses ($\sigma_{t,90,d}$) were normalized to their design strength according to Eurocode 5. Initial tension design strength for the direction perpendicular to the direction of fibres for selected material GL28 ($k_{mod}=0.90$ in $\gamma_m=1.25$) amounts to: $f_{t,90,d}=0.032$ kN/cm². This value should be increased due to the effect of stress distribution in the apex area (dashed part of the beam in Fig. 2a). The amplification factor ($k_{ds}$) amounts to 1.4 for double-tapered and curved beams and 1.7 for saddled beams. However, the design tension strength should be reduced do to the effect of the size (volume) of apex area $V$ ($V_{max}=2V_0/3$, where $V_0$ is total volume of the beam and $V_0=0.01$ m³ is the reference volume value). The final value of design tension strength can be expressed as:

$$f_{t,90,d} = k_{ds} \left( \frac{V_0}{V} \right)^{0.2} f_{t,90,d}$$ (1)

The results in apex area show the characteristic stress distribution for normal bending and transversal radial stresses, which are consistent with the research results obtained by different authors [6],[7]. Also, the obtained stress anomaly on the bottom edge in Fig. 2j is known from the method of finite elements (MKE). The correct value is zero, because there is no loading on the lower beam edge. It can be seen from the distribution of stresses that the numerical results are limited toward the correct zero value on lower beam edge.

It can be seen from Figs. 2e and 2f that the bending stresses in the apex zone are not exceeded for any analyzed beam variant. With the increase of beam slope, the bending stresses decrease, but the radial stresses rapidly exceed the design tension strength (Figs. 2i and 2j). The increase of the apex height, which is the consequence of the changed geometry, only slightly reduces the maximal radial stress. The increase of radial stresses is not dangerous for doubled tapered beams in the analyzed range, but it seems to be critical for all pitch-cambered beams with slopes higher than approximately 10%. In cases where slope exceeds 50%, the actual radial stress is already almost five times greater than the allowable design strength. The main problem is that the design tension strength in the direction perpendicular to fibre direction is low. In our case, the only solution is to increase the dimensions of the beam in the apex area. Therefore, it can be concluded that for a given span and loading, the increase of saddled beam slope or the increase of beam curvature requires an additional increase of beam volume, which is actually unfavourable from the structural and economic points of view.

Another solution exists for the beams with high transversal - radial stresses: special reinforcing devices can be used, which prevents the splitting of lamellas in radial directions. These can be made of wood, metal or even of fibre reinforced plastic [9],[10]. Wooden reinforcing devices are usually made of harder wood plates, which can be glued on both sides of the beam. Additionally, nails can be used in order to increase pressure during the gluing process. The metal devices are usually internal steel screws for wood or ribbed steel bars. Epoxy resin should be poured in pre-drilled holes before mounting the transversal screws.
Figure 2: (a) Double-Tapered beams and (b) Pitch-cambered beams (Saddled beams); (c, d) Contours of normal – bending stresses; (e, f) Distribution of normal – bending stresses over the height of beam in the apex area; (g, h) Contours of transversal – radial stresses; (i, j) Distribution of transversal – radial stresses over the height of beam in the apex area.
3.2 COMPARISONS WITH EUROCODE 5

The comparison between the values obtained by SAP program and the values obtained by Eurocode 5 with the simplified expressions and correction factors are presented in Fig. 3. It can be seen that the correlation is fairly good and that the simplified procedure from the Eurocode is on the safe side.

![Figure 3: Comparison of normalized stresses (upper: normal bending stresses and down: radial stresses) obtained by SAP2000 and Eurocode 5 for apex area of the analyzed pitch-cambered beams](image)

Beside the stress conditions in apex area, another important beam design parameter is the maximal bending stress which does not necessarily occur in the apex area, but rather at the location where the ratio between internal bending moment and beam inertia moment reaches its maximal value. According to elastic beam bending theory (EBT) the design bending stress \( \sigma_{ \text{m,d} } (x) \) can be simply determined as a quotient between design bending moment \( M_{ \text{d} } (x) \) and section modulus \( W(x) \) of rectangular cross section with variable beam height \( h(x) \). In double tapered beams and in straight parts of saddled beams the height change is linear and it depends on angles \( \alpha_{\text{ap}} \) and \( \delta \), as well as the height at the support \( h_s \). In the apex area of the saddled beams however, the height of the beam does not change linearly any more and it additionally depend on the length of the curved apex area \( c \), which, together with angle \( \delta \), determines the radius of curvature of lower beam edge in this area. The \( h(x) \) can be most rationally expressed if the coordinate system origin is placed in the middle of the beam. The expressions obtained for saddled beams are as follows:

- in apex area where change of beam height is nonlinear \( (x < c) \):
  \[
  h(x) = h_s + \left( \frac{L}{2} - x \right) \tan \alpha_{\text{ap}} - \frac{L}{2} \tan \delta + \frac{2c}{\sin(2\delta)} - \left( \frac{c}{\sin \delta} \right)^2 - x^2
  \]  

- in “near support” area where change of beam height is linear \( (c \leq x \leq L/2) \):
  \[
  h(x) = h_s + \left( \frac{L}{2} - x \right) \left( \tan \alpha_{\text{ap}} - \tan \delta \right)
  \]  

Fig. 4 presents the obtained normalized values of bending stresses obtained by “Elastic Bending Theory (EBT)”. Only one (right) half of the beam is presented. Note that \( x/L=0 \) means the middle beam span and \( x/L=0.5 \) the end of beam near the support.

![Figure 4: Normalized bending stresses in double tapered (upper) and saddled (down) beams](image)

It can be seen that the position of maximal stress varies significantly with beam slope. While beam slope is small, the maximal stress can be expected closer to the mid span and while the slope is more significant, the maximal stress occurs closer to the support. Only for smaller slopes of saddled beams the maximum stress have actually occurred in the curved apex area \( (x/L < 0.125 /c = 2 \text{ m} / \text{left from red vertical line in Fig. 4).} \)
For saddled beams we have also made the comparison of results obtained by EBT theory and program SAP. In the most cases the agreement is very satisfactory. In some cases the position of maximal stress in SAP is difficult to obtain, because very similar values appear in different finite elements. The maximal values however never differ for more than 10%. In the cases with very high slopes, the simple linear beam theory also might have reached its limits. The whole procedure of calculating maximal stresses using Equations 5 and their coordinates for EBT is rather complicated and was processed in Mathematica [11]. In this case the usage of SAP have shown some significant advantages over EBT, because it enables the calculation of the complete stress distribution for any cross section. In this case however, the maximum stress is obtained for individual finite elements instead for a specific cross section. This is an advantage over the simplified expressions from the standards, which enable only the stress calculation at certain points where the maximum values are expected.

4 CONCLUSIONS

In our analysis, we have selected a series of beams with different heights and curvatures and compared the results obtained by SAP program and simplified formulae from Eurocode 5. Beside normal bending stresses \( \sigma_{m} \) and radial stresses \( \sigma_{t,90} \), in the apex area we have observed also maximal bending stresses and their locations. The comparison between the results have shown that the correlation is fairly good for all examined cases.

From the obtained results of the beams with the same volumes, but different shapes, it can be concluded that while the slope of the beams increases, the bending stresses decrease and the radial stresses increase. The increase of radial stresses was never critical for any analyzed double-tapered beam (slope \( \leq 15\% \)), but for the pitch-cambered beams, however, the situation in apex area becomes critical for all slopes greater than approximately 10%. The radial stresses in these cases have greatly exceeded the design tension strength perpendicular to fibre direction. The increase of the apex height, which is the consequence of the changed geometry, only slightly reduces the maximal radial stress. Therefore, for pitch-cambered beams with the slope higher than 10%, the only solution is to increase the dimensions of the beam in the apex area or to accommodate the undesirable radial stresses with additional anchors or other measures. The situation could also be improved by adding a tensile element connecting the supports, which would minimize horizontal displacement and reduce radial stresses in apex area. In general, it can be concluded that for a given span and loading, the increase of beam slope is favourable only for tapered beams. For the pitch-cambered beams, the increase of beam slope and curvature over certain value requires an additional increase of beam volume, which is actually unfavourable from the structural and economic points of view.

It was confirmed that correctly and cautiously prepared finite element model can describe all particular stress conditions which are important for practical design of glulam beams with variable height. The finite element analysis enables the calculation of the complete stress distribution for any cross section and any form of a beam or similar frame type structure. Additionally, such a finite element approach enables a combination of different wood qualities (e.g., higher wood quality on the lower and upper sides of the beam) as well as the inclusion of various grain orientations, material characteristics and shapes in the mathematical model. To obtain the correct results, however, the mathematical model and finite element mesh should be prepared with great care and adequate precision in order to account for satisfactory numerical accuracy, as well as for realistic orthotropic material characteristics in different directions corresponding to the actual grain directions in tapered pitch cambered or curved glulam members.

REFERENCES

THE TIMBER HUT THAT PEOPLE CAN ASSEMBLE BY THEIRESELVES  
THE PROJECT OF THE RECOVERY OF THE GREAT EAST JAPAN EARTHQUAKE

Akito Kikuchi¹, Hisamitsu Kajikawa²,

ABSTRACT: This is project of the recovery of the great east japan earthquake. In the housing shortage after the earthquake affected areas has become a serious problem. A possible cause, shortage of materials, lack carpenter, is considered an economic problem. We suggest the timber hut that people can assemble without using any specific equipment. and timber hut is taken into account economic efficiency, workability, about the living environment so they can use it such as the assembly place, shops and offices and so on.

KEYWORDS: timber structure, seismic disaster, temporary housing

1  INTRODUCTION

In the housing shortage after the earthquake affected areas has become a serious problem. A possible cause, shortage of materials, lack carpenter, is considered an economic problem.

The timber hut[figre1] as a way to resolve them. We suggest the small timber hut that people can assemble without using any specific equipment.

We optimized the design so that it may set to the size that two adults can carry the panel used by the existing wood bonding panel method and one 2 ton track may load the material for one unit.

The size of one unit is about 28m², and we assume that the correspondence will be for two years.

We achieved the price cutting by using the existing production line, and we plan the price will be very low price.

People are doing life in the severe environment such as refuges and house we half destroyed in the disaster area. This kit is impossible for the people to get a safe and healthy residence space by assembling the hut in the land prepared by their selves.

This timber hut has a simple structure, so they can use it such as the assembly place, shops and offices and so on.

We think that this suggestion is effective for voluntary activity of the people in the disaster area and improvement of living conditions to get back the daily life quick.

2  CONSTITUENT MEMBERS

2.1 OUTLINE OF THE PANEL

The weight of one panel is 50 kg or less. It is all the sizes and weights that can be carried by two adults.

It corresponds to the cold latitude by using the panel with the heat insulator[figure2] and enhance productivity, and economy by reducing the type of panel.

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2.2 MEMBER LIST AND TRANSPORTATION METHODS

It is possible to offer quickly to raise the productive efficiency in the factory by reducing the kinds of panel. The material for 4 units can be transported in 10 ton track and one unit in 2 ton track. [table 1]

<table>
<thead>
<tr>
<th>Name</th>
<th>Specification</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel</td>
<td></td>
<td>57</td>
</tr>
<tr>
<td>Joint Bolt</td>
<td></td>
<td>224</td>
</tr>
<tr>
<td>Roof Joint Bolt</td>
<td></td>
<td>36</td>
</tr>
<tr>
<td>Timber Base1</td>
<td>90x55x3000</td>
<td>6</td>
</tr>
<tr>
<td>Timber Base2</td>
<td>120x120x3000</td>
<td>6</td>
</tr>
<tr>
<td>Waterproof Tape</td>
<td>50x2000</td>
<td>3</td>
</tr>
<tr>
<td>Concrete Block</td>
<td>300x190x120</td>
<td>43</td>
</tr>
<tr>
<td>Window</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Tatami</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Rope</td>
<td>10mmx30m</td>
<td>1</td>
</tr>
<tr>
<td>Ratchet</td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

3 HOW TO INVEST THE UNIT

NPO becomes point of contact for victims, NPO receives an order from a victim, ordered the Timber hut to MISAWA HOME. MISAWA HOME receives an order, send a volunteer and, if necessary, timber hut to the victims.

4 HOW TO CONSTRUCT

It is possible to correspond to lots of land because it doesn’t need any heavy equipment. People can construct only with lots tightening and easy nailing. [figure5]

5 PLANS

The equipment such as upper and lower waterworks, electricity and gas etc can be added.

The equipment such as kitchen, refrigerators and washing machine can be added.

Space that can set up rest room and prefabricated bath as option. Space they spread out the futon on tatami mat. [figure6, figure7]
1. The subjacent concrete block is laid and crowded.

2. It units with the concrete block by setting up the foundation.

3. The floor panel is laid and crowded.

4. The wall panel is set up from the corner part.

5. The roof panel is set up from the lower part.

6. People go up to the roof and set up the roof panel.

7. The waterproof tape is pasted on the roof panel joint part.

8. The blue cover is pasted and it completes.

*Figure 5: How to Construct*
ACKNOWLEDGEMENTS

This study was been supported by Daisuke Yamagishi[Yuko nagayama architects], Hiroshi Kawakami, Hosogaya Takuro, Atsusi Sugamata, and Motomi kuroda[Chiba institute of technology ]
TIMBER STRUCTURE SEISMIC DESIGN ASSISTED BY MICRO-TREMOR MEASUREMENT

Kazumasa WATANABE¹, Takashi WATANABE², Kimihiro MIYASAKA³

ABSTRACT: This document is to propose a way of seismic design for master carpenters, experts of timber structure. It is mainly composed of following short notes; 1) Master Carpenters’ Techniques, 2) Difficulties in Seismic Design, 3) Usefulness and problems of the Micro-Tremor Measurement, 4) Considerations on how to compensate the problems of Micro-Tremor Measurement, 5) Example of application of Micro-Tremor Measurement in seismic design of timber structures.

KEYWORDS: SEISMIC DESIGN, MICRO-TREMOR MEASUREMENT, TUNING FOLK, MASTER-CARPENTER, JAPANESE CULTURAL PROPERTIES

1 INTRODUCTION

Who could predict the awful phenomena that the devastating tsunamis brought upon the coastal cities? Off the Pacific coast of Tohoku, the earthquake of March 11, 2011, a submarine earthquake was not merely huge, with Magnitude 9.0, but it brought tragedy to the whole North-Eastern coastal region of Japan. Was the escalation of the disaster because the constructions were made of wood? No, even for constructions made of concrete; as it was liquid that flamed, the fire could spread under such phenomena. As for the timber buildings constructed by the master carpenters, their damage due to this earthquake was relatively small except in the area where the Tsunamis attacked directly. The old houses standing at higher places could survive the Tsunamis and withstand well against the earthquake. But as there are some damaged severely, we cannot say that all the master carpenters could execute the seismic design correctly. The dynamic seismic design is far more complicated than the static one.

2 MASTER CARPENTERS’ TECHNIQUES

More than 90% of old buildings classified as National Cultural Properties in Japan are constructed with timber structure by the Master Carpenters. Their techniques are based on the post and beam construction system. But their post and beam system has been evolved gradually from heavy timber system to light timber one. Since about 300 years ago, they compose a beam with a continuous beam and a simple one tied together and also they compose a post with a continuous post and a simple one tied together and often filled with mud wall. The continuous members are used as traction members and simple members are compression ones. The ties between the members are often small members serving as traction members penetrating the continuous and simple members and the in-filled mud wall serves as compression member.

Figure1: Framing with continuous members only

Figure2: Framing with continuous and simple members

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The modulus of elasticity of a member composed of two members can be far bigger than a simple member. And the joint system between the post and the beam can earn the rigidity far easier than in case of joint between the simple post and beam. If the in-filled wall is too solid, the post and beam members can be damaged due to the shear deformation of the frame. But in the usual cases, as the in-filling component is the mud wall reinforced by bamboo lattice and straws, even if the frame deformed largely by a certain horizontal load, the frame and framing members are scarcely damaged. Only the in-filled wall will be damaged so that the frame can be repaired relatively easily.

3 DIFFICULTIES IN SEISMIC DESIGN

The seismic design is rather difficult because we should manipulate the vibration. If it is a music instrument, we can use a tuning folk to tune the vibration. But the building is too big to tune the vibration. Moreover as the seismic design is based on the analysis of the interaction between the ground and building structure, the seismic design should begin with the assumption of quake of ground which shakes the building structure. The quake of ground can be considered as the vibration filtered and amplified through the multiple layers of different soil that can be on the passage of the earthquake vibration transmission from the epicentre to the construction site. In the actual seismic design in Japan several ways to decide the seismic wave are proposed. But in order to decide it the dynamic properties of the ground are necessary. The data obtained through the boring are not sufficient to seize the dynamic properties of the ground. The behaviour of the structure under an earthquake is not clearly visible. As it is too quick and lasts too short period, it is difficult to analyse it.

If the dynamic behaviour can be observed easily, the master carpenters can improve their seismic design in quite a practical way. This is why we propose to use the micro-tremor measurement to visualise both the dynamic behaviour of the ground and that of the building on the site which interact mutually.

4 MICRO-TREMOR MEASUREMENT

The micro-tremor measurement is usually based on the extremely sensitive accelerometer which can seize the natural vibration of the ground and the structure. If the sensors are 3-dimensional, we can observe the 3-dimensional behaviour quite easily.

The micro-tremor measurement is used for the seismic micro-zonation to define the ground condition of the construction site.

The seismic micro-zonation is determined by the several ground characteristics such as predominant period, amplification factor and shear wave velocity. All these parameters can be obtained through the micro-tremor measurement.

The micro-tremor measurement can be applied to the existing buildings but only in the range of elastic behaviour. If the number of sensors increases, the information that the measurement can give us become quite rich; the gravity centre of the building, the centre of rigidity of the building, the transmission time, the dumping factor, the amplification ratio, the specific cycle, etc. What is interesting is that even the weight of the building can be seized under certain hypotheses.

The seismic design for timber structure cannot stay in the elastic zone. The cracking of the mud wall in-filled between the posts and beams is often considered as negligible but it means the behaviour exceeds the elastic zone and even the yield point.

The material properties of timber vary quite easily by several reasons; the tangible and often hidden defects of the timber, the moisture content that varies according to the circumstances, the manufacturing processes that damage the timber and so on.

The material properties of the timber show the clear sense of direction and quite sensitive to the cracking and the embedding phenomena.

The mechanical properties of the joint is also complicated; as you have already found, the master carpenters compose a member with mainly two members and one is continuous and another is simple, the mechanical properties show the difference in direction. Even if it supposed as a pin joint in a direction, it can be rigid joint in another direction. In a joint the traction system and compression system is composed, the rigidity of the joint can vary according to the working ratio of the two systems. If the traction works well the contact between the members for the compression members and the rigidity can increase.

Figure3: Joint showing the different characteristic according to the direction

These detailed considerations are executed by the master carpenters but it seems quite difficult to do the same with even the fastest computer. These should be considered as the probabilistic parameters.

The micro-tremor measurement is quite useful for the elastic behaviour but a certain compensation system is necessary for executing the seismic design for timber structure.

Nevertheless the micro-tremor measurement has a certain difficulties in the practice.

When we measure the micro-tremor of a building and its site, it may be better to use many sensors at the same time. In such case the calibration and the positioning of the sensors is quite important. Their 3-dimensional orientation should be equal between the sensors. If it is difficult the difference of orientation should be known.

And as the accelerometer is too sensitive, the obtained data can include so many noises.
5 HOW TO COMPENSATE THE WEAK POINTS OF THE MICRO-TREMOR MEASUREMENT

The weak point of the micro-tremor measurement is that this can seize the phenomena only in the elastic zone. But in the timber structure the master carpenters utilise the embedding characteristics and even the cracking and sometimes they utilise the deformation due to the drying or ageing.

We have to look back the actual seismic design process. In the static calculation we use the given design load calculated with the weight of the building and the stress of each member due to this design load is calculated. When the stress exceeds the material strength for design, the dimension of this member is judged insufficient and the change of dimension or the change of material is examined.

What we do is relatively simple compared to what the master carpenters do. If the precision of calculation can be increased using the micro-tremor measurement, it should be appreciated.

The modulus of elasticity is quite often used in the process of static resistance calculation. The modulus of elasticity being an intrinsic property of the material, we know already there is certain relationship between the modulus of elasticity and the strengths of the material. If some coefficients are prepared according to our encyclopaedia on the material properties, we can calculate the behaviour more precisely using the modulus of elasticity obtained through the micro-tremor measurement.

6 EXAMPLE OF APPLICATION

The Okazaki House in Tottori, a samurai’s residence constructed in 1835 is now very severe condition, because the house was bought during the demolition process and almost all the walls are taken off and is obliged to be bound up by steel wires.

Figure 4: Actual State of the Okazaki House in Tottori, a Samurai's Residence constructed in 1835

We are planning to repair the house as it was before the demolition work was executed.

Figure 5: Okazaki House before the demolition work

But it takes too much time to repair the mud wall; we are now considering repairing temporally with thin paper panels in-filled with mud. In this process, we have to adjust the seismic characteristics and we are thinking to apply the micro-tremor measurement, particularly when we take off the bandages of steel wire. If the dynamic behaviour of the house does not suit to the dynamic behaviour of the ground, we have to adjust the behaviour by adding the paper panel layer or taking off the panels.

When the temporal panels are replaced with the mud walls as the original state of the house also the work should be executed under observation with micro-tremor measurement.

The Okazaki House is actually covered by a temporary roof constructed with round logs in order to protect the house from the bad weathers. The joints of this temporary roof are just bound with steel wire. If these joints are loosened, the dynamic behaviour of the roof will change. This roof structure should be checked according to the necessity with the micro-tremor measurement.

Figure 6: The temporary roof covering the Okazaki House

7 CONCLUSIONS

The micro-tremor measurement can show the dynamic behaviour of the structure quite easily and can serve as a tuning folk for the master carpenters at least in the elastic zone. If the number of sensors increases, the micro-tremor measurement can give us so many parameters useful for the calculation of stress and strength relationship and can serve as visualizer of the dynamic behaviour. By using the micro-tremor measurement on the process of construction the master carpenters can tune the rigidity of the joints or the members so as to give the predominant period and the amplification or dumping factors appropriate to the ground condition.
In such a way the design process can enter in the process of the construction, which the actual way of design cannot take into account.

ACKNOWLEDGEMENT
The authors are profoundly grateful to the generosity of the committees of the WCTE2012 who gave us the opportunity to present this rough report.

REFERENCES
MECHANICAL BEHAVIOUR OF ANCIENT TIMBER STRUCTURAL ELEMENTS AFTER FIRE

José Amorim Faria¹

ABSTRACT: This paper presents the main conclusions of two experimental campaigns coordinated by the author in 2009 and 2010, using prismatic Portuguese oak beam (probably quercus faginea) samples obtained from ancient round timber existing beams that had been submitted to heavy heating associated to a fire on the building where they had been originally applied. The tests were conducted using EN408:2003 and included tension and compression parallel to grain and the determination of the Young Modulus parallel to grain. A simplified grading system proposed by the author was used. Results indicate that structural timber submitted to fire is highly affected by the high temperatures and looses integrity around nails which diminishes severely the more relevant properties. Therefore it is the advice of the author that, in general, reusing ancient timber beams submitted to fire for structural purposes is not recommended.

KEYWORDS: structural timber submitted to fire, visual grading of old timber, mechanical resistance of timber structural elements, mechanical resistance of ancient oak timber submitted to fire

1 INTRODUCTION
The author has coordinated in 2009 and 2010, two test campaigns on Portuguese oak samples, obtained from ancient round timber existing beams with larger dimensions, that had been submitted to heavy heating associated to a fire on the building where they had been applied.

For the bending tests, in 2009, the sample had 52 beams with 6*10*190, in cm, and in 2010, the sample had 50 beams with 5*8*110, in cm. The elimination of the rejected beams with higher defects was not made, and therefore all the beams of each group were tested. In 2009, compression tests were also performed, using 51 test specimens with 12*12*30, in cm.

A simplified visual grading was performed using a method, proposed by the author, resulting 37 samples approved in 2009 and only 16 samples approved in 2010. The tests were done in FEUP, using EN408 and involved in 2009, compression tests (compression parallel to grain) and bending tests (bending parallel to grain and Young Modulus), while in 2010 only similar bending tests were performed.

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This article presents the main more reliable results obtained in this research campaigns.

2 MAIN PURPOSE OF THE RESEARCH
The majority of the ancient constructions (built before 1950) in the Portuguese architectural field has structural systems based on wood. In this context, only the external walls and some of the internal ones are in many cases built in granite structural masonry while stairs, many structural walls, all the floors and the roofs have timber structures.

This situation is particularly important in the major Portuguese cities urban centres were one can find many heritage buildings that have very high patrimonial value and should be protected and maintained.

In structural terms, during refurbishment and rehabilitation processes, the structural elements to incorporate on the works come in many situations from other existing buildings that have been totally or partially demolished. This “old” timber should only be used when the structural engineer knows with sufficient precision the condition and the mechanical and physical properties of those elements. This concern assumes a higher importance and complexity when those elements come from the reuse of round timber that had been submitted to a fire and that was prepared for reuse, removing the external carbonised parts that resulted from the fire.
After removing the external carbonised parts, it is almost impossible to determine, by simple visual inspection and even using more complex non-destructive evaluation methods, if a structural timber element was previously submitted to a fire (and therefore to very high temperatures).

Many authors refer that it is safe to reuse those kinds of elements, but it is important to validate that idea. In that context, the author decided to coordinate two experimental campaigns that consisted of bending tests of timber structural elements that had been previously submitted to fire. The elements were prepared by removing the carbonised parts of round timber beams after being attacked by a fire.

The main purpose of this research was therefore the preparation of knowledge that could help to answer the question:

- It is safe to use smaller timber elements with rectangular section produced from existing bigger structural timber round elements that had been submitted to a fire?

The reader of the paper will find the answer in the conclusions of the paper (sections 5 and 6).

3 SIMPLIFIED GRADING METHOD

A simplified visual grading method for in situ analysis of existing Portuguese oak timber beams was originally proposed by the author in 2008 and published in [1], in its original form. Publication [5] presents the actual more simplified version of the method.

The method is based on the author’s practical experience evaluating solid timber existing building structures and uses similar information available on grading standards for new or existing structural timber published in France, Spain, Italy and Portugal.

Table 1 presents the basic criteria of the method. The results of the method are, for each structural element, approved or rejected. The mechanical and physical properties of the approved elements comply, in broad terms, with class D18 of EN338:2009.

<table>
<thead>
<tr>
<th>Defect</th>
<th>Limits to be approved</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knots diameter (ø)</td>
<td>ø ≤ 40 (mm)</td>
</tr>
<tr>
<td>Cuts</td>
<td>Ld &lt; 33% de L</td>
</tr>
<tr>
<td>Fibre inclination</td>
<td>i ≤ 20%</td>
</tr>
<tr>
<td>Cracks/fissures general</td>
<td>Lf ≥ min (L/4 ; 1 m)</td>
</tr>
<tr>
<td>Fissures not passed</td>
<td>f≥3/5 (b or h)</td>
</tr>
<tr>
<td>Fissures passed</td>
<td>a ≥max (f1,f2) e /ou (f1+f2) ≤ 2/3 b</td>
</tr>
</tbody>
</table>

Legend: b - width; h – height; L – length; ø – Knots diameter; i – fibre inclination; Ld – cut (length); f – fracture depth; Lf – fracture length; f1 and f2 – fracture length in opposite faces; a – distance between fractures f1 and f2

According to the proposed method, the properties to be used in design studies follow the recommended values that can be found in standard EN338:2009 (class D18).

The method considers the criteria and grading rules defined in:
- NP4305:1995 “Madeira serrada de pinheiro bravo para estruturas: Classificação visual”;
- NFB52-001:2007 “Règles d’utilisation du bois dans les constructions. Classement visual pour l’emploi en structures des principales essences résineuses et feuillues.”;
- UNE 56544:2003 “Clasificación visual de la madera aserrada para uso estructural:madera de coníferas.”;

Of the above mentioned standards, only UNI1119 (Italian) refers to existing timber. This subject represents a relatively insufficiently studied subject.

The method uses the commonly considered timber defects such as knots, cuts, fibre inclination, cracks/fissures in general passed or not passed and biologic defects such as rot.

Originally the method, presented in [1], was more complex than what is shown in Table 1 but practical experience in 2009 (real works, such as those described in document [4], local grading and laboratory grading) recommended a simplification of the method. That simplification was originally proposed in [2]. The definition of defects and all the evaluation and measuring criteria are extensively defined in [5].

4 EXPERIMENTAL CAMPAIGNS

4.1 GENERAL

This section describes in broad terms the test set-up, the samples, the test methodology and the main facts that were noticed during the tests.

A more complete description of the tests may be obtained in documents [2] and [3], in Portuguese.

The samples, in both campaigns, came from the floors of an existing small palace that suffered a severe fire (fig. 1). The samples in 2009 were of much higher quality.

Figure 2 shows the preparation of the timber elements to be tested.
The determination of the physical properties was performed using NP616:1973. The tests were performed in FEUP, Porto, Portugal using EN408:2003 and involved in 2009 compression tests (compression parallel to grain) and bending tests (bending parallel to grain and Young Modulus). In 2010 only the bending tests were performed. Many important conclusions and remarks can be pointed from the study of those two test campaigns.

4.2 SAMPLES, TEST SET-UPS AND RESULTS – 2009 [2]

For this campaign, 52 samples of beams were prepared for the bending tests, each having 6*10*190 cm³. The samples were numbered G1 to G52 and were graded using the method, proposed by the author, and generally defined in section 3. From the grading process, resulted 37 approved samples and 15 rejected. Figure 3 shows the test set-up and an image of a group of samples after preparation, for the bending tests.

Figure 4 shows the Force-displacement diagram for the global set of tests.

Tables 2, 3, 4 and 5 present the results obtained in the 2009 campaign for the global group of the 52 samples and for the set of the 37 approved samples (bending) and for the global set of 51 compression samples.

Table 2: 2009 results – density (symbols as in EC5)

<table>
<thead>
<tr>
<th>Property</th>
<th>Values (Kg/m³)</th>
<th>Values (Kg/m³)</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Global set (52)</td>
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</tr>
<tr>
<td>ρ mean</td>
<td>598</td>
<td>597</td>
</tr>
<tr>
<td>C V (ρ m)</td>
<td>7.0%</td>
<td>7.3%</td>
</tr>
<tr>
<td>ρ u</td>
<td>529</td>
<td>525</td>
</tr>
<tr>
<td>E (max, min values)</td>
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<td>674/527</td>
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### Table 3: 2009 results – bending (symbols as EC5)

<table>
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<td>$f_{m,0,\text{mean}}$</td>
<td>47.1</td>
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<td>$C_V(f_{m,0})$</td>
<td>31.5%</td>
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<td>$f_{m,0}$</td>
<td>22.8</td>
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</tr>
<tr>
<td>$f_{m,0}$ (max./min. values)</td>
<td>62.1/4.4</td>
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### Table 4: 2009 results – Young Modulus (symbols - EC 5)

<table>
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</tr>
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<td>10100</td>
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<tr>
<td>$C_V(E_0)$</td>
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<td>13400/7300</td>
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</table>

### Table 5: 2009 results – compression (symbols - EC 5)

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</tr>
<tr>
<td>$f_{c,0}$</td>
<td>24.6</td>
</tr>
<tr>
<td>$f_{c}$ (maximum and minimum values)</td>
<td>49.1/16.8</td>
</tr>
</tbody>
</table>

### 4.3 SAMPLES, TEST SET-UPS AND RESULTS – 2010 [3]

For this campaign, 50 samples of beams were prepared for the bending tests, each having 5*8*110 cm³. The samples were numbered P1 to P50 and were graded using the method, proposed by the author, and generally defined in section 3. From the grading process, only 16 were approved samples and 34 were rejected. The samples had been chosen as having bad quality intentionally, so that the influence of the grading/inspection methodology could be validated. Figure 8 shows the test set-up and an image of a detail group of a sample for the bending tests, showing clearly its low quality and high amount of defects both natural and caused by fire.

Figure 9 shows the Force-displacement diagram for the global set of tests.

### Table 6: 2010 results – density (symbols as in EC5)

<table>
<thead>
<tr>
<th>Property</th>
<th>Values (Kg/m³)</th>
<th>Values (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Global set (50)</td>
<td>Apv. Set (16)</td>
</tr>
<tr>
<td>$\rho$</td>
<td>599</td>
<td>586</td>
</tr>
<tr>
<td>$C_V(\rho)$</td>
<td>7.7%</td>
<td>9.5%</td>
</tr>
<tr>
<td>$\rho_{k}$</td>
<td>524</td>
<td>495</td>
</tr>
<tr>
<td>$\rho$ (max./min. values)</td>
<td>693/478</td>
<td>671/478</td>
</tr>
</tbody>
</table>

### Table 7: 2010 results – bending (symbols as EC5)

<table>
<thead>
<tr>
<th>Property</th>
<th>Values (MPa)</th>
<th>Values (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Global set (50)</td>
<td>Apv. Set (16)</td>
</tr>
<tr>
<td>$f_{m,0,\text{mean}}$</td>
<td>29.8</td>
<td>36.3</td>
</tr>
<tr>
<td>$C_V(f_{m,0})$</td>
<td>42.1%</td>
<td>34.5%</td>
</tr>
<tr>
<td>$f_{m,0}$</td>
<td>9.2</td>
<td>15.8</td>
</tr>
<tr>
<td>$f_{m,0}$ (max./min. values)</td>
<td>60.0/3.9</td>
<td>60.0/15.8</td>
</tr>
</tbody>
</table>
Table 8: 2010 results – Young Modulus (symbols - EC 5)

<table>
<thead>
<tr>
<th>Property Values (MPa)</th>
<th>Global set (50)</th>
<th>Apv. Set (16)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0,\text{mean}$</td>
<td>10600</td>
<td>11500</td>
</tr>
<tr>
<td>$C_V (E_0)$</td>
<td>27.6%</td>
<td>19.6%</td>
</tr>
<tr>
<td>$E_{0,k}$</td>
<td>5800</td>
<td>7800</td>
</tr>
<tr>
<td>$E_0$ (max./min. values)</td>
<td>15200/2600</td>
<td>14700/6500</td>
</tr>
</tbody>
</table>

5 MAIN RESULTS

From the analysis of the data it can be concluded in specific terms the following:

- It is not possible to identify samples very severely damaged in its internal parts, such as samples P22 and P26 (2010);
- Internal natural defects constitute the highest danger in the grading process, such as sample P2 (2010);
- The propagation of fire through natural fissures and the damage caused by nails (at higher temperatures…) causes severe damage on timber elements submitted to fire;
- Heat produces changes in the internal chemical structure of the timber elements submitted to fire, eventually reducing its mechanical properties and that contributes to reduce the interest in reusing these kind of materials in new structures;
- The bending resistance is the mechanical property that is more influenced by fire; the high differences between the maximum and minimum values obtained on the tests and the very high variation coefficients demonstrate this idea;
- The Young Modulus is not very much influenced by fire in elements that were not severely damaged by fire;
- The failure mode of timber elements previously submitted to fire is very difficult to predict;
- For timber elements with almost no defects, the failure mode is totally similar to elements that were not submitted to fire and in general, on the tests, correspond to traction failure on the central part of the element (zone of circular bending on the centre of the beam);
- The behaviour of timber elements in compression parallel to grain is not very much influenced by fire; that is also not true for elements very heavily damaged specially if they have passed fissures parallel to grain;
- It is the conviction of the author that the highest danger of reusing timber elements very heavily damaged on a specific section; in those cases, damage may have been caused by “burning nails”, rot or insect attacks after fire and not by the loss of mechanical resistance due to fire.

6 CONCLUSIONS

The main conclusions are the following:

- The bending resistance is the mechanical property that is more influenced by fire; the reduction of the capacity of the structural element may be very high and it is very difficult to separate the elements that will probably have an inadequate behaviour; this fact was well demonstrated by the huge differences between the highest and lowest obtained values and the very high value of the coefficient of variation;
- The Young Modulus is similar to the values for new elements, obtained from literature;
- It is very difficult to detect internal problems of the structural elements caused by fire; the more relevant effects difficult to detect relate to the effects of “burning” nails that destroy locally the timber and reduce considerably the integrity of the elements.

It is possible to conclude therefore that reusing timber after fire for structural purposes is completely not recommended. This conclusion results from the fact of being extremely difficult to identify the more or less severely injured elements from simple visual external analysis. In this context, it can be assumed that, although the external look of these structural elements is very similar to new ones after removing the carbonised part of the element, it may happen that important damage has occurred and has destroyed or severely damaged the internal parts of the elements or that the heavy heating may have reduced considerably the integrity and/or resistance of a particular section, contributing to unpredictable fragile failures.

ACKNOWLEDGEMENT

The author wishes to thank the two Directors and all the researchers and employees of the two FEUP Laboratories where the tests were performed (LESE – Seismic and Structural Engineering; LSC – Systems and Components) and the two students involved in the daily coordination of the experimental campaigns (Samuel Pereira and Sandra Leal).

REFERENCES


Note: All references available on http://paginas.fe.up.pt/~jmfaria
ABSTRACT: This paper presents the main conclusions of four experimental campaigns coordinated by the author in 2008, 2009, 2010 and 2011, using prismatic Portuguese oak beam (*Quercus faginea* and *Quercus pyrenaica*) samples obtained from ancient round timber existing beams in the three initial campaigns and from new timber directly obtained from living trees in the Portuguese forest, that originated sawn timber after all the transforming needed operations (harvesting, drying and sawing). The tests were conducted using EN408:2003 and included tension and compression parallel to grain and the determination of the Young Modulus parallel to grain. A simplified grading method proposed by the author was used. The method was also used on a case study on an existing timber building structure. Results indicate that the grading method is not adequate to be used on ancient timber, although it is very convenient to guide the visual evaluation of existing timber building structures. Nevertheless, the method has been applied in 2011 to new Portuguese Oak timber samples with very good results.

KEYWORDS: visual grading of ancient structural timber, mechanical resistance of Portuguese oak, mechanical resistance of ancient Portuguese oak timber structural elements.

1 INTRODUCTION

The author has coordinated in 2008, 2009, 2010 and 2011, four test campaigns on Portuguese chestnut and oak samples, obtained in the three initial campaigns from ancient round timber existing beams with larger dimensions, with known origin in 2008 and 2009 and relatively badly known in 2010. In 2011, the samples came from new timber of Portuguese Oak (*Quercus faginea*) and the process was accompanied all the time by the author, from the tree to the end of the test procedures, including the intermediate sewage and drying processes.

For the bending tests, in 2008, the sample had 26 beams with irregular forms and dimensions of about 15*20*100 in cm, in 2009, the sample had 27 beams with 5*5*100 in cm, and in 2010, the sample had 50 beams with 5*5*140 in cm. In 2011, the sample had 51 beams with 4,8*8,5*190 in cm.

The elimination of the rejected beams with higher defects was not made, and therefore all the beams of each group were tested.

This article presents the more important and reliable results that were obtained.

2 MOTIVATION

The evaluation of the structural safety of timber structures in existing buildings has as its main difficulty the analysis of the state of decay and of the integrity of the existing structural elements, during the actions of pre-existence verification and naturally in the definition of the mechanical properties to be used in the stability analysis calculations.

Therefore, in structural terms, the existing timber in an ancient building or that is meant to be used in its recovery although having source in other buildings, previously fully or partially demolished, should only be used when its corresponding more relevant mechanical and physical properties are known with adequate precision.

The evaluation of the mechanical properties of ancient Portuguese oak, considering its intense use in floors, stairs and roofs of traditional Portuguese buildings built between 1800 and 1950, especially in the North of the country, represents a fundamental area of research at the University level with the purpose of supplying practitioners of rehabilitation processes (above all structural engineers) with adequate tools to enhance the reuse of ancient existing timber, regardless of its origins.

The motivation of the research presented in this paper has been, in this context, the intention to prepare a very simple inspection and grading tool that could be used to common practitioners to grade existing Portuguese oak timber.
timber elements and define the relevant properties to be used in calculations.

It is also important to note that Portugal has not a grading standard for new Portuguese oak timber structural elements and that the tradition is to use standard coming from other European Latin countries (Spain, France, Italy) or to adapt the NP4305:1995 standard used to grade maritime pine (*Pinus pinaster Ait*).

In this context, the author has developed in the University of Porto, Portugal a research project between February 2008 and July 2011 related with that subject.

### 3 SIMPLIFIED GRADING METHOD

A simplified visual grading method for *in situ* analysis of existing Portuguese oak timber beams was originally proposed by the author in 2008 and published in [1]; in its original form. Publication [5] presents the actual more simplified version of the method.

The method is based on the author’s practical experience evaluating solid timber existing building structures and uses similar information available on grading standards for new structural timber existing in France, Spain, Italy and Portugal.

The method considers the criteria and grading rules defined in:

- NP4305:1995 “Madeira serrada de pinheiro bravo para estruturas: Classificação visual”,
- NFBS2-001:2007 “Règles d’utilisation du bois dans les constructions. Classement visual pour l’emploi en structures des principales essences résineuses et feuillus.);
- UNE 56544:2003 “Classificación visual de la madera asserrada para uso estructural: madera de coníferas.”;

Table 1 presents the basic criteria of the method. The results of the method are, for each structural element, approved or rejected. Approved elements comply with class D18 of EN338:2009.

**Table 1: Grading Method – Portuguese oak**

<table>
<thead>
<tr>
<th>Defect</th>
<th>Limits to be approved</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knots diameter (a)</td>
<td>≤ 40 (mm)</td>
</tr>
<tr>
<td>Cuts</td>
<td>Ld ≤ 33% L e Ld &lt; 100 cm</td>
</tr>
<tr>
<td>Fibre inclination</td>
<td>i ≤ 20%</td>
</tr>
<tr>
<td>Cracks/fissures general</td>
<td>Lf ≥ min (L/4 : 1 m)</td>
</tr>
<tr>
<td>Fissures not passed</td>
<td>f≤≤/5 (b or h)</td>
</tr>
<tr>
<td>Fissures passed</td>
<td>a ≥ max (f1,f2) e /ou (f1+f2) ≤ 2/3 b</td>
</tr>
</tbody>
</table>

Legend: b - width; h – height; L – length; a – Knots diameter; i – fibre inclination; Ld –cut (length); f – fracture depth; Lf – fracture length; f1 and f2 – fracture length in opposite faces; a – distance between fractures f1 and f2.

According to the proposed method, the properties to be used in design studies follow the recommended values that can be found in standard EN338:2009 (class D18). Of the above mentioned standards, only UNI1119 (Italian) refers to existing timber. This subject represents a relatively insufficiently studied subject.

The method uses the commonly considered timber defects such as knots, cuts, fibre inclination, cracks/fissures in general passed or not passed and biologic defects such as rot.

Originally the method, presented in [1], was more complex than what is shown in Table 1 but practical experience in 2009 (real works, such as those described in document [3]), local grading and laboratory grading) recommended a simplification of the method. That simplification was originally proposed in [2].

The definition of defects and all the evaluation and measuring criteria are extensively defined in [5].

### 4 EXPERIMENTAL CAMPAIGNS

#### 4.1 GENERAL

This section describes in broad terms the test set-up, the samples, the test methodology and the main facts that were noticed during the tests.

A more complete description of the tests may be obtained in documents [1], [2], [4] and [5], in Portuguese.

The tests were performed in FEUP, using EN408:2003 and involved in 2008 and 2009, compression tests (compression parallel to grain) and bending tests (bending parallel to grain and Young Modulus), while in 2010 and 2011 only the bending tests were performed.

Tests were conducted using the standard’s methodology. The samples were prepared using the standard’s criteria although in 2008, the rules were not applied.

#### 4.2 SAMPLES, TEST SET-UPS AND RESULTS – 2008 and 2009

The method, proposed by the author, was used to classify the samples resulting 21 samples approved in 2008 and 13 samples approved in 2009.

The samples, in the 2008 and 2009 campaigns, came from the floors of an existing building in Porto’s Old town (Ribeira quarter), built around 1790 (see figure 1).

![Fig. 1 – sample - compression (2008); existing beams before removal (2008 and 2009)](image)

The 2008 campaign had the purpose of exploring in broad terms what could be expected in the future.
The samples were therefore very rough and had short length, because they used the same section as the original beams. Hence, 52 samples were prepared, 26 for the compression tests and 26 for the bending tests, representing several different beams, and in general were from timber that could be not used to prepare smaller samples for many reasons (end of beams, heavily cracked beams, etc).

For bending tests the samples had 1 meter length and around 15/18*20/24 cm².

For compression tests samples were cut with more precision and had 20*20*30 cm³.

Figure 2 shows the test set-ups for the 2008 test campaign.

The determination of the physical properties was performed using NP616:1973.

Using the grading methodology defined in 3., 21 samples were approved and 5 rejected (bending samples).

Table 2 presents the results obtained in the 2008 campaign (global set of beams).

Table 2: 2008 results (symbols as in Eurocode 5 – EC5)  
<table>
<thead>
<tr>
<th>Property</th>
<th>Values (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m,\text{mean}}$</td>
<td>28</td>
</tr>
<tr>
<td>$f_{m,k}$</td>
<td>23</td>
</tr>
<tr>
<td>$f_{c,0,\text{mean}}$</td>
<td>39</td>
</tr>
<tr>
<td>$f_{c,0,k}$</td>
<td>21</td>
</tr>
</tbody>
</table>

In the 2009 campaign, smaller samples were prepared using the beams obtained from the demolition of the floors of the building. In the 2008 campaign the beams had similar cross sections (bending) as in situ.

Hence, 55 samples were prepared, 28 for the compression tests and 27 for the bending tests, representing several different beams. As the original beams were very ancient, the cracks were very frequent and the samples showed many defects.

For bending tests the samples had 5*5*100 cm³. For compression tests samples were cut with more precision and had 12*12*30 cm³.

The determination of the physical properties was performed using NP616:1973.

Using the grading methodology defined in 3., only 13 samples were approved and 14 rejected (bending samples). Figure 3 shows the test set-ups for the 2008 test campaign.

Figure 4 shows images of samples after the completion of tests in compression and bending. The picture on the left shows one sample after test together with one sample before testing.

Table 3 presents the results obtained in the 2009 campaign for the global group of the 27 samples (bending).

Table 3: 2009 results – bending (symbols as in EC 5)  
<table>
<thead>
<tr>
<th>Property</th>
<th>Values (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m,\text{mean}}$</td>
<td>45.5</td>
</tr>
<tr>
<td>$f_{m,k}$</td>
<td>20.5</td>
</tr>
<tr>
<td>$f_{m}$ (maximum and minimum values)</td>
<td>76.4/23.2</td>
</tr>
<tr>
<td>$E_0,\text{mean}$</td>
<td>11200</td>
</tr>
<tr>
<td>$E_0,k$</td>
<td>7800</td>
</tr>
<tr>
<td>$E_0$ (maximum and minimum values)</td>
<td>15400/7400</td>
</tr>
</tbody>
</table>

The results for the sub-group of the approved samples were considered as not representative, due to the small size of the group (14 samples).

Table 4 presents the results obtained in the 2009 campaign (compression).

Table 4: 2009 results – compression (symbols - EC 5)  
<table>
<thead>
<tr>
<th>Property</th>
<th>Values (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{c,0,\text{mean}}$</td>
<td>41.9</td>
</tr>
<tr>
<td>$f_{c,0,k}$</td>
<td>28.8</td>
</tr>
<tr>
<td>$f_{c}$ (maximum and minimum values)</td>
<td>52.2/26.5</td>
</tr>
</tbody>
</table>

4.3 SAMPLES, TEST SET-UPS AND RESULTS – 2010

The method, proposed by the author, was used to classify the samples resulting, 33 samples approved in 2010 out of the global set of 50, with 5*5*140 cm³.

In 2010, the samples came from ancient beams of unknown origin, existing in a contractor’s warehouse.
TIMBER ENGINEERING CHALLENGES AND SOLUTIONS

4.4 SAMPLES, TEST SET-UPS AND RESULTS – 2011

In 2011, it was decided to use new timber to validate the method and understand the differences between this situation and the grading of existing timber. Existing trees were chosen and harvested (see figure 8). To speed the process, the timber was sawn before drying and cut to the test dimensions. It was then dried and finally prepared to have a rectangular section (during the drying process some samples lost their linearity) and that is the reason with the dimensions are 4.8*8.5 cm² in section!

Figure 8 shows images of the tests after failure.

The method, proposed by the author, was in this moment used to classify in the laboratory the 51 samples with 4.8*8.5*190 cm³ resulting, 30 samples approved in 2011.

Figure 9 shows the test set-up and an image of a set of samples prepared for testing.

Figure 10 shows the Force-displacement diagram for the global set of tests.
Table 6 presents the results obtained in the 2011 campaign for the global group of the 51 samples and for the set of the 30 approved samples.

Table 6: 2011 results – bending (symbols as in EC 5)

<table>
<thead>
<tr>
<th>Property</th>
<th>Global set</th>
<th>Approved set</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m,\text{mean}}$</td>
<td>53.1</td>
<td>55.0</td>
</tr>
<tr>
<td>$c_V$</td>
<td>23.9%</td>
<td>23.4%</td>
</tr>
<tr>
<td>$f_{m,k}$</td>
<td>32.3</td>
<td>33.4</td>
</tr>
<tr>
<td>$f_{m,\text{max(min. values)}}$</td>
<td>86.6/17.4</td>
<td>86.6/26.4</td>
</tr>
<tr>
<td>$E_{0,\text{mean}}$</td>
<td>9900</td>
<td>10200</td>
</tr>
<tr>
<td>$c_V$</td>
<td>12.6%</td>
<td>11.7%</td>
</tr>
<tr>
<td>$E_{0,k}$</td>
<td>7900</td>
<td>8200</td>
</tr>
<tr>
<td>$E_{0,\text{max(min. values)}}$</td>
<td>12000/6700</td>
<td>12000/7300</td>
</tr>
</tbody>
</table>

Figure 11 shows images of the tests after failure.

5 MAIN RESULTS

After the conclusion of the 4 experimental campaigns and also of the implementation of the procedure in real rehabilitation works, several results can be presented.

5.1 INFLUENCE OF DEFECTS IN THE MECHANICAL BEHAVIOUR OF TIMBER

Concerning defects the main observations that arose from the experimental campaigns are:
- Knots and fissures are the more important defects that originated almost all the failures;
- The local defects in section (cuts) and defects of linearity did not influence in general the failures;
- The location of defects is the more relevant factor of failure; small defects can be of great importance if they are located in sections submitted to heavy loads;
- The bending moments and shear in each section have great influence in the effect of the defects; a simple grading is not sufficient to evaluate if a sample will fail or not.

5.2 VISUAL GRADING OF EXISTING STRUCTURAL TIMBER – GENERAL RECOMMENDATIONS

In what concerns general recommendations about the activity of grading ancient existing timber structural elements, the main conclusions that were obtained from the experimental campaigns are:
- Density is not relevant for grading and should not considered as a criteria to be analysed, because it doesn’t represent any relevant paper in the observed failures; hence it represents a very difficult property to be evaluated in situ, demanding the extraction of samples and the performance of destructive (although of little importance) action on existing elements;
- It is not possible to evaluate the complete influence of Knots, exclusively using its diameter, because it is very important the location and the groups of knots; the knots influence depends also on the possibility of the existence near to knots of inclined fibres and/or of fissures;
- The influence of fissures is very similar to knots and depends completely of its location in the timber element and also of the location within the timber element section;
- The inclination of fibres when having low grade and acting alone is not a very relevant defect but becomes very relevant when appearing together with knots and fissures and has influenced, in those situations, a lot of failure modes;
- During the inspection of existing timber structures, the evaluation of the integrity of the sections and of the position and dimension of knots is of great importance;
- The inspection of existing timber structures should be performed by specialists and is of great relevance when evaluating its stability for future uses.

5.3 MORE COMMON TYPES OF FAILURE

Concerning the failure modes the main conclusions that arose from the experimental campaigns are the following:
- The more common cause of failure is associated with the rupture of the fibres of wood in the traction zone of the section, due to circular bending in the central zone of the sample (not caused by defects or caused by defects);
- Some fragile failures occurred, in some cases without warning, in samples with a good apparent condition, but showed after inspection after failure relevant internal defects, impossible to identify exclusively using visual grading;
- Many samples tested in 2010, showed heavy defects caused by nails; those nails caused important fragilities on the elements; the very corrosive effect of steel over ancient timber is of very relevant importance and must be considered when using pre-existent timber elements, coming from demolitions;
- In many cases, it is not possible to determine before testing what will be the relevant defect for failure.

5.4 VIABILITY OF THE PROPOSED GRADING METHOD

Concerning the viability of the method proposed by the author it can be pointed the following:
- The experimental campaigns have not validated the viability of the method;
- The method should only be used as an indicative method although it is very useful to be used on inspections of existing structures;
- It is fundamental to discover all the timber elements in bad condition in order to eliminate and replace them with new solutions;
- A simple visual inspection is not sufficient to obtain certainty about the internal condition of the existing timber elements and the effect of old nails on the behaviour of the more affected sections of the elements;
- Any grading method (and also the proposed method) cannot be used as the single tool to evaluate the condition of existing structures, but should always be used as the main tool to be used in those conditions;
- The characteristic values of the properties to be used in calculations for new timber should not be used as reference for existing timber elements; the adoption of lower values is more convenient, due to the impossibility of evaluating thoroughly all the relevant existing problems;
- It is therefore convenient to use in the structural calculations lower values than those found on the corresponding grading class for new graded timber;
- The classification/inspection should be always performed by an experimented and skilled practitioner with relevant standards, regulations, technical and scientific knowledge about ancient timber structures;
- In what concerns the Young Modulus, results proved that the method is of good help to evaluate this parameter;
- The property that is more difficult to evaluate is the bending capacity because it is very much influenced by internal defects;
- As some fragile failures happened it is very important to guarantee that ALL the elements in bad condition are found.

6 CONCLUSIONS

The main conclusions are the following:
- The proposed grading method is not suitable to be used to grade existing timber elements;
- The method can be used to help *in situ* inspections; pathologies caused by steel nails, drying, insect and fungi attacks are very important in what concerns existing elements;
- Very conservative values should be used for the characteristic values of the mechanical properties to be used in stability calculations;
- The method is very easy to use and proved to be very convenient and efficient to grade new Portuguese oak wood (it should be clarified that actually there is not any national method available and that using other country’s standards seemed to be much less economical due to the rejection of the major part of the tested elements).

ACKNOWLEDGEMENT

The author wishes to thank the two Directors and all the researchers and employees of the two FEUP Laboratories where the tests were performed (LESE – Seysmic and Structural Engineering; LSC – Systems and Components) and the three students involved in the daily coordination of the experimental campaigns (Samuel Pereira, Albino Ramos and Joel Silva).

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Note: All references available on [http://paginas.fe.up.pt/~jfaria](http://paginas.fe.up.pt/~jfaria)
MECHANICAL CHARACTERIZATION OF OLD CHESTNUT CLEAR WOOD BY NON-DESTRUCTIVE AND DESTRUCTIVE TESTS

Beatrice Faggiano¹, Maria Rosaria Grippa¹

ABSTRACT: The paper presents an experimental study based on non-destructive (NDT) and destructive tests (DT) performed on defect-free specimens made of old chestnut wood (Castanea Sativa Mill.). The purpose is to provide reliable relationships between physical and mechanical properties and sclerometric and resistographic parameters for characterizing the mechanical behaviour of the clear material. The study has been developed within a wide research activity aimed at the in situ mechanical evaluation of chestnut timber properties by using combined NDT techniques.

KEYWORDS: Clear chestnut wood, Non destructive tests, Compression tests, Mechanical characterization by NDT

1 INTRODUCTION

In the field of timber mechanical characterization, the visual inspection represents the most simple and oldest non-destructive method, which allows to grade wood in end-use categories, through the identification of critical area, defects and decay (UNI EN 518, 1997; UNI 11035, 2003; UNI 11119, 2004). However the visual grading method is not entirely reliable due to the biased influence of the human factor and the qualitative aspect of acquired information.

More precisely material strength properties can be determined by direct testing of timber elements according to standardized procedures (UNI EN 519, 1997; UNI EN 408, 2004). However, destructive techniques are rarely acceptable in historical and ancient constructions, because they require to remove members from the structure or to extract from them small specimens for testing. Moreover, it is often not possible to provide samples with standard dimensions for laboratory destructive testing.

As an alternative, different instrumental non-destructive techniques (NDT) have been developed over the past decades, specially for detecting internal defects and decay of investigated elements. NDT methods present several advantages, such as their practical utilization, transport and efficiency; however, up to now, although they allow to gather a lot of useful information about the internal condition of members and their residual load-carrying capacity, these are only qualitative and do not lead to the determination of the mechanical properties, necessary to estimate the strength and stiffness capabilities of the structural systems.

With the purpose of the implementation of consistent methods for in situ mechanical characterization of timber by using combined NDT techniques, an experimental campaign, including non-destructive (ultrasonic, sclerometric, resistographic) and destructive tests (in compression and bending), was developed and is actually conducted on structural elements and defect-free specimens, made of old chestnut wood (Castanea Sativa Mill.) [1, 2]. The general aim of the research is to provide, by means of statistical analysis, reliable NDT–DT correlations, which can be used for estimation of wood density, strength and stiffness material properties. In addition, the study is also aimed at evaluating the influence of typical defect patterns on timber performances as respect to the clear material. Since the present weakness of non-destructive methods is specifically the lack of standardization both in the application of the techniques and in the interpretation of acquired data, the study, at the end of laboratory tests and results interpretation, would provide standardized guidelines to be used for practical applications.

The research activity was developed in the framework of the Italian project PRIN 2006 “Diagnosis techniques and totally removable low invasive strengthening methods for the structural rehabilitation and the seismic improvement of historical timber structures”, Prof. M. Piazza coordinator, Dr. B. Faggiano scientific responsible of the research unit UNINA (University of Naples “Federico II”) [1, 2]. The experimental investigations are actually in progress within the DPC-RELUIS 2010 – 2013 Italian project, Line 1, Task 1.
2 MAIN NDTs STATE OF ART

Ultrasonic stress wave is one of the popular NDT global method used for wood, based on the propagation of sound waves through the investigated element. Generally, it is possible to correlate the efficiency of wave propagation with the physical and mechanical properties of wood; high propagation velocities are associated with greater fracture resistance and absence of material defects. For practical purpose, the relation between the dynamic modulus of elasticity and the static value is particularly relevant, it being explained by the viscous-elastic behaviour of wood [3]. Therefore, the propagation velocity of elastic waves, together with the material density, immediately show information on the stiffness coefficients of the material. Some authors tried to determine residual strength of structural elements that were used in ancient constructions or that were attacked by biological agents [4], or tried to determine qualitative properties by modelling wood as homogeneous isotropic material, assuming that clear and defected wood can be modelled as a fluid, neglecting bending stiffness [5].

The wood test hammer system, so-called Pilodyn method, by means of Pilodyn 4JR or similar equipment, is an alternative for fast and non-destructive estimation of wood density. Empirical correlations between the pin penetration depth and wood density are proposed by several authors [6, 7, 8], with regression coefficients ranging from 0.74 to 0.92. Studies were also carried out to define correlations with mechanical properties, such as strength and modulus of elasticity [6, 9].

The resistographic method is nowadays among the most used one to determine density properties of wood products, to find wood decay, rot and cracks aiming at evaluating the residual resistant section, to analyze annual ring structures, to measure real dimensions of transversal section of structural elements [6, 10, 11]. Many studies reveal that this method may be used at best in conjunction with the other ND methods and techniques for providing qualitative or more global condition assessment.

3 THE SPECIMENS

All tested specimens were extracted by cutting eleven base structural elements, made of chestnut wood (Castanea sativa, Mill.), provided from timber roofing trusses of an ancient masonry building in Naples, dated the beginning of the 19th century [1, 2]. Twenty samples (type C_L; Fig. 1a) were obtained, about two from each structural element, for non-destructive investigations and compression tests parallel to grain, they having standard dimensions, 5×5×30 cm sized, according to UNI EN 408 (2004). The specimens presented non-extensive areas affected by superficial defects, such as ring shakes, cracks and slope of grain.

Figure 1: Specimens features: a) Type C_L; b) Type DF-C

In addition compression tests were performed on defect-free (DF) specimens, 2×2×4 cm³ sized, the longitudinal axis being along the grain orientation according to UNI ISO Italian codes. In particular, in order to analyze the behaviour in compression of the base material in both parallel and perpendicular to grain directions, three groups of clear specimens were obtained: 1) thirty-tree specimens (type DF-C_L), three extracted from each structural element, for longitudinal tests, according to UNI ISO 3787 (1985); 2), 3) twenty-two specimens each, for radial (type DF-C_rad) and tangential (type DF-C_rad) loading applications respectively, according to UNI ISO 3132 (1985) (Fig. 1b). The moisture content of the specimens was estimated equal to about 10-12% at a temperature of 20±2 °C. The wood density was defined weighting and measuring the dimensions of the samples.

Figure 2: Density average values.
In Figure 2 density values are reported for both groups \( C_L \) and \( DF \), for samples extracted by each of the eleven base structural elements. The strong agreement between the two groups is apparent.

## 4 NON-DESTRUCTIVE TESTS

Sclerometric and resistographic tests were carried out on specimens type \( C_L \) \[12\]. The Wood Pecker mechanical test hammer for wood was used for sclerometric tests, which allow to measure the penetration of a blunt metallic needle, 2.5mm diameter and 50mm length, shoot into the outer layers of wood by means of five spring blows. The shots were performed by using the device with penetration orthogonal to the test surfaces in both longitudinal (L) and transversal (T) directions (Fig. 3), in particular, on the lateral faces of the samples, in both radial (rad) and tangential (tg) directions.

**Figure 3:** ND test set-up: sclerometric shots

IMLRESI F400 device was used for resistographic measurements. It is a drilling resistance measuring system based on the energy used to introduce a needle, 1.5mm to 3.0mm diameter, through the wood with a regular advancing speed. The spent energy is measured electronically every 0.1mm. In particular, an advancing speed of about 20cm/minute was used during the tests. Furthermore, on end-sections of the samples, parallel to grain perforations about 5cm long were realized (Fig. 4).

**Figure 4:** ND test set-up: resistographic measures

Test set-up are given in Table 1, where, for both sclerometric and resistographic methods, the number of investigations per specimen is reported. A typical resistographic profile is depicted in Figure 5 for a perforation in transversal/tangential direction. As unique measure of drilling resistance, the mean value of amplitude \( A_m \) has been calculated for each profile at only homogeneous zones, that is disregarding low or null resistance measures.

**Table 1:** Non-destructive methods: tests number

<table>
<thead>
<tr>
<th>Test types</th>
<th>Direction</th>
<th>Tests number per direction</th>
<th>Tests number per specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sclerometric</td>
<td>L</td>
<td>1 per end section</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>T</td>
<td>3 rad; 3 tg</td>
<td>6</td>
</tr>
<tr>
<td>Resistographic</td>
<td>L</td>
<td>2 per end section</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>T</td>
<td>2 rad; 2 tg</td>
<td>4</td>
</tr>
</tbody>
</table>

**Figure 5:** Typical transversal resistographic profile

As NDT parameters, the average values of penetration depth (PD) and mean amplitude (\( A_m \)) are considered for each group of specimens extracted by the same structural element.

**Figure 6:** NDT parameters: a) Sclerometric PD [mm]; b) Resistographic \( A_m \) [%]
The obtained NDT results are represented in Figure 6 for both longitudinal (L) and transversal (T) tests, the latter ones calculated as average of radial (rad) and tangential (tg) values.

Concerning the sclerometric results, it is possible to observe that the longitudinal measures are generally larger than the transversal ones, the average ratio $PD_L/PD_T$ being equal to 1.25, whereas a similar response has been obtained through radial and tangential shots. Concerning resistigraphic measurements, independently from the tests direction, all specimens groups exhibit a similar response, the average ratio $Am_{L}/Am_{T}$ being equal to 1.06.

5 COMPRESSION TESTS

Compression tests parallel to grain were carried out on both specimens type $CL$ (n. 20) and $DF-CL$ (n. 33), according to UNI EN 408 and UNI ISO 3787, respectively (Fig. 7a, b). Radial and tangential compression tests were performed on specimens type $DF-C_{rad}$ (n. 22) and $DF-C_{tg}$ (n. 22) respectively, taking into account the orientation of the annual rings with respect to the direction of the applied load (Fig. 7c, d), according to UNI ISO 3132. Tests were performed under force control, using the Mohr and Federhaff AG machine of 400kN capacity [13].

![Figure 7: Compression tests](image)

The experimental results are represented in Figure 8 in terms of compression strength ($fc$) and modulus of elasticity ($Ec$), as average values obtained by each group of samples extracted by the same structural element. It is worth to notice that the two specimens groups $CL$ and $DF-CL$ exhibit different responses in terms of strength, whereas the stiffness properties are nearly similar each other. In particular, structural elements type $CL$ are characterized by a compression strength about twice smaller than the clear material, due to the presence of natural defects, as fissures and ring shakes.

Concerning the tests on defect-free samples only, similar both stiffness and strength properties in radial and tangential directions are provided. Moreover, the average ratios $f_{c,0}/f_{c,90}$ and $E_{c,0}/E_{c,90}$ are equal to about 12 and 14, respectively, being the transverse behaviour significantly less performing as respect to the longitudinal one.

![Figure 8: DT parameters: a) Compression strength [N/mm²]; b) Modulus of elasticity [N/mm²]](image)

6 RELATIONS BETWEEN DT PROPERTIES

The correlations between physical, strength and stiffness properties are statistically analyzed. The obtained determination ($R^2$) coefficient matrix is given in Table 2.

<table>
<thead>
<tr>
<th></th>
<th>$\rho$</th>
<th>$f_{c,0}$</th>
<th>$E_{c,0}$</th>
<th>$f_{c,90,rad}$</th>
<th>$E_{c,90,rad}$</th>
<th>$f_{c,90,tg}$</th>
<th>$E_{c,90,tg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>1</td>
<td>0.876</td>
<td>0.628</td>
<td>0.302</td>
<td>0.464</td>
<td>0.577</td>
<td>0.416</td>
</tr>
<tr>
<td>$f_{c,0}$</td>
<td>0.876</td>
<td>1</td>
<td>0.769</td>
<td>0.124</td>
<td>0.284</td>
<td>0.353</td>
<td>0.231</td>
</tr>
<tr>
<td>$E_{c,0}$</td>
<td>0.628</td>
<td>0.769</td>
<td>1</td>
<td>0.145</td>
<td>0.241</td>
<td>0.156</td>
<td>0.120</td>
</tr>
<tr>
<td>$f_{c,90,rad}$</td>
<td>0.302</td>
<td>0.124</td>
<td>0.145</td>
<td>1</td>
<td>0.463</td>
<td>0.584</td>
<td>0.561</td>
</tr>
<tr>
<td>$E_{c,90,rad}$</td>
<td>0.464</td>
<td>0.284</td>
<td>0.241</td>
<td>0.463</td>
<td>1</td>
<td>0.570</td>
<td>0.773</td>
</tr>
<tr>
<td>$f_{c,90,tg}$</td>
<td>0.577</td>
<td>0.353</td>
<td>0.156</td>
<td>0.584</td>
<td>0.570</td>
<td>1</td>
<td>0.680</td>
</tr>
<tr>
<td>$E_{c,90,tg}$</td>
<td>0.416</td>
<td>0.231</td>
<td>0.120</td>
<td>0.561</td>
<td>0.773</td>
<td>0.680</td>
<td>1</td>
</tr>
</tbody>
</table>

Results emphasize good correlations between modulus of elasticity ($E_c$) and strength ($f_c$), especially for compression tests in longitudinal direction (Fig. 9).

![Figure 9: Compression strength ($f_{c,0}$) vs Modulus of elasticity ($E_{c,0}$)](image)
In particular, density appears one of the main factors that influence the material behaviour, being strongly related with both $f_{c,0}$ and $E_{c,0}$ properties (Fig. 10). With reference to the relationships in transversal compression, medium and good $R^2$-values are provided by $E_{c,90}$ vs $f_{c,90}$ linear correlations, respectively for radial and tangential tests (Fig. 11).

$\rho = 0.1135 \rho - 2.95$
$R^2 = 0.876$

$E_{c,0} = 11.975 \rho - 71.98$
$R^2 = 0.623$

$E_{c,0} = 0.0052 E_{c,90,rad} + 2.521$
$R^2 = 0.463$

$E_{c,0} = 0.0052 E_{c,90,tg} + 2.521$
$R^2 = 0.463$

$\rho = 0.1135 \rho - 2.95$
$R^2 = 0.876$

$E_{c,0} = 11.975 \rho - 71.98$
$R^2 = 0.623$

$E_{c,0} = 0.0052 E_{c,90,rad} + 2.521$
$R^2 = 0.463$

$E_{c,0} = 0.0052 E_{c,90,tg} + 2.521$
$R^2 = 0.463$

Table 3: Density vs NDT parameters: $R^2$-values

<table>
<thead>
<tr>
<th>Sclerometric parameters [mm]</th>
<th>PD rad</th>
<th>PD tg</th>
<th>PD T</th>
<th>PD L</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$-values</td>
<td>0.692</td>
<td>0.606</td>
<td>0.716</td>
<td>0.327</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistographic parameters [%]</th>
<th>Am rad</th>
<th>Am tg</th>
<th>Am T</th>
<th>Am L</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$-values</td>
<td>0.488</td>
<td>0.419</td>
<td>0.463</td>
<td>0.380</td>
</tr>
</tbody>
</table>

Figure 12: Density ($\rho$) vs Penetration Depth (PD T)

For the prediction of compression strength parallel to grain, no relevant difference in terms of $R^2$-values can be pointed between sclerometric and resistographic methods, in both longitudinal and transverse directions (Tables 4, 5). Medium correlations are found, as the scatter plots in Figure 13 show.

Furthermore, the statistical analysis has evidenced very low $R^2$-values for the correlations between experimental NDT variables and strength values in transverse direction (Tables 4, 5). In any case, the best result has been obtained by adopting the transversal sclerometric parameters.

Figure 13: NDT-DT correlations:

a) Compression strength ($f_{c,0}$) vs Penetration Depth (PD);

b) Compression strength ($f_{c,0}$) vs mean Amplitude ($A_m$)

7 ESTIMATION OF DENSITY AND STRENGTH BY NDT

Aiming at evaluating the efficiency of NDT techniques as useful tool for the mechanical characterization of clear wood, the correlations of sclerometric and resistographic parameters with density and strength are examined [14]. The corresponding $R^2$-values are provided in Tables 3. Strong agreement is found between wood density and transversal pin penetration depth (PD T; Fig. 12), while the relation between density and drilling amplitude ($A_m$) presents generally larger scatters.
Table 4: Compression strength ($f_c$) [N/mm²] vs Penetration Depth (PD) [mm]: $R^2$-values

<table>
<thead>
<tr>
<th>PD</th>
<th>$f_{c,0}$</th>
<th>$f_{c,0,rad}$</th>
<th>$f_{c,0,tg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>rad</td>
<td>0.606</td>
<td>0.107</td>
<td>0.435</td>
</tr>
<tr>
<td>tg</td>
<td>0.407</td>
<td>-</td>
<td>0.315</td>
</tr>
<tr>
<td>T</td>
<td>0.547</td>
<td>-</td>
<td>0.408</td>
</tr>
<tr>
<td>L</td>
<td>0.345</td>
<td>-</td>
<td>0.103</td>
</tr>
</tbody>
</table>

Table 5: Amplitude (Am) [%]: $R^2$-values

<table>
<thead>
<tr>
<th>Am</th>
<th>$f_{c,0}$</th>
<th>$f_{c,0,rad}$</th>
<th>$f_{c,0,tg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>rad</td>
<td>0.388</td>
<td>-</td>
<td>0.115</td>
</tr>
<tr>
<td>tg</td>
<td>0.404</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>T</td>
<td>0.409</td>
<td>-</td>
<td>0.100</td>
</tr>
<tr>
<td>L</td>
<td>0.553</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

8 CONCLUSIONS

By means of destructive tests in compression of defect-free specimens, the following results have been obtained:
- Mean strength parallel to grain ($f_{c,0}$) equal to 60 N/mm², about 12 times greater than the same one obtained in perpendicular direction ($f_{c,90}$);
- Average values of strength in radial and tangential directions ($f_{c,0}$) very similar each other, equal to about 5.5 N/mm²;
- Average ratio $E_{c,0}/E_{c,90}$ equal to 14, with a mean longitudinal modulus of elasticity of about 6500 N/mm².

The statistical analysis of results by means of linear regressions models have emphasized a robust reliability of sclerometric method in transversal direction for prediction of wood density. The latter one represents one of the main factor that influence the material performance, it being strongly related with both $f_{c,0}$ ($R^2 = 0.88$) and $E_{c,0}$ ($R^2 = 0.63$) properties. Finally, good correlations has been also found between $f_{c,0}$ and $E_{c,90}$ ($R^2 = 0.77$), whereas, with reference to the relationships in transverse direction, medium and good $R^2$-values are provided by $E_{c,90}$ vs $f_{c,90}$ linear correlations, for radial and tangential directions, respectively.

The achieved correlations can be considered novel tools for in situ characterization of old chestnut clear wood. These could be used to predict mechanical properties of timber members in actual sizes, having compared the behaviours in compression of clear wood and structural elements, the latter being characterized by defects and irregularities.

In these directions, results calibrations are required by extending the laboratory investigations and statistical elaborations on both further samples and wood species.

ACKNOWLEDGEMENTS

The research activity has been developed within the PRIN 2006 and it is actually in progress in the framework of DPC-RELUIIS 2010 – 2013, Line 1, Task 1, both Italian projects.

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WOOD EFFECTIVE THICKNESS IN BRITTLE AND MIXED FAILURE MODES OF TIMBER RIVET CONNECTIONS

Pouyan Zarnani¹ and Pierre Quenneville²

ABSTRACT: Rivets are tight-fit fasteners to make high strength timber connections. There are three mechanisms of failure for riveted connections; the brittle tear-out of a plug of wood defined by the rivet’s perimeter, the ductile yielding of rivets with wood embedding and the mixed failure mode which has a brittle manner where the wood fails with some deflection of rivets before rivets reach complete yielding. Reliable estimation of the effective wood thickness is required to predict precisely the wood load-carrying capacity which is to be compared to the rivet capacity to derive the ultimate connection strength and its failure mode. In the proposed method, the effective wood thickness of the brittle failure mode is modelled based on the elastic deformation of the rivet as a beam on an elasto-plastic foundation. For the mixed failure mode, the effective wood thickness is derived based on the EYM yield model predictions. Thickness of the failed block from current tests on Radiata Pine LVL and glulam in brittle/mixed failure modes confirm the validity of the proposed method and show that this predictive method can be used for timber riveted connections for more precise prediction of the connection capacity.

KEYWORDS: Rivet connection, brittle/mixed failure mode, effective wood thickness

1 INTRODUCTION

The timber rivet is one of the best fasteners due to its high strength capacity and cost-efficiency. Timber rivet connections have been used successfully in Canada and U.S. in different types of heavy structures over the last 3 decades.

There are three mechanisms of failure for riveted connections; the brittle tear-out of a plug of wood defined by the rivet’s perimeter, the ductile yielding of rivets with localized wood crushing and the mixed failure mode which is a brittle failure of the wood with some deflection of the rivets before the rivets reach complete yielding [1].

Rivets are part of the CSA-O86 [2] and US-NDS [3] structural wood design standards, however the current standards do not take into account the wood effective thickness in brittle/mixed failure modes for calculating the strength of wood in riveted connections. The brittle/mixed failure modes should be avoided since it induces the brittle downfall of the whole structure. A precise estimation of the effective wood thickness provides better predictions of the wood load-carrying capacity which is to be compared to the rivet capacity to derive the ultimate connection strength [4].

2 PREDICTIVE MODEL FOR WOOD EFFECTIVE THICKNESS

2.1 BRITTLE MODE

In the proposed model, the effective wood thickness (Eq. 1) is determined from the elastic deformation of the rivet as a beam on an elasto-plastic foundation (Fig. 1). The rivet is supported by springs with bilinear response that simulate the local nonlinear embedding behaviour of the timber surrounding it.

The stiffness of the springs is based on the ideal elasto-plastic response of the wood embedment behaviour parallel-to-grain obtained form tests (Fig. 2 and 3).

Figure 1: Spring model of elastic deformation of rivet as a beam on an elasto-plastic foundation

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A similar model was used by Johnsson and Stehn [5] for the prediction of the wood effective thickness in nail connections brittle failure. Their model was based on the beam on elastic foundation theory. However, the current model considers also the plastic deformation domain of the wood surrounding the rivet (especially near the head) during the elastic deflection of the rivet in connection brittle failures. This plastic response of the wood embedment behaviour explains the non-linear load-slip curves from brittle failure tests observed approximately beyond 0.5 mm displacement (Fig. 4).

In the current model, the modulus of elasticity of the rivet is considered $E_s=210$ GPa and the poisson’s ratio $\nu=0.3$. The rivet cross-section dimensions used in the model is based on a rectangular shape, 6.4 mm by 3.2 mm with rounded corners having 1 mm radius [6].

$$t_{df,\ell} \sim \begin{cases} 0.95L_p & \text{for } L_p \text{ equals to } 28.5 \text{ mm} \\ 0.85L_p & \text{for } L_p \text{ equals to } 78.5 \text{ mm} \\ 0.75L_p & \text{for } L_p \text{ equals to } 78.5 \text{ mm} \end{cases}$$

### 2.2 MIXED MODE

The embedded length of the rivet under the yielding failure modes was considered as the effective wood thickness ($t_{ef}$) for mixed failure modes which can be seen in Figure 5 and can be derived by Equation 2 based on Johansen’s yield theory [7] which is the foundation for the EYM formulas in Eurocode 5 [8]. In this paper, the rivet yielding mode is based on the number of plastic hinge formation in the fastener observed in the tests which also can be predicted by a consistent yield model proposed by Zarnani and Quenneville [9].

$$t_{ef,y} = \begin{cases} L_p & \text{Mode I_m} \\ \frac{M_{y,l}}{f_{h,0}d_l^2} & \text{Mode III_m} \\ \frac{2M_{y,l}}{f_{h,0}d_l^2} & \text{Mode IV} \end{cases}$$

where $d_l$ is the rivet cross-section dimension bearing on the wood parallel-to-grain (equal to 3.2 mm), $f_{h,0}$ is the embedment strength of the wood which can be determined as a function of $d_l$ and the density of the wood [9] and $M_{y,l}$ is parallel-to-grain moment capacity of the rivet (equal to 30000 Nmm) [6].
3 RESULTS AND DISCUSSION

In current tension tests (Fig. 6) on LVL and glulam, the thickness of the failed block, \( t_{\text{block}} \), in the majority of the brittle failures was observed between 0.8\( L_p \) to 0.95\( L_p \). This corresponds to the elastic deformation of the rivets since there were no observed plastic deflections (Fig. 7a). However, in some connection groups, considerable decrease of \( t_{\text{block}} \) with a slight distortion of the rivets was visible (Fig. 7b). This effect can be associated to the larger displacement in the load-slip plots (Fig. 4). In these groups, \( t_{\text{block}} \) corresponded to the effective wood thickness, \( t_{\text{ef}} \), depending on the governing failure mode of the rivets.

This failure mode is called mixed mode since the wood fails with some deflection of the rivets before they reach complete yielding. The test configurations for the brittle failures include 2 rivet lengths of 40 and 65 mm on both LVL and glulam and for the mixed modes cover all 3 rivet sizes on LVL and only the rivet length of 90 mm for glulam. The predicted and average observed values for the effective wood thickness under brittle/mixed failure modes on LVL and glulam were approximately the same.

**Figure 6:** Typical specimen in testing apparatus

**Figure 7:** Thickness of the failed block; (a) Brittle failure mode, (b) Mixed failure mode
A comparison is shown in Table 1 between the wood effective thickness of the brittle/mixed failure modes for the experimental results and the prediction model (Fig. 8). One can note that there is good agreement between the predictions and the failed block thicknesses from the tests.

Table 1: Experimental results on wood effective thickness compared to predictions from proposed model

<table>
<thead>
<tr>
<th>Penetration length / rivet length (mm)</th>
<th>( t_{ef}^* ) brittle failure mode (mm)</th>
<th>( t_{ef}^{**} ) mixed failure mode (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigid rivet head (predicted/observed)</td>
<td>No head rigidity* (predicted/observed)</td>
<td>Rigid rivet head (predicted/observed)</td>
</tr>
<tr>
<td>28.5/40</td>
<td>0.95Lp/0.90Lp</td>
<td>0.7Lp</td>
</tr>
<tr>
<td>53.5/65</td>
<td>0.85Lp/0.84Lp</td>
<td>0.65Lp</td>
</tr>
<tr>
<td>78.5/90</td>
<td>0.75Lp/0.5Lp</td>
<td>0.6Lp</td>
</tr>
</tbody>
</table>

* Predicted values to show the effect of rivet head rotational fixity.

4 CONCLUSIONS

The proposed method provides reliable estimation of the effective wood thickness in brittle/mixed failure modes. Precise prediction of the effective thickness is required for the wood load-carrying capacity on account of the fact that it shall be compared by the rivet capacity to derive the ultimate connection strength and its failure mode.

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MONOTONIC TEST OF SHEAR WALL PANEL MADE FROM MANGIUM WOOD

Sulistyono¹, Naresworo Nugroho², Surjono Surjokusumo³, Osly Rahman²

ABSTRACT: Indonesia is located on one of the tectonic most active zone of the world. Many people lose their live because of collapsing buildings. To reduce the risk of being buried under a collapsed house earthquake safe houses especially for low income families - are urgently required. Therefore, an earthquake resistant house is designed to overcome problems in structural building security due to earthquake disaster. This research aimed to find out the performance of shearwall panel made from mangium wood on the prefabricated seismic resistance house. There are 5 design patterns size (6.8 x 120 x 240) cm of shear wall, which include: shearwall with straight sheathing (type A), shearwall with 45° diagonal sheathing, (type B), shearwall with diagonal windowed sheathing (type C) and shearwall doored sheating (type D and E). Those shearwall were tested with racking test (ISO/DIS 22452-2009) using a monotonic lateral load and calculation of seismic forces by equivalent static seismic analysis (SNI 1726-2002). The result showed that the diagonal sheathing type is stronger and more rigid as it has a triangulation truss. Type A design is appropriate for a small seismic zone, furthermore type B, C, D, E are suitable for a medium and big seismic zone.

KEYWORDS: mangium wood, monotonic test, shear wall

1 INTRODUCTION

Indonesia is located on one of the tectonic most active zone of the world. There are six zones of earthquake accordance to SNI03-1726-2002 [1]. Zone 6 is the worst dominated by north Papua area, following zone 2, 3, 4, and 5 in west Sumatra and Java region, and the safety zone of earthquake laid in zone 1 (Kalimantan). There were more than 30 earthquakes with magnitude > 5 SR, with the worst in Tasikmalaya (7.3 SR) and Padang (7.6 SR) in the last September 2009. Before that, Indonesia had bad history with the earthquake in Aceh - North Sumatra and south Java area in last five years ago.

There are many designs of earthquake resistance house including the traditional house. Division of Wood Engineering and Building Design, Faculty of Forestry, Bogor Agricultural University have developed the house with knock down and pre-fabrication system using Akasia wood (Acacia mangium). This species is used because of its wood relatively new known as wood for construction, in other that the wood comes from fast growing species of plantation forest. The mechanical properties of this wood are quite good for the light – medium construction [2, 3]. This research aimed to find out the performance of shear-wall panel made from Mangium wood on the prefabricated seismic resistance house

2 MATERIALS AND METHOD

Acacia mangium woods used were from national industrial plantation forest, PT Inhutani II, in Pulau Laut, South Kalimantan. The materials were plank in air-dried condition with moisture content of 14-16%. The plank, than were constructed in various type of shear wall. There are 5 design patterns size (6.8 x 120 x 240) cm of shear wall, which include: shearwall with straight sheating (type A), shearwall with 45° diagonal sheating, (type B), shearwall with diagonal windowed sheathing (type C) and shearwall doored sheating (type D and E) as shown if Fig. 1. Those shearwall were tested with racking test (ISO/DIS 22452-2009) [4] using a monotonic lateral load (Fig. 2) and calculation of seismic forces by equivalent static seismic analysis (SNI 1726-2002)
### Table 1: Shearwall Design with Various Sheathing Type and Opening Size

<table>
<thead>
<tr>
<th>Shearwall Design</th>
<th>Sheathing Type and Opening Size</th>
</tr>
</thead>
</table>
|                  | Type = A  
|                  | Area ratio \( r \) = 1.0  
|                  | Opening size:  
|                  | door = -  
|                  | window = - |
|                  | Type = B  
|                  | Area ratio \( r \) = 1.0  
|                  | Opening size:  
|                  | door = -  
|                  | window = - |
|                  | Type = C  
|                  | Area ratio \( r \) = 0.79  
|                  | Opening size:  
|                  | door = -  
|                  | window = 100x120 cm |
|                  | Type = D  
|                  | Area ratio \( r \) = 0.58  
|                  | Opening size:  
|                  | door = 200x120 cm  
|                  | window = - |
|                  | Type = E  
|                  | Area Ratio \( r \) = 0.37  
|                  | Opening size:  
|                  | door = 200x120 cm  
|                  | window = 100x120 cm |

**Figure 1: Shear Wall Design with Various Sheating type and Opening Size**

### 3 RESULTS AND DISCUSSION

The result showed that diagonal sheathing (Type B), is more stronger than horizontal sheathing (Type A) and also more rigid as it has a triangulation like truss. Type A and C design are the good performance for racking strength and racking stiffness as shown in Fig 3, although type C has a window. The opening size can be decreasing the performance of shear wall, but not depending in area ratio of shear wall.

**Figure 3: Shear-stress Diagram of Various Type of Mangium Panel Wall.**

### 4 CONCLUSIONS

Type A design is appropriate for a small seismic zone, furthermore type B, C, D, E are suitable for a medium and big seismic zone

### REFERENCES


NON-LINEAR MODELLING OF WOODEN LIGHT-FRAME AND X-LAM STRUCTURES

Massimo Fragiacomo¹, Claudio Amadio², Giovanni Rinaldin³, Ljuba Sancin⁴

ABSTRACT: Cross-laminated (‘X-lam’) timber panels and light-frame structures represent a new effective construction technology, with significant potential for multi-storey earthquake-resistant buildings. Despite the extensive use of these systems, there is a lack of design methods to evaluate the capacity in terms of dissipated energy and global response under cyclic loads. In the paper a method that allows an accurate evaluation of the energy dissipated by a subassembly made of X-lam panels connected to each other and to the foundation with metal fasteners is proposed. The X-lam panel is regarded as linear-elastic and all dissipated energy is assumed to take place in the connections. Appropriate user elements are implemented in the ABAQUS software package as external subroutines for different metal connections, such as screws, angle brackets, hold-downs and others. These elements allow the user to implement a hysteretic behaviour with pinching very close to the experimental results with allowance for strength and stiffness degradation. The model was calibrated and validated on cyclic tests carried out at CNR-Ivalsa on single X-lam walls connected to the foundation. The model was then extended to light-frame structures. These numerical models were able to predict the correct cyclic response in terms of deflection and energy with an overall good approximation.

KEYWORDS: cyclic modelling, wooden structures, X-lam, light-frame, non-linear analysis

1 INTRODUCTION

This paper presents a new component model for dynamic (seismic) analysis of cross-laminated (X-lam) and light-frame buildings. X-lam panels are schematized with linear-elastic shell elements with a composite section of orthotropic layers, while the steel connections are modelled using non-linear springs with hysteretic behaviour. The model is based on the evidence that energy dissipation of X-lam wooden buildings subjected to earthquake excitation mainly occurs in the connections between panels and with the foundations.

Light-frame walls are schematized with 2 diagonal non-linear hysteretic springs; the remaining parts of the buildings are considered as elastic. The advantage of the proposed model is the possibility to calibrate each spring on the basis of cyclic tests carried out on different types of connection, with the possibility to predict the cyclic behaviour of entire panels, subassemblies, and buildings. A phenomenological way to represent the non-linear hysteretic behaviour has been chosen, since it allows the user to fully control and model the metal connections in a simple way. In particular, the model includes allowance for stiffness and strength degradation, which are features of great importance in the cyclic behaviour of timber structures. All the phenomenological relationships discussed in the paper were based on the results of force vs. displacement cyclic experimental data currently available.

2 COMPONENTS APPROACH

In this paper, a components approach has been used to model X-lam and light-frame timber structures, with the aim to predict the cyclic (seismic) behaviour with good accuracy. In this approach, each connector (angle bracket, hold-down, screw) or the entire light-frame wall (excluding the connections to the foundation and to the adjacent and upper panels) is schematized with a non-linear spring characterized by a hysteretic behaviour. The model has been implemented in a widespread

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³ Giovanni Rinaldin, PhD candidate, Department of Civil Engineering and Architecture, University of Trieste, Piazzale Europa, 1, 34127 Trieste, Italy. E-mail: giovanni.rinaldin@phd.units.it
⁴ Ljuba Sancin, Former graduate student, Department of Civil Engineering and Architecture University of Trieste, Piazzale Europa, 1, 34127 Trieste, Italy. E-mail: ljuba.sancin@gmail.com
software package such as Abaqus using an external user subroutine written in Fortran.

2.1 CONSTITUTIVE MODEL OF THE COMPONENTS

Each metal connection (component) has been modelled as a non-linear spring with hysteretic behaviour. The actual curves have been approximated with piecewise linear laws, more specifically tri-linear curves, which have been parameterized to allow the user to fully control their shape. Three different types of curve have been developed: for angle brackets, for screws and for hold-downs. Each curve is made of several branches composing the backbone curve and the hysteretic cycle. The non-linear spring connects two coincident points in the undeformed state, hence it has zero length. In the most general case, every spring returns to the solver the three forces that develop in its plane and the corresponding three stiffnesses (see Table 1). Although only planar springs with three degrees of freedom have been considered in this study for the sake of simplicity as this is the most important case, the theory can be easily generalized to the case of a spatial spring with six degrees of freedom.

Table 1: Basic definition of non-linear springs

<table>
<thead>
<tr>
<th>Congruence equations – Spring degrees of freedom:</th>
<th>Nodal equilibrium equations:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi = \varphi_2 - \varphi_1$</td>
<td>$T_1 = T$</td>
</tr>
<tr>
<td>$\gamma = u_2 - u_1$</td>
<td>$T_2 = -T$</td>
</tr>
<tr>
<td>$\varepsilon = v_2 - v_1$</td>
<td>$M_1 = M$</td>
</tr>
</tbody>
</table>

2.2 HYSTERETIC LAWS

Figure 1 displays the shear force vs. shear displacement (slip) piecewise linear law used to model the behaviour of screw and angle bracket connections. The model is defined by 9 independent input parameters and uses 32 state variables per cycle. They are:
1. Elastic stiffness;
2. Yielding force;
3. First inelastic stiffness (hardening branch);
4. Peak strength;
5. Second inelastic stiffness (softening or hardening branch);
6. $K_m$ factor: it sets the unloading stiffness of branches #4 and #50, which is obtained by multiplying the elastic stiffness by this factor $K_m$;
7. $RC$ parameter: it sets the lower limit of branches #5 and #40 by multiplying the force value $F$ attained before entering the unloading path by this parameter $RC$;
8. $SC$ parameter: it sets the lower limit of branches #4 and #50 by multiplying the force value attained before entering the unloading path $F$ by this parameter $SC$;
9. Ultimate displacement, $D_u$: when this value is attained, a brittle failure occurs.

Figure 2 displays the axial force vs. axial displacement piecewise linear relationship used to model the behaviour of the hold-down connector. Same parameters as before were used.

Four additional parameters are used to calibrate the strength and stiffness degradation, which are explained in the following Section.

Figure 1: Piecewise linear relationship of screws and angle bracket springs

Figure 2: Piecewise linear relationship of hold-down springs
2.3 STIFFNESS AND STRENGTH DEGRADATION

Stiffness and strength degradations have been implemented in the model as they are both important features of timber connections. A degradation of stiffness proportional to the maximum displacement attained during the load history has been assumed for the last unloading branches #5 and #50 (after the pinching effect) for both spring models. This effect has been taken into account with relationship (1).

\[ k_{\text{deg}} = k_{\text{el}} \left( 1 - \frac{D_{\text{max}}}{D_{\text{ult}}} \left( 1 - d_{yj} \right) \right) \]  

where:

- \( k_{\text{deg}} \) = degraded stiffness;
- \( k_{\text{el}} \) = elastic stiffness;
- \( D_{\text{max}} \) = maximum displacement attained during the load history;
- \( D_{\text{ult}} \) = ultimate displacement;
- \( d_{yj} \) = stiffness degradation parameter.

The strength degradation depends on the energy dissipated and on the maximum displacement attained during the load history. Due to the complexity of evaluating the dependence of the strength degradation on both these quantities, three calibration parameters have been introduced: a linear and two exponential ones. The adopted relationship is reported in (2).

\[ \Delta d = \gamma \cdot d_{yj} \left( \frac{E_{\text{dis}}}{E_{\text{dis}(A)}} \right)^{\alpha} \left( \frac{D_{\text{max}}}{D_{\text{ult}}} \right)^{\beta} \]  

where:

- \( \Delta d \) = additional displacement at reloading;
- \( \gamma \) = linear parameter;
- \( d_{yj} \) = displacement at yielding force;
- \( \alpha \) = exponential degradation parameter;
- \( E_{\text{dis}} \) = dissipated energy;
- \( E_{\text{dis}(A)} \) = dissipated energy at the beginning of unloading path;
- \( D_{\text{max}} \) = maximum displacement attained during the loading history;
- \( D_{\text{ult}} \) = ultimate displacement;
- \( \beta \) = exponential degradation parameter.

2.4 CALIBRATION

Each type of spring was calibrated on the experimental results reported in [1] for X-lam connections and in [2] for light-frame wall. The calibration was done by following the steps listed herein after:

1. the yielding and peak force were extracted from the experimental results;
2. the elastic stiffness was estimated once a good fit of the yielding displacement was obtained;
3. the hardening stiffness of the plastic branch was chosen on the basis of the backbone curve;
4. the other parameters, such as the stiffness and the strength degradation factors, were evaluated in an iterative way until a good fit between the experimental curve and the model was obtained.

To speed up the calibration process of the spring components on the experimental results, the software So.ph.i. (acronym for SOftware for PHenomenological Implementations) has been developed using the Visual Basic .NET language [3]. So.ph.i. allows the user to visualize the results of the calibration made upon an experimental data set of a certain component [4]. In
addition, So.ph.i releases an input data file in the right format that will be used in the user subroutine implemented in Abaqus for cyclic modelling of the corresponding spring component. An automated calibration procedure based on EN 12512:2001 has been implemented in So.ph.i. This allows the user to obtain automatically elastic stiffness and yielding force values according to the code [5].

In Table 2 the mean characteristics of single component calculated from tests presented in [1] is shown. These values have been used in all subsequent analyses.

<table>
<thead>
<tr>
<th>Parameter description</th>
<th>Angle bracket: shear</th>
<th>Angle bracket: tension</th>
<th>Hold-down: tension</th>
<th>Hold-down: shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic stiff.</td>
<td>2.09</td>
<td>2.52</td>
<td>4.51</td>
<td>3.16</td>
</tr>
<tr>
<td>1st inel. stiff.</td>
<td>0.35</td>
<td>0.42</td>
<td>0.75</td>
<td>0.28</td>
</tr>
<tr>
<td>2nd inel. stiff.</td>
<td>-0.17</td>
<td>-0.40</td>
<td>-0.69</td>
<td>-0.13</td>
</tr>
<tr>
<td>Yielding force</td>
<td>22.98</td>
<td>19.22</td>
<td>40.46</td>
<td>3.64</td>
</tr>
<tr>
<td>Maximum force</td>
<td>27.71</td>
<td>23.47</td>
<td>48.33</td>
<td>14.51</td>
</tr>
<tr>
<td>Unloading ratio</td>
<td>0.94</td>
<td>0.98</td>
<td>0.99</td>
<td>0.81</td>
</tr>
<tr>
<td>Reloading ratio</td>
<td>0.72</td>
<td>0.88</td>
<td>0.89</td>
<td>0.60</td>
</tr>
<tr>
<td>Ultimate displ.</td>
<td>64</td>
<td>40</td>
<td>50</td>
<td>48</td>
</tr>
</tbody>
</table>

The diagonal springs for light-frame wall are calibrated on the experimental results presented in [2]. The parameters used are briefly presented in Table 3.

<table>
<thead>
<tr>
<th>Parameter description</th>
<th>Diagonal spring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic stiff.</td>
<td>4.95 kN/mm</td>
</tr>
<tr>
<td>1st inel. stiff.</td>
<td>0.42 kN/mm</td>
</tr>
<tr>
<td>2nd inel. stiff.</td>
<td>0.44 kN/mm</td>
</tr>
<tr>
<td>Yielding force</td>
<td>19.90 kN</td>
</tr>
<tr>
<td>Maximum force</td>
<td>33.10 kN</td>
</tr>
<tr>
<td>Unloading ratio</td>
<td>0.78 %</td>
</tr>
<tr>
<td>Reloading ratio</td>
<td>0.70 %</td>
</tr>
<tr>
<td>Ultimate displ.</td>
<td>60.35 mm</td>
</tr>
</tbody>
</table>

The diagonal springs for light-frame wall are calibrated on the experimental results presented in [2]. The parameters used are briefly presented in Table 3.

The experimental-numerical comparison is displayed in Figure 6, showing an overall acceptable approximation. It must also be pointed out that, unlike other software packages, no convergence problems arose at any time during the cyclic analysis carried out with the proposed model.
Also the total energy was calculated and compared with the experimental one, showing that the numerical values are quite close to the ones from the test.

**Figure 7:** Time-history of the total energy during the cyclic tests performed at IVALSA-CNR on a X-lam panel [1]

### 3.2 LIGHT-FRAME WALL

A light-frame wall, tested in [2] under cyclic loading (Figure 8), has been schematized with the proposed model.

The springs in this case are representing the entire wall, and they are placed along the wall diagonals (Figure 9).

**Figure 8:** Test set-up for the light-frame wall, from [2]

**Figure 9:** Beam elements and springs used to model the light-frame wall cyclic test

The hysteretic law used for diagonal springs is the same of angle brackets, and the kinematic behaviour of the element has lightly changed to return the proper force components in both in-plane degrees of freedom. Results in terms of base shear vs. top displacement and total energy are reported in Figures 10 and 11. The good fit with experimental data has to be ascribed to the direct calibration of the whole model on test results.

**Figure 10:** Numerical and experimental comparison on cyclic behaviour of light-frame wall
4 CONCLUSIONS

A component approach for non-linear dynamic analysis of structures made from cross-laminated solid timber or light-frame walls was presented in this paper. A spring for every type of connectors has been modelled in a phenomenological way. Based on experimental evidence, all energy dissipation was assumed to take place in the steel connectors, whilst the timber panel was regarded as linear elastic. Two different hysteretic loops characterized by a tri-linear backbone curve with significant pinching effect were implemented in Abaqus software package using external user-subroutines. Allowance for strength and stiffness degradation, based on the dissipated energy and maximum displacement attained, was also made. Such hysteretic laws were then fitted with the experimental results available for hold-downs, angle brackets, screwed connection or entire walls. A program was also developed to make the calibration of the hysteretic law on the experimental results easier. After calibration, the non-linear spring components were used to model a cyclic test carried out at CNR-Ivalsa on a full-scale single cross-lam panel, showing an overall acceptable accuracy both in terms of lateral displacement and dissipated energy. Also a light-frame wall under cyclic loading has been analysed with a good outcome. The presented modelling has been successfully extended to light-frame buildings using two diagonal springs with the non-linear relationship adopted for angle brackets. The proposed hysteretic model is very robust and unlike other software packages does not suffer from convergence problem. As such, it could be used to model the seismic performance of cross-laminated buildings and subassemblies. The cyclic behaviour of the steel connectors allows a correct estimation of the dissipated energy and, consequently, a reliable prediction of the seismic capacity of the whole timber building. The model will be used, in particular, to (i) fully understand the experimental results of the full scale 3-storey and 7-storey crosslam buildings tested on a shaking table in Japan [6]; (ii) extend the experimental results to different building configurations including non-residential destinations; and (iii) investigate the behaviour factors of multi-storey cross-lam timber buildings.

ACKNOWLEDGEMENT

The writers would like to acknowledge Prof. Ario Ceccotti and Mr. Igor Gavric for the experimental data of the cyclic tests carried out on crosslam panels at IVALSA CNR Trees and Timber Institute (Italy), and for the useful advice on the numerical modelling. The partial support of the first author from the Italian Civil Defense Department through the ‘RELUIS 2010-13 – Network of the Italian University Laboratories on Seismic Engineering’ research grant – Task 2.1.4. ‘Technological and Code Innovation in Seismic Engineering – Seismic Design of New Construction – Timber Structures’ is gratefully acknowledged.

REFERENCES

CREEP OF THIXOTROPIC ADHESIVES IN BONDED-IN TIMBER CONNECTIONS AS A FUNCTION OF TEMPERATURE AND HUMIDITY

Adlin Roseley¹, Martin P. Ansell², Dave Smedley³, Shane Porter⁴

ABSTRACT: Thixotropic, shear-thinning epoxy adhesives are the optimum choice for in situ assembly of timber structural joints under ambient conditions. These adhesives are suitable for injection into roof members and under bridge decks where gravity is a problem for conventional adhesives. However these ambient cure adhesives possess relatively low glass transition temperatures (Tg) and there are concerns associated with their stability under creep loads in conditions of high temperature and humidity. These concerns have been addressed in a paper presented at WCTE 2010 and in a subsequent journal publication where creep data generated in a dynamic mechanical thermal analyser (DMTA) has demonstrated the stability of these adhesives above Tg. In this paper results of creep tests on bonded-in connections, at a design shear stress of 2N/mm² at the rod to adhesive interface, are presented at temperatures of 20, 30, 40 and 50°C with levels of relative humidity (RH) of 65, 75, 85 and 95%. Creep experiments were conducted in a walk-in environmental chamber and creep was followed with laser displacement sensors. At intermediate combinations of temperature and RH level (e.g. 30°C, 75%RH), secondary creep was observed up to a limit. Under extreme conditions of high temperature and RH tertiary creep to failure occurred.

KEYWORDS: Ambient cure epoxy adhesives, thixotropy, timber, creep, viscoelasticity.

1 INTRODUCTION

Epoxy adhesives for the in situ repair of timber structures must be curable at ambient temperature and must be thixotropic (shear-thinning) when overhead injection is a necessity. Such low temperature curing systems should be able to withstand service conditions of relatively high temperature and humidity where there is a possibility of time-dependent creep or fatigue failure in bonded joints [1,2]

An investigation of creep in thixotropic epoxy adhesives using a Dynamic Mechanical Thermal Analyser (DMTA) indicated that between Tg and Tg + 15°C the adhesives creep to a limit, behaving initially as classic viscoelastic polymers but above Tg + 15°C behaving like rubbers with negligible creep [3,4]. This paper presents the results of an investigation of the creep behaviour of adhesively bonded timber to rod connections under several conditions of temperature and relative humidity.

2 EXPERIMENTAL METHODS

Two adhesives under investigation were two-part systems designed to cure at ambient temperature and formulated to exhibit shear thinning properties. The first adhesive, labelled Epoxy 1, contained silica fume nanoparticles and had a glass transition temperature (Tg) of 32.5°C measured with a DMTA. Epoxy 2 was a system filled with carboxyl-terminated butadiene acrylonitrile (CTBN) rubber and silica fume nanoparticles with a Tg of 50°C. The adhesives were thoroughly mixed to uniform consistency before application.

The samples produced for the creep study were double-ended samples and was designed to represent bonded-in joints in timber structures (Fig.2). The samples were fabricated by joining a Laminated Veneer Lumber (LVL) block to a timber block using a GFRP pultruded rod bonded into a centrally drilled hole. Attached to the GFRP rod was an aluminium plate allowed laser sensor displacement measurements to be made. Creep samples were tested in a walk-in environmental chamber (Fig.3). The LVL end of the double-ended sample was mounted on steel supports and the larger timber block supported attached weights. A constant static tensile creep load was applied equal to 20% of the bonded-in tensile strength. Temperature was held
constant at 20, 30, 40 and 50°C while the humidity level was held at 65%, 75%, 85% and 95% RH. A laser displacement sensor recorded the creep of the adhesive interface under constant load.

Figure 2. Double-ended samples

Figure 3. Creep test rigs located in an environmental chamber.

3 EXPERIMENTAL RESULTS

The initial creep response of joints adhesively bonded with Epoxies 1 & 2 is presented in Figures 4 and 5 for four combinations of temperature and RH.

Figure 4. Creep displacement versus time for Epoxy 1 as a function of temperature and relative humidity.

At 20°C and 65% RH, Epoxies 1 & 2 displayed negligible primary creep. After 250 minutes testing, Epoxy 1 exhibited a larger total displacement compared to Epoxy 2.

Figure 5. Creep displacement versus time for Epoxy 2 as a function of temperature and relative humidity.

The samples experienced more primary creep and subsequently settled down to a constant strain at 30°C and 75% RH. Much larger displacements were observed when the samples were tested under the extreme condition of 50°C and 95% RH when primary creep proceeded rapidly into secondary creep. At the end of a tertiary creep stage, Epoxy 1 samples failed before completing the 250 minute test. Epoxy 2 failed in tertiary creep after 192 minutes. The combination of high temperature and humidity clearly increased the susceptibility of the adhesive to creep failure as might be expected.

4 DISCUSSION AND CONCLUSIONS

The creep performance of two thixotropic timber adhesives has been examined as a function of temperature and relative humidity. The filled system has more resistance to creep under extreme conditions. In the full paper a more comprehensive set of creep results will be presented and mathematically modelled and the failure mode of bonded-in connections will be reported. It will be concluded that thixotropic adhesives perform a special role in allowing on site bonding of timber structures with sufficient structural integrity to prevent creep failure.

REFERENCES

PROPOSE OF DECAY-ACCELERATION METHOD FOR REAL SIZE COLUMN-SILL JOINT AND EVALUATION OF STRENGTH PROPERTIES

Takuro Mori¹, Kotaro Kawano², Kei Tanaka², Yoshiyuki Yanase³, Hiroshi Kurisaki⁴, Mitsunori Mori⁵, Yasunobu Noda⁵, Masafumi Inoue², Yasuhiro Hayashi⁶ and Kohei Komatsu¹

ABSTRACT: It is important to evaluate the existing strength of decayed structural members and joints in use for long term safety. In this study, the decay-acceleration method for the real size column-sill joints was proposed and the pull-out strength properties were evaluated. As a result, it was possible to prepare the partial decayed specimens of real size column-sill joint attacked by brown rot fungi. In this case, no significant differences of maximum load due to the duration of decay were shown. It was seemed that the smaller the maximum load was, the larger the penetration depth using Pilodyn® was.

KEYWORDS: Bio-deterioration, Column-sill joint, Ultrasonic velocity, Penetration depth, Brown rot fungi

1 INTRODUCTION

There are a lot of timber structures and wooden houses in Japan. Many of these structural members and joints have suffered from bio-deteriorations induced by fungi, termites and so on. It is very important to evaluate the existing strength of structural members and joints, and to decide when repair and replacement of the members in use for long term safety. It was not enough to accumulate the data of the relationship between existing strength of structural members or joints and bio-deterioration.

The objectives of this study are to propose the partial decay-acceleration method for the real size column-sill joint and to evaluate the strength properties. In addition, the relationship between the strength properties of decayed specimens and the penetration depth of Pilodyn® or the ultrasonic velocity propagating in the specimen was evaluated.

2 EXPERIMENTAL METHOD

2.1 SPECIMENS

Japanese cedar Sugi members were cut into specimens as shown in Fig. 1. The joint of column-sill was connected with the T-shape steel connector CP-T by nail ZN65.

2.2 DECAY-ACCELERATION METHOD

The method of preparation of specimens decayed partially by brown rot fungi (Fomitopsis palustris) was proposed. At first, specimens were brushed surfaces of joint part with ethanol (Fig. 2(a)) and air-dried. Then the stainless steel container containing the brown rot fungi on the potato dextrose culture (Fig. 2(b)) was installed at the joint part (Fig. 2(c)). Then the joint part and the
container were wrapped around with the PVC film to keep wet condition. Finally, there were put on the condition with the temperature of 28°C and the relative humidity of 75%. Durations of exposure were set for 2 months, 4 months and 12 months.

2.3 STATE OF DECAY
After the exposure test, the surface of specimen was covered by the fungi. It was seemed that the longer the duration of exposure was, the wider the extent that the fungi attached on the surface of specimens was (Fig.3(a-c)).

By cutting the specimens of decayed part and visual inspection of specimens after the pull-out test, the fungi attacked near the surface and the progress of fungi into the specimen was not observed, regardless of the duration of decay. There were some specimens which decayed around the nail (Fig.3(d)).

### Table 1: List of specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Duration of decay</th>
<th>State</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>CT-D</td>
<td>Sound</td>
<td>Dry</td>
<td>1</td>
</tr>
<tr>
<td>CT-W</td>
<td></td>
<td>Wet</td>
<td>1</td>
</tr>
<tr>
<td>CB-D</td>
<td></td>
<td>Dry</td>
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<tr>
<td>CB-W</td>
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<tr>
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<td>Wet</td>
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</tr>
<tr>
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</tr>
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<td>2B-W</td>
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</tr>
<tr>
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<td>3</td>
</tr>
<tr>
<td>4T-W</td>
<td></td>
<td>Wet</td>
<td>1</td>
</tr>
<tr>
<td>4B-D</td>
<td></td>
<td>Dry</td>
<td>1</td>
</tr>
<tr>
<td>4B-W</td>
<td></td>
<td>Wet</td>
<td>1</td>
</tr>
<tr>
<td>12T-D</td>
<td>Twelve months</td>
<td>Dry</td>
<td>6</td>
</tr>
<tr>
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<td></td>
<td>Wet</td>
<td>2</td>
</tr>
<tr>
<td>12B-D</td>
<td></td>
<td>Dry</td>
<td>2</td>
</tr>
<tr>
<td>12B-W</td>
<td></td>
<td>Wet</td>
<td>2</td>
</tr>
</tbody>
</table>

2.4 MEASUREMENT OF DIAGNOSTIC EQUIPMENT
To evaluate the degree of decay before or after the exposure test, the velocity of ultrasound propagating in the wood specimen using Dr. WOOD® (Fig.4(a-b)), and the penetration depth into the specimen using Pilodyn® (Fig.4(c-d), moisture content using Wood Moisture

![Figure 1: Shape and size of specimens (Unit: mm)](image1)

![Figure 2: Decay-acceleration method](image2)

![Figure 3: State of decay](image3)

![Figure 4: Device of diagnose bio-deterioration](image4)
Tester® HM-520 were measured non-destructively. It was also investigated the volume density after pull-out test. The ultrasonic velocity was measured in the three or four patterns at the column or the sill, respectively (Fig.5). The penetration depth was measured at the thirteen places with 50mm intervals at the sill (Fig.6).

Figure 5: Measurement position of velocity of ultrasonic sound propagation

Figure 6: Measurement position of penetration depth

2.5 PULL-OUT TEST METHOD

Specimens, shown in Fig. 1, were applied for cyclic and monotonic pull-out load test by the hydraulic jack, and the relative displacement was measured between the column and the sill (Fig.7).

3 RESULTS AND DISCUSSIONS

3.1 MEASURED VALUE OF DIAGNOSTIC EQUIPMENT

Figure 8 shows the difference of ultrasonic velocity at the bottom of column before and after decay in 2T and 2B. The velocities of decayed specimens were faster than those of sound specimens. Similar results were also shown in the specimens of 4T, 4B, 12T and 12B. It was difficult to evaluate by using this equipment in this case. Figure 9 shows the difference of penetration depth between the sound part with far distance from the joint and the decayed part. Penetration depth of decayed part was larger than the depth of sound part.

Figure 7: Set-up for pull-out test

Figure 8: The difference of ultrasonic propagation velocity at column and sill

Figure 9: The difference of penetration depth

3.2 RELATIONSHIP BETWEEN LOAD AND DISPLACEMENT AND FAILURE MODE OF PULL-OUT TEST

Figure 10 shows the typical relationship between load and displacement. Similar tendency that the load reduced gradually after reaching maximum load after splitting failure occurred at the sill was shown in the all specimens. The deformation of steel connector and the pull-out of nail without splitting failure of the sill was only observed in the wet specimens.
Figure 11 shows the failure mode due to pull-out test. The three kinds of failure mode were shown as follows:
1) the splitting failure of sill,
2) the pull-out of the nail from sill,
3) the deformation or failure of steel connector.

3.3 COMPARISON WITH DURATION OF DECAY PERIOD AND MAXIMUM LOAD

Figure 12 shows the maximum load of each specimen. No significant differences of maximum load due to the duration of decay were shown, though wet specimens of 4T and 12T showed the strength reduction.

3.4 THE RELATIONSHIPS BETWEEN VALUE OF DIAGNOSTIC EQUIPMENT AND RESULTS OF PULL-OUT TEST

Figure 3(a-c) shows the state of decayed specimen after 2, 4 and 12 months. The surface of specimen attacked by fungi was softened and cracked, and the area attacked by fungi corresponded to the wrapped area. This decay method has enabled the partially decayed specimen, because the part of specimen to connect with the base or the jig for pull-out test was not decayed.

Figure 13 shows the relationship between the maximum load and the ultrasonic velocity of radial direction at the bottom of column and the correlation coefficient (r). In the specimen of 4T and 12T, it was seemed that the larger the maximum load was, the larger the ultrasonic velocity was. But 2T and CT were shown opposite tendency.

Figure 14 shows the relationship between the maximum load and the ultrasonic velocity of radial direction at near the joint of sill and the correlation coefficient (r). In the specimen of CT, no significant difference between the maximum load and the ultrasonic velocity was shown. In the 4T, it was seemed that the larger the maximum load was, the larger the ultrasonic velocity was. But 2T and 12T were shown opposite tendency.

Figure 15 shows the relationship between the maximum load and the penetration depth. It was seemed that the smaller the maximum load was, the larger the penetration depth was. In this study, the penetration depth of over 30mm was not shown in these specimens, thus the nail showed as much the same performance as initial one.

Figure 16 shows the relationship between the maximum load and the volume density at the sill with decayed part. In the specimens except CT, acceptable correlation was
shown, though the volume density was difficult to obtain without destructive.

\[ \text{Figure 13: Relationship between maximum load and ultrasonic velocity of radial direction at the bottom of column (m/s)} \]

\[ \text{Figure 14: Relationship between maximum load and ultrasonic velocity of radial direction at the sill near the joint} \]

\[ \text{Figure 15: Relationships between maximum load and penetration depth at center of sill} \]

\[ \text{Figure 16: Relationships between maximum load and volume density at sill of decayed part} \]

4 CONCLUSIONS

The partial-decayed specimens of real size column-sill joint attacked by brown rot fungi were prepared, and the pull-out strength properties and the performance of the equipment of diagnose bio-deterioration were evaluated. The results are as following,

1. The method of the partial decay-acceleration has enabled the partial-decayed specimen for real size test.
2. The data of the strength properties of decayed column-sill joint was accumulated.
3. No significant differences of maximum load due to the duration of decay were shown, though wet specimens of 4T and 12T showed the strength reduction.
4. In the pull-out test, it was seemed that the smaller the maximum load was, the larger the penetration depth was.

For the future, we will make more severe damaged specimen and evaluate them.

ACKNOWLEDGEMENT

This work was supported by the Grants-in-Aid for Scientific Research of JSPS (Head: Takuro Mori, 21686053) and the Grant-in-Aid for Scientific Research (A) (Head: Yasuhiro Hayashi, 22246072), and Collaborative Research Program RISH Kyoto University.
ENDURING PERFORMANCE OF SELF-TAPPING SCREW CONNECTION IN WOOD STRUCTURE MEMBERS

Xiaojun Yang, Youfu Sun, Zheng Wang

ABSTRACT: In order to learn the enduring performance of self-tapping screw in wood structure members, screw connection’s creep performance has been studied through the experimental method of screw holding power. The study shows that: the intensity of screw connection lies in the occlusion level and the force of friction between wood and screw; Burger mode can precisely simulate pine’s (Pinus Stp.) short-term creep performance. In wet condition, the amount of creep is apparently larger than that in dry condition. Temperature and relative humidity are two important factors that influence creep. The higher stress level is, the larger amount of screw holding creep. Creep rate is the largest in the wet condition and high stress level. Therefore, screw connection becomes invalid in short time. Under the consideration of safety, screw holding power should be controlled within 40% of the maximum stress level in the long term.

KEYWORDS: Wood structure members, Screw connection, Creep performance

1 INTRODUCTION

In China, modern wood structure building has developed fast in recent years. A series of building codes have already been issued. They are symbols of the resurrection of wood structure building which has been in a state of standstill for more than 20 years in China [1]. Modern wood structure building is drawing more attention than ever and has won the approval of professional personage. Metal pieces are indispensable parts of wood structure building. It mainly acts as a connecting function. Screw connections are the most commonly used for their great strengths such as tightness, tenacity, simple construction, safety and reliability [2,3]. For example, nail-beam connection is applied in roof slab and floor slab; oblique nail-baseboard connection in wall bone beam of light wood structure building; screw-wall bone beam connection in inside and outside panel of shear wall and so on. Screw connection’s reliability is often tested by holding power. Owing to holding power’s short test period and its long-term stress while used in wood structure members, reliability of screw connection will gradually decline with the length of its service. If lower than structure members’ stress, “screw nipping” appear. At present, the study on enduring performance of screw connection has seldom been reported. As wood and wood materials are all viscoelasticity, screw connection in wood structure members also possesses viscoelasticity. Therefore, with the viscoelasticity theory applied and often-used screw as the object of study, short-term creep performance in wood structure members has been studied in order to provide instruction on optimal use of screw.

2 CREEP MODEL

The intensity of screw connection normally lies in the occlusion level and the force of friction between wood and screw[4]. Connection parts are made of elastic wood layer and iron. The stress-strain behavior of connection parts combines both characteristics of spring material and elastic material. In this paper, viscoelasticity behaviour of screw connection is to be simulated with the help of the Burger mode that is frequently used in wood creep [5-10]. The model is described by Equation (1).

\[ \varepsilon = \sigma \left[ \frac{1}{K_c} + \frac{1}{K_{de}} \left( 1 - e^{-\frac{t}{\eta \sigma}} \right) + \frac{t}{\eta} \right] \]

where \( \sigma \) = stress, \( \varepsilon \) = strain, \( K = \) Coefficient of elastic deformation, \( \eta = \) Viscosity coefficient and \( t = \) Load time.

In tensile test, Equation (1) can be transformed into Equation (2).

\[ Y(t) = A_1 + A_2 \left[ 1 - \exp(-A_3 t) \right] + A_4 t \]

where \( Y(t) = \) Creep function with time history, \( A = \) Undetermined coefficient and \( t = \) Load time.

3 EXPERIMENTAL METHODS

3.1 MATERIALS

In this paper, wood for research is pine (Pinus Stp.) of SPF (spruce-pine-fir), basic parameters of which are the first class timber, air dry density 0.581 g/cm³, moisture content 13%. Self-tapping screw is galvanized wood screw (for use only in wood structure) with its diameter: 3.2mm, length: 40mm.

3.2 TEST SAMPLE PROCESS

Testing wood is selected from the same lumber in order to minimize the influence of selecting difference on

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testing result. Testing dimension of pine is 90 mm (length) x 80 mm (width) x 38 mm (height). Drilled and implanted with screw in the middle of wood samples’ wide face, its basic parameter is guide hole diameter 2.8 mm, guide hole depth 25 mm, magnitude of interference between screw and wood hole 1.4mm.

3.3 SCREW HOLDING POWER TEST
In accordance with the GB/T 14018-1992 [11] wood holding power testing method, pine’s holding power is measured separately from universal testing machine. Its value is 2 892.5 N (average of 10 testing wood, coefficient of variation 4.3%).

3.4 CREEP TEST
Creep experimental design: screw holding stress level is the average maximum holding power’s 40%, 50% or 70%. Testing condition: (1) dry condition (temperature 24 ℃, relative humidity 45%); (2) wet condition (temperature 50 ℃, relative humidity 90%). Testing period is 720 minutes. Creep experimental design of samples is illustrated in Table 1.

<table>
<thead>
<tr>
<th>Sample name</th>
<th>Stress level (%)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
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<td>LW</td>
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<td>70</td>
<td>Dry</td>
</tr>
<tr>
<td>HW</td>
<td>70</td>
<td>Wet</td>
</tr>
</tbody>
</table>

3.5 CREEP PROGRAM DESIGN OF TESTING MACHINE

The experimental equipment is GDS-100/UTM4304 high-low temperature and wet-hot creep lasting test machine. Its testing program is set as follows: a. displacement control (5 mm / min) as start control mode; b. sampling interval (30-600 sec) as start control mode; c. loading time (720 min) as start control mode, power control (on) as final control mode. Loading method of screw holding is as shown in Figure 1.

4 RESULTS AND DISCUSSION

![Figure 1: Chart of loading method](image)

![Figure 2: Fitted data and chart of sample LD’s screw holding creep](image)

![Figure 3: Fitted data and chart of sample LW’s screw holding creep](image)

![Figure 4: Fitted data and chart of sample MD’s screw holding creep](image)
After test samples destroyed, screw thread groove is filled with tiny wood; screw thread and timber become completely interlocked; the results show that method of making testing sample is reasonable. As indicated from Figure 8, during the load process, screw load increases linearly rapidly with the increase of pullout displacement. Under the function of interlocking and friction drag, test sample’s holding power reaches its peak level and then gradually declines. Here screw connection is mainly effected by friction. As indicated from Figure 2 to Figure 7, under the function of 3 stress levels, every creep curve’s deformation rule, which exhibits preliminary and steady creep’s short-term characteristics, is nearly consistent. Preliminary creep is unstable. Its feature is that the beginning creep velocity is high, but with time goes on the velocity declines. The characteristic of steady creep is that creep develops with the lowest velocity under certain stress level and condition, this characteristic is illustrated with low slope straight line on the creep curve. Burger creep model (2) can be used as regression equation, which is applied to data processing of each creep curve respectively in dry and wet environment. Equation parameters $A_1$, $A_2$, $A_3$, $A_4$ are indicated in Figure 1. Accurate representation of creep and correlation coefficient $R^2$ above 99% are indicative of Burger model’s application in screw connection creep properties. 

Samples (between LD and LW, MD and MW, HD and HW) in the same stress level and different condition, their instantaneous elastic deformations are almost the same; however, along with the load time extending, the creep deformation difference increases gradually. The reason for this is that samples for creep testing are transferred gradually from room temperature condition to test box condition. Screw connection parts during the initial loading are almost in the same state. As time continues, wood in the connection parts becomes adapted to the test box condition. Since wood absorbs moisture from air in the wet condition, texture becomes soft; furthermore, moisture absorption creep phenomenon inevitably appears in wood connection parts. This phenomenon leads to the rate increase of creep deformation. In the wet environment and stress
level of 70%, screw parts break down at 460 min, which indicates that screw connection parts have poor high temperature and high humidity resistance. Therefore, when applied in engineering, screw parts should be protected from dampness and necessary measurements should be taken to handle protective treatment in these parts.

Among the entire test samples in dry condition and different stress level (40%, 50% and 70%), creep deformation differs greatly in stress levels but follow the same rule. In the stress level of 40% and 50%, creep deformation rate increases slowly; however, in the stress level of 70%, creep deformation rate increases obviously higher than in other stress level and the possibility of creep damage will be improved. In wet condition, creep deformation rate is higher than in other conditions. Test samples in stress level of 50% and 70% are all destroyed in a short time. Model parameter A1 reflects that in the scope of elasticity, the binding layer’s ability of resistance to deformation in tensile stress. Stress level and A1 ratio are almost unanimous. Screw and wood connection parts’ elastic properties have less to do with stress level and they can be recovered completely after unloading. A2 and A3 show connection parts’ viscoelastic deformation. Based on the fitted data from Figure 2 to Figure 7, A2 and A3 are closely related to stress level and environment. The higher stress level or wet environment, the less wood connection parts’ ability to restore original size. A4 reflects connection layer’s viscosity deformation. The higher stress level or wet environment, the greater wood connection parts’ permanent deformation.

5 CONCLUSIONS

Self-tapping screw’s joint strength depends on interlocking and friction drag between wood and screw. Screw connection parts’ creep deformation appears in a relative short period of time. It depends on the value of tensile stress and loading time. Screw connection parts’ short-term creep process can be divided into two phases; First phase: it increases rapidly in a nonlinear way; Second phase: it grows linearly with low rate. Under the wet condition, connection parts’ creep value is apparently larger than that in dry condition. Connection parts can be easily destroyed in wet condition and high stress level. Temperature and relative humidity are two important factors that influence creep. Under the consideration of safety, screw holding power should be controlled within 40% of the peak stress level in the long term. Burgers mode can be used to simulate short-term creep of pine’s connection parts under wet or dry condition. The accuracy is sufficient to meet the requirements of engineering. Creep deformation increases with the growth of stress level. The creep curves in different stress level show the same law, namely, creep deformation appears in an early phase and then slows down in the later phase. Creep testing time can’t completely reflect screw connection parts’ creep properties. Further study on the long-term creep will be carried out in the future.

ACKNOWLEDGEMENT

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REFERENCES

CONSISTENT YIELD MODEL FOR STRENGTH PREDICTION OF TIMBER RIVET CONNECTION UNDER DUCTILE FAILURE

Pouyan Zarnani¹ and Pierre Quenneville²

ABSTRACT: The timber rivet is one of the best timber connection fasteners due to its cost-efficiency, high strength capacity and its ability to behave in a ductile manner. In structures such where the ductile behaviour and energy dissipation of the connections can be desirable under dynamic loads, the application of rivet connection would ensure more structural safety. Buchanan and Lai [1] and Stahl et al. [2] applied the European Yield Model (EYM) developed by Johansen [3] to predict the rivet connection capacity under ductile failure mode, however, the yield theory predicts no increase of strength for longer rivets though the actual tests show higher rivet capacity up to 25%. In the proposed model for ductile failure, an additional withdrawal and frictional contribution to the lateral load-bearing capacity due to the rivet distortion during yielding is considered to fill this gap. In addition, methods are developed for the embedment and withdrawal resistance to be applied in the yield model. Results of current tests on LVL and data available from literature on glulam confirm the validity of this new analysis and show that this predictive analytical model can be used as design provision for timber rivet connections in engineered wood products for more precise predictions of the ductile failure mode.

KEYWORDS: Timber structure, connection, rivet, ductile failure, EYM

INTRODUCTION

Rivets are tight-fit fasteners used to make high strength timber connections. The timber rivet connections have been used successfully in Canada and U.S. in the different types of structures over the last 3 decades such as in connections of a glulam arch spanning 100 meters and high span trusses. Rivets are a cost effective alternative to large fasteners such as bolts which cause large localized stresses and can force brittle ruptures in the timber. In structures where the ductile behaviour and energy dissipation of the connections can be desirable due to the applied wind and seismic loads, the use of rivet connection would assure more structural safety under these dynamic loads [3].

For riveted connections, there are two major mechanisms of failure; the brittle tear-out of a plug of wood defined by the rivet’s perimeter [4,5] and the ductile yielding of rivets with localized wood crushing. For predicting the ductile capacity of dowel-type fasteners such as bolts, nails and screws the well-known European Yield Model (EYM) is used. The EYM is based on the yield theory developed by Johansen [6]. He provided the foundation for the prediction formulas in Eurocode 5 [7]. He assumed a rigid-plastic behavior for wood under bearing and dowel in bending. Also, he assumed that no axial tension occurred in the fastener and, thus, no frictional and withdrawal resistance contribution to the lateral load-bearing capacity were considered. Based on the possible number of plastic hinge formation in the fastener for different ductile failure modes, the resistance can be determined by taking the least value. Because rivets are always used in single shear, and because the rivet head can be considered to be rotationally fixed where it is wedged into the steel plate’s hole, only three yield modes need to be considered (Fig. 1).

![Figure 1: Ductile failure modes for riveted connection](image)

Buchanan and Lai [1] and Stahl et al. [2] applied the European Yield Model (EYM) to predict the rivet connection capacity under ductile failure mode and they obtained good estimates. However, for longer rivets with length of 65 and 90 mm which fail in Mode III and IV respectively, the yield theory predicts no increase of strength though the actual tests show higher rivet capacity up to 25%. In the proposed model for ductile failure, an additional withdrawal and frictional contribution to the lateral load-bearing capacity due to the rivet distortion during yielding is considered to fill this gap.

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PROPOSED CONSISTENT YIELD MODEL

The length where the rivet compresses the wood during yielding and causes localized wood crushing can be interpreted according to Figure 2 as the effective thickness, \( t_p \) [8].

\[
P_r = \min \left\{ \begin{array}{ll}
\frac{f_{h,0}}{d_p} d_i & \text{Mode I}_m \\
\frac{f_{h,0}}{d_p} (2t_f - L_p) + P_{ra} & \text{Mode III}_m \\
\frac{f_{h,0}}{d_p} d_i + P_{ra} & \text{Mode IV}
\end{array} \right.
\]  

(1)

in which,

\[
P_{ra} = \frac{t_f}{4.8^2 + t_f^2} C_f C_w F_w + \frac{4.8}{4.8^2 + t_f^2} C_w F_w
\]  

(2)

and,

\[
t_f = \left\{ \begin{array}{ll}
\frac{L_p}{M_{y,f} + 4L_p^2} & \text{Mode I}_m \\
\frac{M_{y,f} + L_p^2}{2} & \text{Mode III}_m \\
2 M_{y,f} & \text{Mode IV}
\end{array} \right.
\]  

(3)

**Figure 2:** Effective thickness based on the rivet embedded length in different failure modes

Proposed model (Equations 1-3) is based on EYM prediction plus additional lateral resistance, \( P_{ra} \). This additional strength is caused by the force components from vector resolution of the rivet inclined withdrawal resistance, \( F_r \), which would appear as the interface friction between the plate and wood and direct lateral resistance (Figure 3). This extra resistance occurs just in failure Mode III\(_m\) and IV and is directly proportional to the rivet penetration length.

\[
P_r = \min \left\{ \begin{array}{ll}
\frac{f_{h,0}}{d_p} f_{ef} d_i & \text{Mode I}_m \\
\frac{f_{h,0}}{d_p} (2t_f - L_p) + P_{ra} & \text{Mode III}_m \\
\frac{f_{h,0}}{d_p} f_{ef} d_i + P_{ra} & \text{Mode IV}
\end{array} \right.
\]  

Figure 3: Additional lateral resistance induced by the distortion of the rivet

and \( d_i \) is the rivet cross-section dimension bearing on the wood parallel-to-grain, (equal to 3.2 mm), \( M_{y,f} \) is the parallel-to-grain moment capacity of the rivet, (equal to 30000 Nmm, [2]), and \( C_f \) is the friction coefficient between the steel plate and wood surface which can be assumed 0.7. For the best agreement between data from current tests and literature with predictions, the factor \( C_p \) was adjusted to 0.2 to account for the effective withdrawal resistance acting after the rivet deformation.

The ultimate rivet yielding load \( P_r \) is the highest load reached at displacements up to 4.8 mm as shown in Figure 3. This common measure of ultimate load on a rivet group is based on a maximum displacement equal to the average of the rivet’s cross section dimensions; 6.4 by 3.2 mm.

**PREDICTION OF EMBEDMENT STRENGTH**

Frank and Quenneville [9] proposed Equations 4 and 5 for bolt yield embedding strength \( f_{h,5\%} \) of Radiata Pine based on 5%-offset method.

\[
f_{h,0} (\text{parallel}) = \begin{cases} 0.075(1-0.0037d)p & \text{for LVL} \\ 0.072(1-0.0024d)p & \text{for Lumber} \end{cases}
\]  

(4)

\[
f_{h,90} (\text{perpendicular}) = \begin{cases} f_{h,0}/1.5 & \text{for LVL} \\ f_{h,0}/2.0 & \text{for Lumber} \end{cases}
\]  

(5)

where \( d \) is the dowel diameter and \( p \) is the mean density of the specimen. The values for lumber is valid for glulam as well.

The same prediction equations were adopted with an additional 20% increase. The reason for applying this increasing factor is related to the fact that the embedment strengths obtained using the 5% offset load are found to vary between 80% and 86% (with average of 83%) of the values at a displacement of 2.1 mm [1]. The definition of 2.1 mm deformation is the limit used by Hilson et al. [10] and Buchanan and Lai [1] for measuring embedment strength. Therefore, by applying the mentioned correction factor to Eqs. 4 and 5 the rivet bearing strengths correspondant to 2.1 mm deflection can be written as

\[
f_{h,0} = \begin{cases} 0.090(1-0.0037d)p & \text{for LVL} \\ 0.086(1-0.0024d)p & \text{for glulam} \end{cases}
\]  

(6)

\[
f_{h,90} = \begin{cases} 0.060(1-0.0037d)p & \text{for LVL} \\ 0.043(1-0.0024d)p & \text{for glulam} \end{cases}
\]  

(7)

where \( d_p \) = rivet cross-section dimension bearing on the wood perpendicular-to-grain, (equal to 6.4 mm).

The proposed model for determining the embedment strength for rivets was verified by conducting tests on LVL and glulam (Fig. 4). The test setup shown in figure 4 is similar to that used by Buchanan and Lai [1]. The embedment strength was defined as the strength corresponding to 2.1 mm deflection based on the recorded load-slip curves (Fig. 5). As shown in Table 1, there is a good agreement between the predictions and the test values. This predictive method can be used in the proposed yield model.
PREDICTION OF WITHDRAWAL RESISTANCE

As the rivet is driven into place, the wood fibres are pushed to each side. The fibres are compressed by the rivets creating friction between fibres and the rivets [11]. The major withdrawal resistance of the rivet is caused by the friction force acting on its shank surfaces along the grain. This friction is proportional with the normal force establishes by bearing of the wood perpendicular to grain after the rivet being driven (Fig. 6). Since the dimension of the rivet across the grain is 3.2 mm so the induced normal stress on each shank surface can be predicted as the load causes embedding deformation of the wood perpendicular to grain equal to half of this dimension, 1.6 mm.

Coefficient of variation for LVL and glulam was about 11% and 14% respectively, similar for both directions.

<table>
<thead>
<tr>
<th>Wood product</th>
<th>Density (kg/m³)</th>
<th>Predictions (MPa)</th>
<th>Test mean values (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( f_{h,0} )</td>
<td>( f_{h,90} )</td>
</tr>
<tr>
<td>RP-LVL</td>
<td>590</td>
<td>52.7</td>
<td>34.7</td>
</tr>
<tr>
<td>RP-glulam</td>
<td>480</td>
<td>41.3</td>
<td>20.5</td>
</tr>
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</table>

Coefficient of variation for LVL and glulam was about 11% and 14% respectively, similar for both directions.
Based on the nonlinear load-slip plots for embedding strength perpendicular-to-grain from conducted tests, the load causing wood bearing deformation perpendicular-to-grain at 1.6 mm is approximately 90% of the load at 2.1 mm deflection. Therefore, the withdrawal resistance of the rivet, \( F_w \), can be determined by

\[
F_w = L_p f_w
\]

where

\[
f_w = 0.9 C_p L_p / f_{h,90}
\]

\( L_p \) is the shank dimension along the grain equal to 2 times 6.4 mm and \( C_p \) is the calibration factor equal to 0.3 for the best fit with the results from current withdrawal tests on Radiata Pine LVL and glulam and tests done by Karacabeyli and his colleagues [12] in Douglas fir-Larch. The proposed model for withdrawal strength for rivets is verified by conducted tests on LVL and glulam (Fig. 7).

\[ \text{Figure 7: Test setup for withdrawal strength tests} \]

The average withdrawal strength derived from the test load-slip curves (Fig. 8) was 113.4 N/mm for Radiata Pine LVL and 43.9 N/mm for Radiata Pine glulam with a coefficient of variation of 9.8% and 24.3% respectively. The predicted values for RP LVL and glulam withdrawal strength are 83.9 and 49.6 N/mm correspondingly which are in line with the test mean values. The prediction equations 8 and 9 can be used for \( F_w \) value in the proposed yield model (Eq. 1). It is important to note that the specimens were tested 1 hour after assembly. Therefore, it is recommended to consider a reduction in withdrawal strength due to relaxation of the wood fibres around the rivet. A decrease of 30% in mean ultimate withdrawal load was observed by Karacabeyli et al. [12] after a waiting period of two months. The rate of this reduction with time is unknown, and needs further study.

\[ \text{RESULTS AND DISCUSSION} \]

As shown in Figure 11, a comparison was done using the current rivet yielding tests (Fig. 9 and 10) on Radiata Pine LVL and glulam and data from literature under ductile mode failures to validate the proposed analysis. In addition, the new analysis was compared with other prediction models proposed by Stahl and the Canadian O86-09 code [13]. The Stahl’s model [2] is based on AFPA [14] dowel fastener yield model which is a version of the EYM. Also, the Canadian code defines the rivet capacity using a parabolic function for the rivet penetration length. Two sets of data were collected from the literature: tests performed by Buchanan and Lai [1] on Radiata Pine glulam and those of Stahl et al. [2] on Southern Pine glulam.

\[ \text{Figure 9: Yielding modes of the rivets in ductile failure} \]

\[ \text{Figure 8: Load-slip plots for withdrawal tests; (a) RP-LVL, (b) RP-glulam} \]
It can be noted that the predictions by the Canadian code are above the line therefore non-conservative (Figure 11). Also, Stahl’s model which is based on the yield theory predicts similar values for longer rivets though the actual tests show up to 25% increase on the lateral resistance capacity.

Figure 10: Typical load slip curves for rivet yielding tests parallel to grain in RP-LVL

Figure 11: Comparison of analyses and test data in ductile failure modes

CONCLUSIONS
The proposed analysis for ductile failure mode improved the predictions with a correlation coefficient ($r^2$) of 0.92. In this new analysis, supplemental lateral resistance which is directly proportional to the rivet penetration length is added up to EYM predictions to have a predictive method based on a consistent yield model applicable for different LVL and glulam wood products. In addition, the model includes methods to predict the embedment and withdrawal strengths to be used in the model.

ACKNOLEDGEMENT
The authors wish to thank the Structural Timber Innovation Company (STIC) for funding this research work.

REFERENCES
RESEARCH ON COMPOSITE ACTION BETWEEN SHEATHING AND JOIST IN TIMBER FLOOR: TEST, FORMULATION, AND FINITE ELEMENT MODEL ANALYSIS

Haibei Xiong¹, Jiahua Kang², Xilin Lu³

ABSTRACT: In this paper 12 rectangle-shaped section beams (RB) and 12 T-shaped section composite beams (CTB) were tested to experimentally examine the bending performance of the composite timber beam. Besides, the finite element model of CTB (CTB-FEM) was also developed and modelled to investigate the composite action in the CTB. Results showed that relative slip between the web and the flange (RS) in the CTB was small enough to ensure the composite action and load sharing between the flange and the web. It was demonstrated that, as expected, the bending stiffness and capacity of the CTB with 150mm nail spacing (NS) was the largest, followed by the CTB with 100 mm-NS, and ending with the CTB with 75 mm-NS. The results of the study clearly demonstrate that composite action should be taken into account to economically design timber floors. A classic formulation was introduced and validated, and it was recognized that the formulation can be used to timber floor stiffness calculation. Besides, the method that developed the CTB-FEM could be easily expanded to establish the timber floor finite element model that can be used to do floor vibration simulation induced by pedestrian walking with consideration of the composite action between the sub-floors and the joists in timber floor.

KEYWORDS: CTB; nail connection; formulation; stiffness calculation; CTB-FEM

1 INTRODUCTION

Timber floors are widely used in wood-framed constructions around the world. The timber floor mainly contains multiple parallel joists (dimension lumber, or I-Joist, or Truss), sub-floor panels (OSB or Plywood), blockings, and a concrete topping. Generally, the timber floor has been design by considering the joists as the only structural elements that act as simple beams with assumption of similar properties and acting independently using strength-based or deflection-based rules. It has been assumed that the wood-based sub-floor’s only function is to transfer loads to the joists. Actually, the joist does not act alone as a simple beam in carrying the imposed loads, the sub-floor panel acts with the joist to form a CTB [1], and in the CTB, the joist acts as the web and the sub-floor panel acts as the flange. Several researchers [2][3][4][5] have investigated and testified the composite action between the sub-floor panels and the joists in the timber floor. These studies demonstrated that the composite action can add significantly to a timber floor’s performance. On the other hand, it was found that the strength-based and deflection-based design criterions were sometimes failed to control the annoying vibration of the timber floor induced by human walking. Lots of studies have been conducted in the timber floor vibration research area, and the vibration control-based design methods have been developed, for example, in the Forintek report [6], limitation value on the combination of the fundamental frequency ($f$) and floor centre point-load deflection ($d$), namely, $d < (f/18.7)^{2.27}$, was recommended as the design criterion in order to preventing the annoying timber floor vibration. All of these methods required the exact calculation of the parameters, such as the fundamental frequency, point-load deflection of the timber floor based on the integrated floor system standpoint. So the study focused on the composite action between the sub-floor panels and the joists in the timber floor were conducted and presented in this paper. The study aimed to introduce a method that can be used for calculating the stiffness of the CTB based on test results, which then could be recommended by code for the timber floor stiffness calculation. Besides, a CTB-FEM was developed and validated, which then could be easily expanded to a finite element model of timber floor [7].
(TF-FEM) for simulating the vibration response induced by pedestrian force.

2 TIMBER BEAM TEST AND RESULTS

2.1 TEST INTRODUCTION

In order to experimentally investigate the composite action between the sub-floor panels and joists in a timber floor, 12 rectangular-shaped section beams (RB) and 12 CTBs were designed for testing (figure 1). There were three different NS adopted in the CTB, namely, 150 mm, 100 mm, and 75 mm, and for each NS there were four beam specimens.

![Figure 1: Beam specimen](image1)

In the test beam deflection at location A ~ E was recorded by the displacement transducers (DT). RS in the CTB were measured also by DT at location F ~ I. The load was applied by the hydraulic actuator (HA) and the reaction force was recorded by the pressure transducer (PT). Strain gauges (SG) were adopted to measure the strain of the flange at mid-span as well as the web strain. A pc-based data acquisition system capable of simultaneous data acquisition was adopted in the test. All the beams were tested on a 2760 mm span under four-point loading using force-control protocol with rate of 1.5kN/min. The test setup (figure 2) and loading protocol were referred to China code for timber structure test [8] and ASTM [9][10]. Figure 3 showed a CTB in test.

![Figure 2: Test setup](image2)

2.2 TEST RESULTS

Load-deflection behavior comparison between CTB with different NS and RB when the mid-span deflection was smaller than the maximum value regulated in China code for timber design [11] was shown in figure 4. One can see that there was no nonlinear performance of the timber beams before the mid-span deflection reached the maximum allowable value. And based on the comparison between the load-deflection fitting curves (figure 4), it was found that the bending stiffness of the CTB was larger than that of the RB, and the stiffness was increased with NS decreasing. The CTB with 150 mm-NS, 100 mm-NS, and 75 mm-NCS, had a bending stiffness increment of 13.27%, 40.31%, and 57.73% compared with the RB, respectively. The stiffness increment percentage showed a very obvious linear relationship with the NC (figure 5).

![Figure 3: CTB in test](image3)

![Figure 4: Load-deflection behavior comparison (fitting curves)](image4)

![Figure 5: Stiffness increment–NC relationship](image5)
Before mid-span deflection of the CTB with 150 mm-NS reached the maximum allowable value, the averaged RS at locations F and I, and G and H were 0.76 mm and 0.29 mm, respectively. The most important thing was that the RS was decreased with NS decreasing (table 1). Typical load-slip behavior of the sheathing-to-lumber nail connection was shown in figure 6 obtained from the monotonic test on 15 nail connection specimens, it can be found that the connect stiffness at low slip level was large, for example, the secant stiffness at 1 mm relative slip was 800 N/mm, and the tangent stiffness was 1100 N/mm approximately. So the nail connection fastened the flange to the web in CTB had the ability to transfer the interaction of the two components. Besides, flange strain at mid span was detected, so axial force existed in the flange when the CTB was bended. Based on the discussion mentioned above we could concluded that the nail connection can transfer the interaction between them when the CTB was bended, which made the CTB performance better than that of the RB.

Table 1: Averaged RS in CTB with different NS (Unit: mm)

<table>
<thead>
<tr>
<th>Beams</th>
<th>Locations</th>
<th>RS (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTB (NS 150mm)</td>
<td>F and I</td>
<td>0.76/1.19</td>
</tr>
<tr>
<td></td>
<td>G and H</td>
<td>0.29/0.71</td>
</tr>
<tr>
<td>CTB (NS 100mm)</td>
<td></td>
<td>0.66/1.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.22/0.63</td>
</tr>
<tr>
<td>CTB (NS 75 mm)</td>
<td></td>
<td>0.60/0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.13/0.56</td>
</tr>
</tbody>
</table>

Table 2: CTB section stiffness comparison (×10¹¹ N.mm²)

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Test</th>
<th>Formulation</th>
<th>CTB-FEM</th>
<th>RB</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTB (NS:150mm)</td>
<td>4.41</td>
<td>4.65</td>
<td>4.60</td>
<td>3.90</td>
</tr>
<tr>
<td>CTB (NS:100mm)</td>
<td>5.49</td>
<td>4.94</td>
<td>4.80</td>
<td>4.41</td>
</tr>
<tr>
<td>CTB (NS:75mm)</td>
<td>6.11</td>
<td>5.22</td>
<td>4.96</td>
<td>5.49</td>
</tr>
</tbody>
</table>

Figure 6: Stiffness increment–NC relationship

3 STIFFNESS CALCULATION FORMULATION FOR CTB

3.1 FORMULATION INTRODUCTION

It was found that RS occurred in the bended CTB mentioned above, so this kind of CTB, however, cannot be analyzed by the simple equations defining the properties of most built-up composite sections. Because the CTB employ non-rigid nail connection (figure 6) in fastening the flange to the web, there was a slip plane between the two components. A classic equation (1) to analyze this kind of CTB was derived initially by Norris [12] and deeply developed afterwards by Edward [13] and McCutcheon [14] was shown below:

\[
E_{I_{eff}} = E_{I_u} + \frac{K}{1 + \frac{10K}{L'S}} h^2
\]

(1)

\[
K = \frac{(E_A)(E_{A_w})}{E_A + E_{A_w}}
\]

(2)

Where \( E_{I_{eff}} \) equal to composite bending stiffness of the CTB; \( E_{I_u} \) equal to bending stiffness if the web and flange were completely unconnected, usually taken as the stiffness of the web alone; \( L \) equal to distance between gaps in the flange, in this paper \( L=2760 \) mm; \( h \) equal to distance between the centroids of the web and the flange, in this paper \( h=125 \) mm; \( S \) equal to nail connection stiffness per unit length; and \( E_A, E_{A_w} \) equal to axial stiffness of the flange and the web, respectively. From the equation one can see that the strength and stiffness added by composite action in the CTB depend upon the axial stiffness of the flange and the web, the interlayer slip stiffness, and the presence or absence of gaps in the flange. In this paper, \( E_f \) for flange equal to 6000 Mpa, and \( E_w \) for web equal to 9128 Mpa with consideration of the transverse shear in the web. The two values for \( E_f \) and \( E_w \) were based on material property tests.

3.2 COMPARISON ON FORMULATION AND TEST RESULTS

From table 1, it was found that RS varied along the axes of the beam, with the location separated far away from the mid-span point the RS increased. So in the CTB nail connection stiffness varied with different section location as well as the stiffness of the CTB according to Equation (1). In this paper, secant nail connection stiffness based on the averaged RS from location F and I, and G and H (table 1) was adopted to calculate the apparent bending stiffness [8] of the CTB. Nail connection secant stiffness (S) according to figure 6 for CTB with 150 mm-NS, 100 mm-NS, and 75 mm-NS were 7.83 N/mm/mm, 12.35 N/mm/mm, 17.73 N/mm/mm, respectively. Calculation results were listed in table 2, it was found that the equation can predict the stiffness of the CTB with consideration of the different nail connection spacing to some extent.
4 CTB-FEM ANALYSIS

A CTB-FEM was developed to numerically investigate the composite action between the flange and the web in CTB in the ANSYS finite element program (figure 7). In the CTB-FEM, BEAM4 element and SHELL63 element were adopted to model the web and the flange, respectively. And these two-type elements were assumed to elastic, values of MOE were 9128Mpa and 6000Mpa, respectively. Each nail in the CTB was model as a non-linear spring element (COMBIN39) having a horizontal stiffness along the longitudinal of the CTB. The non-linear spring element connected the node of the SHELL63 element to the corresponding end of the rigid link on the BEAM4 element. The rigid links had a length equal to h, and they distributed along the beam with spacing of 150 mm, 100 mm, and 75 mm, which corresponding to the three different NC adopted in CTB. And the line consisted of the ends of the links represented the upper edge of the web, BEAM4 elements were adopted to model these links and were given a very high value of modulus of elasticity as compared to the stiffness of the other elements to avoid the significant deformation of themselves.

![Figure 7: CTB-FEM](image7.png)

![Figure 8: Load-deflection behavior comparison (CTB-FEM)](image8.png)

The loading protocol and setup used in test were adopted in the analysis conducted on the CTB-FEM. The load-deflection behavior (figure 8) and the beam deflection under load of 20kN (figure 9) demonstrated that the obvious composite action existed in CTB in which the non-rigid nail connection was adopted to fasten the flange to the web. And with NS decreased the RS decreasing (figure 10), which resulted in composite action enhanced in the CTB (figure 8 and figure 9). The conclusions obtained based on the CTB-FEM results were same with that of test.

5 DISCUSSIONS

The apparent stiffness results from formulation and CTB-FEM (table 2) were very closely although they were both smaller than that of test. The formulation and CTB-FEM were ideal methods for stiffness calculation which without consideration of the friction between the flange and the web that can reduce the RS and meanwhile enhance the composite action between them as well as other reasons. And another very important reason that made the testing apparent stiffness of the CTB larger than that of the formulation and CTB-FEM was the test setup, the load acted on the flange (figure2...
and figure 3) of the CTB prevented large RS occurring, and meanwhile enhance the interaction between the web an the flange. The two reasons mentioned above could not be taken consideration into the CTB-FEM. It was found the RS were large obtained from CTB-FEM than that of test (table 1). So errors existed in between the test and formulation or CTB-FEM results. And with consideration of the error analysis mentioned above, we recognize that the formulation introduced into this paper can be used to calculate the section stiffness of the CTB as well as the timber floor. And to some extent, the CTB-FEM also had the ability to predict the bending behavior of the CTB, the most important thing was that the method that developed the CTB-FEM could be easily expanded to establish the timber floor finite element model that can be used to do floor vibration simulation induced by pedestrian walking \[7\].

6 CONCLUSIONS

Based on this limited study, the following conclusions are offered:

The test and finite element analysis results demonstrated that obvious composite action existed in the CTB depended on the nail connections that fasten the flange to the web. With NS decreased the composite action between the flange and the web increasing. So that proper contribution results from composite action should be considered in the timber floor. A classical formulation introduced and validated in this paper could be used to timber floor stiffness calculation. Also, a CTB-FEM was developed in the paper, and it was found that the CTB-FEM can reproduce the performance of the CTB. The method that develop the CTB-FEM could be easily expanded to establish the timber floor finite element model that can be used to do floor vibration simulation induced by pedestrian walking.

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HEALTH ASSESSMENT OF HAUFF TIMBER STRUCTURES IN BRAZIL

Carlito Calil Junior¹

ABSTRACT: This article presents the health assessment of Hauff timber structures in Brazil. Large wood structures originated with the engineering company HAUFF, whose production of wood structures contributed in great measure to the technological advance of the industry of construction engineering with wood in Brazil. Based on graphic, photographic and descriptive documentation, this paper offers examples of roof structures and of structures for other purposes built by HAUFF, focusing on structural systems of roofs, bridges, and scaffolding that constitute most of HAUFF’s production.

KEYWORDS: Brazil, historic, timber structures, assessment, Hauff structures

1. HAUFF TIMBER STRUCTURES IN BRAZIL

Erwin Hauff, founder of the company HAUFF, was born in Vienna, Austria and graduated in civil engineering from the Technical University of Munich in 1920. At the end of World War I, Mr. Hauff moved to Brazil, where he became fascinated in studies of Brazilian forest species upon observing the physical characteristics of their wood. He collected samples of a wide variety of species, observing their drying behavior, their defects such as cracking and warping, and their workability. All these observations were based on empirical trials[1]. Hauff built several types of roofing structures for warehouses, deposits, bridges, soffit scaffolding and other works in the city of São Paulo in the 1920s, until, in 1928, he founded the civil engineering company HAUFF. As head of this company, he designed and built wood truss structures with dowelled connectors, which earned him national and international acclaim. His international prestige came through the Technical University of Munich, which conferred on him the honorary degree of lecturing professor for his contribution to the area of wood structures constructed in Brazil. Erwin Hauff’s experience in the field of wood structures and his knowledge of the technology of this material contributed to the formulation of the Brazilian NB-11/51 standard for timber design. The proposal of the “Hauff System” was characterized by its use of joint covers, common dovetail joints and wooden dowels used for the connection of skewed beams, forming the nodes of the structural elements. Another relevant characteristic of this system was the use of the truss system composed of simple girder sections or of multiple elements (of simple sections) nailed together to form a girder of the desired length to resist active loads. This technique was widely utilized in structural designs of road bridges, soffit scaffolding for reinforced concrete road and rail bridges, towers, scaffolding framework, falsework, silos and roofing structures in general (Figure 1a e 1b).

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HAUFF’s success in the 1920s, 30s and 40s was due, among other things, to the greater availability of skilled labor which was abundant in those days as a result of foreign immigration, which brought to Brazil a large contingent of individuals of medium level very experienced and highly qualified in carpentry trades, as well as in other general construction tasks. Also, at times, HAUFF itself brought over several European technicians to train their workers, who were required to be qualified at a technical or specialized level in one area of carpentry, since the Brazilian labor market was going through a crisis, particularly in the late 40s and 50s. These foreign professionals were, in large part, responsible for training many excellent Brazilian carpenters.

1.1. The phases of “Hauff” Wood Structures in Brazil

Of Hauff company’s three phases, the first stood out from the others for the technological innovation in wood structures which the company introduced into construction engineering. These structures included road bridges, soffit scaffolding, framework scaffolding, antenna towers, falsework, roofing frameworks in general, and silos. Among the wood structures constructed by HAUFF to serve as road bridges, the Guarulhos Bridge deserves special mention because it was considered the bridge with the largest free span in South America in those days, spanning 52 meters between supports, with a lower deck of 3.30 meters width (Figure 2).

Another significant example of a wooden bridge built in the same period is the three-hinged arch structure constructed over the Tietê River in the city of São Paulo. This bridge had a free span of 38 meters and a total length of 48 meters. Like the Guarulhos Bridge, this one was also treated with a carbolineum-type preservation product and was supported upon a reinforced concrete foundation. Figure 3 offers a partial view of this bridge.

Another sector of construction engineering where HAUFF’s framework stood out was the subsector of buildings, due to the rationality of construction and the reuse which it allowed during the construction work. One of the most significant examples is the framework used in the construction of the São Paulo Municipal Market, which was built in the period of E. HAUFF & Cia. These frameworks consisted of two-hinged rigid frames with 17-meter spans, built for one third of the length of the work, and moved to the intermediate and last point (Figure 4).
Structures built according to the “Hauff System”, which was characterized by the presence of carved dowel connections and trusses, predominated in the first period of wood roofing projects and works. This initial period, in turn, was also divided into two phases. This division was marked by the type of tiles used in roofing, and the first phase, which occurred between 1925 and 1937, was characterized by the use of French-type fired clay tile roofing (Figure 5).

The second phase, which began in 1937, was characterized by the use of fiber cement corrugated roofing panels. The use French tiles required a roof framework composed of slats, rafters and purlins. Structural elements such as arches, rigid frames, trusses, etc. consisted of bars with more robust sections, since this type of roofing was heavier than fiber cement panels. Another important aspect of French tiles is the roof bracing system, which is simpler than the system used for frameworks for supporting fiber cement roofing panels, because clay tiles are not fixed to the roof web and therefore do not transfer the effect of wind suction to the structural elements (Figures 6 and 7).
During this period, it was also common to use arch structures with two or three hinges and wood or steel ties. The combination of the shape of the arch and the mechanical characteristics of wood in resisting tensile and compressive loads parallel to the fibers resulted in a structural solution that could cope with large spans with the rational use of wood. This structural solution was widely used by HAUSS for twenty to thirty meter spans, when the arches were braced and supported on brick walls, columns and reinforced concrete beams, or on wooden columns (Figure 9).

The nondestructive techniques used in this work were evaluation of visual characteristics related to structural problems, that is, absence of bracing, buckling members, deterioration due wood humidity and deterioration due fungi and insects attacks. Figures 1 to 4 shows this type of structural problems.

2. HEALTH ASSESSMENT OF ACTUAL HAUSS TIMBER STRUCTURES

Nowadays we still have in use important Hauff timber structures and its assessment is very important. Nondestructive evaluation is the science of identifying the physical and mechanical properties of materials without altering its end-use capabilities and then using this information to make decisions regarding appropriate applications. Nondestructive evaluation (NDE) technologies have contributed significantly toward detect structural problems. Table 1 show types of tests and techniques evaluation categorized as nondestructive [2].

![Figure 9 – Two-hinged arch truss roof, 70 m – Source: César, S. F.](image1)

![Figure 10 – Vertical sawn lumber nailed arch 45 m – moisture migration from roofs.](image2)

![Figure 11– Lamellar roof structure, 25 m – moisture migration from tiles](image3)

![Figure 12 – Plywood arch 50 m – support deterioration from termites](image4)

**Table 1 – Nondestructive techniques (PELLERIN & ROSS, 2002)**
Most of the types of deterioration has been analyzed using visual analysis and also mechanical equipments like resistograph techniques. Detailed evaluation and technical report has been carried out with the support of the São Paulo State Research Support Foundation, Brazil, in order to make the rehabilitation of these timber structures in Sao Paulo state.

3. CONCLUDING REMARKS

HAUFF can be considered a company that introduced wood structure technologies in the country and which was in large part responsible for training artisans linked to the production of wooden structures, i.e., master carpenters, draftsmen and designers, and for enriching the body of technical knowledge if many engineers who participated in this production. In this way the assessment and rehabilitation of Hauff timber structures must be an important subject for the future of timber structures in Brazil.

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Osaki City, Miyagi Prefecture seismic damage in East Japan earthquake

Hiroshi Watanabe

ABSTRACT: In this paper, Osaki, Miyagi earthquake damage for East Japan, which aims to identify and compare the results of a study by vibration analysis and field survey. As a result, the city compared to the maximum depth measured Tsukidate, Osaki in poor soil, the seismic waves are amplified, it was found that was severe to wooden buildings waves. In addition, a lot of ground and water management problems due to construction, the building is confirmed by a number of harmful ants was inferred that the larger the damage.

KEYWORDS: earthquakes, vibration analysis, traditional Japanese buildings

1 INTRODUCTION

In the East Japan earthquake, has been focused mainly Rikuzentakata Tsunami, more than a few have been reported seismic damage. East Japan earthquake has been confirmed in a large movement of the plates around the source region, even away from the epicenter, which also features a relatively large tremors were observed.

Our laboratory surveyed Osaki, Miyagi Prefecture, Sendai, Ishinomaki, Kesennuma, Iwate Prefecture Ichinoseki performed at Rikuzentakata for from April 11 to April 13.

This paper describes that the seismic damage in Osaki. Osaki City has been confirmed damages of liquefaction, the slope of the RC building, several houses and traditional stock house were collapsed.

In the emergency safety check is carried out by Osaki government, 2610 buildings, 338 Of the "danger (red)," The 396 "Needs Attention (yellow)" status (as of 2011.4.15), respectively.

2 Analysis of the seismic

East Japan earthquake was caused by the collapse of the Pacific Ocean plate boundaries more. The collapse of this large, increasingly active movement of active faults in Japan as a whole, raised several times aftershocks over M6.

As a component of the seismic waves are almost short-period. Therefore pure vibration damage were few, compared to past earthquakes, such as the Kobe earthquake.

On the other hand, in areas with soft ground extends the period of site amplification, the earthquake caused damage in many areas. Osaki soft ground is also studied past paper has been found that the subsidence had occurred before the earthquake. Also, heavily damaged buildings located waterways and paddy, revealed by aerial photographs.

3. The land history of Osaki-City

The Osaki city located north part of Sendai plane, which named Osaki plane. Osaki plane had formed since several Eai-rivers flood. One of the river, which stream center of Osaki city Furukawa area, named Odae river, "Odae" means dry up, namely once Eai-river (Main stream) had dried up. Therefore, Furukawa area has a lot of canal, and soil is much soft.

Miyagi Prefecture is investigating the land subsidence in the prefecture from the past, in which are also referred to the land subsidence in Osaki. Reported that in that land subsidence district Furukawa, Osaki City is remarkable.

Looking at the amount of land subsidence around the point of Furukawa, Osaki, Akira Furukawa largest in the district, the amount of subsidence of 30 years, has reached 25.6cm.

The figure below is a diagram representing the amount of land subsidence in the contour of the district of the city
center Furukawa, Osaki.

In addition, Osaki, the land reclamation was a paddy field in the past, and residential and commercial land in recent years. Although you are, this can also

Osaki city 2001, (green area is paddy)

Osaki city 2011, (decrease paddy area, and turn the residential, commercial area)

Soft ground is, it is said that in general, and to amplify the seismic waves. Seismic wave seen in the earthquake, to record observations at JMA, and site amplification has been observed.

In a hospital near the JMA, the crack in the ground of about 20cm height was observed.

Therefore, velocity response spectrum of JMA-Furukawa, were observed with a period of severe wooden houses more than 1-2 second. In addition, the maximum acceleration is larger north-south direction. Coincide with direction tombstone fall and the residual deformation of the building.

3 Vibration Analysis by wallstat

To clarify the earthquake damage to buildings Osaki, and analyzed using wallstat ver1.11.3 developed by Building Research Institute. Based on sample data, and produce two models, used in the analysis.

The first model is a relatively new model for wooden house construction, the second model is a model of old wooden houses of traditional construction. The difference between the two is the presence of brace. Input wave data which observed across the nation.

Osaka city 2001, (Red capital is JMA Furukawa mikkamachi point, blue one is K-net Furukawa)

Osaka city 2011, (decrease paddy area, and turn the residential, commercial area)
Results of vibration analysis of seismic waves, seismic waves of K-net Tsukidate, M7 observed, both models did little damage. According to the Survey by NILIM Tsuchimoto, there was little damage to buildings vibrate. In addition, since that was the observation equipment placed on the cliff, whether was not actually observed a larger earthquake.

In the analysis of seismic wave input Furukawa, the following result were obtained. The model which has fewer seismic resistance elements gets more serious damage.

In recently research, the old traditional construction building has long unique cycle, compared with new shaft assembly construction building. Overlap in conditional of geotechnical characteristics and short-period seismic waves, led to more serious damage than traditional construction. The following sections describe the damage to individual buildings.

4 Building damage

Building damage as a feature of the district Furukawa, Osaki City, there is termite damage. Cause is considered to be mainly about two.

One is that it is considered in relation to the ground, the soil of the district Furukawa, Osaki City, that contains a lot of moisture. As is also the place name, the district there are many waterways Furukawa, it is still extant. This means that, at the same time that forms the quaint space to Furukawa, indicating that it is soft ground.

The authors, in the schoolyard of the elementary school Furukawa, and to discover the traces boiling is evidence that the occurrence of liquefaction, and other have observed such as the slope of the building of over-storey 10 reinforced concrete that is near the station Furukawa, JR. These will be mentioned as a factor that produced the surrounding environment in addition to the increase in the damage of the building by the ground, which is easy to bring powerful enemy for wooden, termite damage.

Second, age is the age of the building. Age seems to be the construction of a building seems to be greatly damaged buildings Furukawa, Osaki City, district, and the previous one in 1970 either. These buildings, the quality of construction is low, and appropriate treatment of water has not been made. As a result, the invasion of termites and was seen to the structural part, the decay of wood-rotting fungi by structural parts. Of course, the historic buildings in the district Furukawa, from the soft ground that stood over the long, because it was affected by these predators, damage to buildings has become greater. In addition, in the old building's architecture is often heavy building materials are used in the roof, were also seen the collapse of the barn of cement tile.

4.1 Damage to public buildings and commercial buildings

Damage to commercial buildings, was focused on building a wooden mortar. Most damage is flaking of the wall. In many buildings damaged walls, termite damage was found inside. Termite damage has been observed in almost all destroyed buildings of wooden buildings, we surveyed. In addition, these commercial buildings are likely to have been built not long after World War II period. That is, the time was a nationwide shortage of building materials.

4.2 Traditional construction methods for building storehouses and damage

Storehouses which built in the Edo Period are located old castle town nearby Zuisenji-temple. There are a lot narrow canal, soft ground.
The damage of Traditional buildings (representive storehouses) can be classified by the presence of reinforcement.

The main hall, renovation was done, like Insulation is installed, only damage was a little chipped walls, Kannon temple and gate showed serious damage, such as fall down.

Similarly, more light damages the storehouse which has been made seismic reinforcement). On the other hand, storehouse (reinforced not yet) wall were much collapsed.

5 CONCLUSIONS

Results of this study reconfirmed that most important element in building is ground. In particular, the traditional architecture of the old construction methods, seismic resistance element is less, tends to be more serious damage by vibration.

Is omitted in this report, we have investigated Ishinomaki City (heavily damaged by the tsunami), except coastal tsunami damage, good hill ground vibration damage almost no. It was severely damaged by Osaki’s, which also includes large commercial building frontage.

On the other hand, In Osaki city, soft ground caused the serious damage to buildings, liquefaction damage was observed in RC buildings such heavy compared to wood density. In related news, it was confirmed that damage was a lot of ants in a wooden building damage. Where the ground is poor, many of which have problems of water. Wood is a great enemy to Wed, which reaffirmed that as soon as maintenance is essential for proper design and construction, and preservative treatment.

Incidentally, when writing this paper, we are grateful to Dr Nakagawa, Building Research Institute and Analysis provides you teach analysis software.

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MECHANICAL MODEL OF MUD WALL BASED ON SOIL MECHANICS

Naoki Utsunomiya 1, Mitsuhiro Miyamoto 2, Minoru Yamanaka 3 and Manabu Matsushima 4

ABSTRACT: Mud walls consist of wooden frame, lattice substratum and viscous soil mixed with straw. Mud walls resist the lateral forces during an earthquake by compressive and shear strength of wall clay. So the seismic performance of mud walls is influenced by the mechanical characteristics of wall clay. In this paper, we propose the mechanical model of mud walls based on soil mechanics to estimate the relationship between lateral loading and deformation. We performed full-scale static lateral loading tests of mud walls to compare the estimation results with the test results. We also performed unconfined compression tests to grasp mechanical characteristics of wall clay. From the results of these tests and calculations, it is found that the relationship between lateral loading and deformation of specimens can be estimated with the mechanical model proposed in this paper.

KEYWORDS: Mud wall, Soil mechanics, Cohesion, Angle of internal friction, Static loading test

1 INTRODUCTION

Mud walls consist of wooden frame, lattice substratum made of bamboo etc. and viscous soil mixed with straw shown in Figure 1. Soil is gathered near the region where it is used. Straw is compounded with soil based on experience and intuition of plasterers. Mud walls resist the lateral forces during an earthquake mainly by shear strength of wall clay. So the seismic performance of mud walls is decided by the mechanical characteristics of wall clay such as cohesion and angle of internal friction. The mechanical characteristics of wall clay are greatly influenced by the region or the mixed amount of straw. Full-scale or portion-wall tests, which take much money and time, are needed to grasp the relationship between lateral loading and deformation of mud walls which varies according to the region in Japan. If the relationship between lateral loading and deformation of mud walls can be estimated from the results of element tests, such as unconfined compression tests, based on soil mechanics, it can be easy to evaluate the seismic performance of mud walls without full-scale or portion-wall tests.

The objective of this paper is to estimate the relationship between lateral loading and deformation of mud walls with initial failure in shear from the results of unconfined compression tests based on soil mechanics. In addition, compared with full-scale test results, the accuracy of estimation results is examined.

2 MODELLING OF RELATIONSHIP BETWEEN LATERAL LOADING AND DEFORMATION OF MUD WALLS

2.1 FLOW FOR ESTIMATION

We have performed experimental study about wall clay [1, 2]. Shear strength of wall clay mixed with straw was grasped based on soil mechanics. In addition, new circle test pieces for unconfined compression tests of wall clay were proposed. With these test pieces, the mechanical characteristics of wall clay are estimated more accurately than traditional test pieces. In this study, the mechanical characteristics of wall clay are estimated based on the results of unconfined compression tests shown in Figure 2(a). The strength of mud walls with initial failure in shear is estimated considering the fracture mode of wall clay shown in Figure 2(b). The relationship between lateral loading and

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deformation of mud walls is estimated from the results of unconfined compression tests shown in Figure 2(c).

2.2 ESTIMATION OF STRENGTH

Wall clay is restricted by wooden frames when mud walls are loaded to the lateral direction. The fracture plane in shear is found when wall clay is stressed up to the allowable shear stress. The shear strength of wall clay \( \tau \) is calculated from cohesion \( c \), vertical stress \( \sigma \) and angle of internal friction \( \phi \) based on mohr-coulomb failure criterion expressed in Equation (1).

\[
\tau = c + \sigma \tan \phi \tag{1}
\]

In this paper, we use the circle test pieces of wall clay shown in Figure 2(a). The dimension of test pieces is 125mm in diameter and 250mm in height. From the results of unconfined compression tests, cohesion \( c \) and angle of internal friction \( \phi \) of wall clay is estimated. We take the picture of fracture plane in shear. From these pictures, the angle of fracture plane in shear \( \alpha \) is estimated. The angle of internal friction \( \phi \) is expressed in Equation (2) based on mohr-coulomb failure criterion.

\[
\phi = 2\alpha - \frac{\pi}{2} \tag{2}
\]

The cohesion \( c \) is expressed in Equation (3) based on mohr-coulomb failure criterion because test pieces are not subjected to the lateral pressures.

\[
c = \frac{\sigma_{\text{max}} \left( \sec \phi - \tan \phi \right)}{2} \tag{3}
\]

Where, maximum compressive strength is \( \sigma_{\text{max}} \).

Mud walls loaded to the lateral direction are restricted by wooden frames. The lateral load is transferred through the compressive strut. The crack of mud walls with initial failure in shear is supposed to be occurred at A-B-C-D surface shown in Figure 2(b). Wall clay is collapsed in shear and the strength of mud walls is ultimate when the compressive strut is unable to transfer loads. So it is supposed that wall clay is collapsed in shear when mud walls are stressed up to the allowable shear stress at B-C surface. Wall clay resists by Nakamura and Onaoshi soil when mud walls are subjected to the lateral loads [3]. So the real thickness of mud walls \( t_w \) resisting to the lateral loads is regarded except lattice substratum including the thickness of Nuki.

The mechanical model in this paper is shown in Figure 3. The distance of B-C surface is \( L_w = H - L \) and the width of compressive strut is \( L_w = \frac{L_w}{2} \cos \theta \). The force along the compressive strut is \( P_v = P \cos \theta \) with lateral load \( P \). The compressive stress at B-C surface \( \sigma_c \) and B-C surface stress \( \sigma_c' \) are expressed as follows.

\[
\sigma_c = \frac{P}{L_w t_w} = \frac{P \cos \theta}{L_w t_w \cos \theta} = \frac{P}{L_w t_w} \tag{4}
\]

\[
\sigma_c' = \sigma_c \cos \theta = \frac{P \cos \theta}{L_w - t_w} \tag{5}
\]

The maximum lateral load \( P_{\text{max}} \) is expressed in Equation (6) with real thickness of mud walls \( t_w \) and length of shear resistance \( L_w \). Equation (6) is expressed in Equation (7) with Equation (1) and (5).

\[
P_{\text{max}} = \frac{L_w t_w \tau}{1 - \cos \theta \tan \phi} \tag{7}
\]

Where, \( H - L > 0 \) and \( \theta = \pi/4 \).

It is found that the maximum lateral load \( P_{\text{max}} \) increases as the real thickness of mud walls \( t_w \), cohesion \( c \) and angle of internal friction \( \phi \) increase.

The relationship between lateral loading and deformation of mud walls in this study is supposed to be tri-linear shown in Figure 2(c). As shown in Figure 4, the ratio of maximum lateral load \( P_{\text{max}} \) to yield load \( P_y \) is almost 0.64 from the test results in the past [4-7]. So yield load \( P_y \) is expressed in Equation (8).

\[
P_y = 0.64 P_{\text{max}} \tag{8}
\]

2.3 ESTIMATION OF DEFORMATION

Shear modulus of wall clay \( G \) is expressed in Equation (9) with elastic modulus of wall clay \( E \) and Poisson’s ratio \( \nu = 0.2 \). Where, \( E \) is the inclination of straight line which passes through the origin and half of maximum compressive strength. Shear stress of wall clay in the elastic range of mud walls \( \tau \) is expressed in Equation (10) with shear deformation angle \( \gamma \).

\[
G = \frac{E}{2(1+\nu)} \tag{9}
\]

\[
\tau = \gamma G \tag{10}
\]
The relationship between lateral load $P$ and shear stress of wall clay $\tau_2$ is expressed in Equation (11) with the real thickness of mud walls $t_w$ and the inside dimension of mud walls $L$.

$$\tau_2 = \frac{P}{t_w L}$$  \hspace{1cm} (11)

Yield deformation angle $\gamma$ is expressed in Equation (12) with the relationship $\tau_1 = \tau_2$. Deformation angle at maximum strength $\gamma_{\text{max}}$ is expressed in Equation (13) with second stiffness $K'$ which is the inclination of straight line from yield load $P_y$ to maximum lateral load $P_{\text{max}}$. As shown in Figure 4, the ratio of initial stiffness $K$ to second stiffness $K'$ is almost 0.38 from the test results in the past [4-7].

$$\gamma = \frac{2P_y (1+\nu)}{t_w LE}$$  \hspace{1cm} (12)

$$\gamma_{\text{max}} = \frac{(P_{\text{max}} - P_y)}{K'} + \gamma$$  \hspace{1cm} (13)

Shear cracks at A-B-C-D surface of mud walls shown in Figure 1(b) are expanded and combined as mud walls deform. So the large cracks are found on mud walls after the maximum strength.

The shear deformation of circle test pieces in unconfined compression tests are shown in Figure 1(a). The relationship between vertical strain of circle test pieces $\delta$ and shear deformation angle of mud walls $\gamma$ is expressed in Equation (14) with the diameter of circle test pieces $D$.

$$\gamma = \frac{\delta}{D}$$  \hspace{1cm} (14)

The relationship between vertical strain of circle test pieces $\delta$ and shear deformation angle of mud walls $\gamma$ after the maximum strength is expressed in Equation (15) with the vertical strain of circle test pieces at maximum load $\delta_{\text{max}}$ and Equation (13).

$$\gamma = \frac{\delta - \delta_{\text{max}}}{D} + \gamma_{\text{max}}$$  \hspace{1cm} (15)

Mud walls keep the maximum strength $P_{\text{max}}$ until shear deformation angle $\gamma$ calculated from the vertical strain of circle test pieces $\delta$ with Equation (15), where the load is 85% of maximum load shown in Figure 1(c).

The strength of mud walls after the maximum strength $P_r$ is calculated with the shear deformation angle of mud walls $\gamma_r$ and the vertical strain $\delta_r$ and the compressive stress $\sigma_r$ of circle test pieces after the maximum strength. As shown Figure 5, residual cohesion $c'$ is decrease as vertical stress and mohr’s stress circle are small. Residual cohesion $c'$ is expressed in Equation (16). The strength of mud walls after the maximum strength $P_r$ is expressed in Equation (17) with Equation (16).

$$c' = \frac{\sigma_r (\sec \phi - \tan \phi)}{2}$$  \hspace{1cm} (16)

$$P_r = \frac{c' t_w (H - L)}{1 - \cos \theta \tan \phi}$$  \hspace{1cm} (17)

### 3 STATIC LATERAL LOADING TEST

In this chapter, we perform the static lateral loading tests of mud walls to grasp the fracture mode and the relationship between lateral loading and deformation.

#### 3.1 ELEMENT TEST OF WALL CLAY

We perform unconfined compression tests of Arakabe, Urakaeshi, Oonaoshi and Nakamuri soil mixed with straw which are used for the specimen. We apply loads at the rate of 1% compressive strain a minute. After the tests, we calculate cohesion $c$, angle of internal friction $\phi$ and elastic modulus $E$ based on the evaluation method [2].

Table 1 shows compressive strength, elastic modulus, cohesion, angle of internal friction and shear strength of wall clay by element tests, respectively. Reference [8] provides that the maximum compressive strength of wall clay is more than 0.30N/mm$^2$ in Arakabe and 0.55N/mm$^2$ in Nakamuri. Compared with this standard, the compressive strength of wall clay in the tests is 1.8 times in Arakabe and 1.8 times in Nakamuri.

#### 3.2 OUTLINE OF FULL-SCALE TEST

The length of specimen is 1820mm. In the tests, the effect of moment resistance at each connection is removed to grasp only the effect of wall clay shown in Figure 6. The construction method of mud walls shown in Figure 7 is as follows. Arakabe soil is plastered after
Table 1: Results of element test

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Amount of mixed straw (kg/100l)</th>
<th>Compressive strength $\sigma_{max}$ (N/mm²)</th>
<th>Elastic modulus $E_{50}$ (N/mm²)</th>
<th>Cohesion $c$ (N/mm²)</th>
<th>Angle of internal friction $\phi$ (°)</th>
<th>Shear strength $s$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arakabe</td>
<td>1.8</td>
<td>0.58</td>
<td>90.4</td>
<td>0.24</td>
<td>12</td>
<td>0.28</td>
</tr>
<tr>
<td>Urakaeshi/Onaoshi</td>
<td>2.0</td>
<td>0.72</td>
<td>126.6</td>
<td>0.32</td>
<td>7</td>
<td>0.36</td>
</tr>
<tr>
<td>Nakanuri</td>
<td>1.1</td>
<td>0.97</td>
<td>251.7</td>
<td>0.37</td>
<td>16</td>
<td>0.46</td>
</tr>
</tbody>
</table>

3.3 LOADING AND MEASURING INSTRUMENT

Figure 8 shows the outline of loading instrument. The ground sill of specimen is fixed with anchor bolts. The beam of specimen is subjected to lateral loads through the servo actuator by tie-rod system. Specimen is subjected to cyclic lateral loads, gradually increasing the real shear deformation angle $\gamma_0$ symmetrically from 1/600, 1/450, 1/300, 1/200, 1/150, 1/100, 1/75 to 1/50rad. The number of cyclic loading is three. Finally, specimen is loaded to 1/10rad at one end.

The lateral loads subjected to specimen are measured with load cell which is attached at the edge of servo actuator. Displacement sensor H1 is attached on beam and H2 on ground sill to measure the lateral displacement. Displacement sensor V3 and V4 are attached at the bottom of column to measure the vertical displacement.

3.4 FRACTURE MODE

Figure 9 shows the failure process of mud walls. The compressive failure at the contact surface between wooden frame and wall clay is occurred at 1/450rad. Cracks are developed near vertical Nuki at 1/150rad but the lateral load is not decrease. About 5mm gap on Urakaeshi side at the contact surface between wooden frame and wall clay by drying shrinkage is lost at 1/150. The small shear cracks on Nakanuri side are occurred at 1/100rad. The shear cracks are expanded and combined as mud walls deform. The cracks along lateral Nuki on Nakanuri side and along Mawatashi bamboo and vertical Nuki on Urakaeshi side are occurred at 1/75rad. The fracture mode shown in Figure 2(b) is found at 1/50rad when mud walls are the maximum strength. The fracture mode in the test is same as the estimation formula.

4 COMPARISON OF TEST AND ESTIMATION RESULTS

Table 2 shows the parameter to estimate the relationship between lateral loading and deformation of mud walls. The mechanical characteristics of Nakanuri soil is used to calculate easily. The thickness of Nakanuri and Onaoshi soil is the lower limit of estimation and that of Nakanuri, Onaoshi and Urakaeshi soil is the upper limit of estimation, respectively. Compared with the estimation results, the initial stiffness of test results is 0.24 times to the upper limit and 0.37 times to the lower limit because the gap at contact surface between wooden frame and wall clay by drying shrinkage is occurred. So the elastic modulus of wall clay is multiplied by the correction coefficient to calculate the yield deformation angle.
Table 2: Parameters

<table>
<thead>
<tr>
<th>Wall clay</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion $c$ [N/mm²]</td>
<td>Length $L$ [mm]</td>
</tr>
<tr>
<td>0.37</td>
<td>1715</td>
</tr>
<tr>
<td>Angle of internal friction $\phi$ [°]</td>
<td>Height $H$ [mm]</td>
</tr>
<tr>
<td>16</td>
<td>2625</td>
</tr>
<tr>
<td>Elastic modulus $E$ [N/mm²]</td>
<td>Thickness $t_w$ [mm]</td>
</tr>
<tr>
<td>252</td>
<td>Upper limit</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu$</td>
<td>Lower limit</td>
</tr>
<tr>
<td>0.2</td>
<td>36</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

In this paper, the mechanical characteristics of wall clay are estimated from the results of element tests based on soil mechanics. The relationship between lateral loading and deformation of mud walls with initial failure in shear is estimated with the mechanical characteristics of Nakanuri soil. Compared with full-scale test results, the accuracy of estimation results is examined. As a result, it is found that the envelope of test results is almost same as that of estimation results so the proposed formula is effective. It is desirable that the thickness of mud walls is considered to be the thickness of Nakanuri and Onaoshi soil considering the safety when the proposed formula is used.

REFERENCES


PERFORMANCE OF WOOD PORTAL FRAME SYSTEMS AS ALTERNATIVE BRACING SYSTEMS IN LIGHT WOOD-FRAME BUILDINGS

Chun Ni1, Mohammad Mohammad2, Abdullah Al Mamun3, Ghasan Doudak4

ABSTRACT: In this paper, results are presented from a testing program focused on evaluating the performance of portal frame systems. A total of nine full-scale portal frame assemblies with six different configurations were tested under monotonic and reversed cyclic loading. The portal frames were 3.66 m in length and 2.44 m in height, with 406 mm wall segment at each end of the portal frame. From the test results of full-scale portal frame, it was observed that the corner joint between the header and narrow braced wall segment dominates the lateral load carrying capacity and ultimate displacement of the portal frame. The installation of metal straps considerably increases the lateral load carrying capacity of the portal frame assemblies. Straps placed directly on the lumber framing showed increased resistance compared to those installed on the OSB. Portal frames with hold-downs have greater lateral load carrying capacity and ultimate displacement than those without hold-downs.

KEYWORDS: Portal frame, braced wall, lateral load resistance, monotonic loading, cyclic loading

1 INTRODUCTION

The lateral load resistance of light wood frame buildings is generally provided by braced walls sheathed with panels or diagonal lumber boards. To ensure that buildings have adequate lateral load carrying capacity to resist moderate-to-high wind and seismic loads, prescriptive requirements on the minimum length of braced walls, along with maximum spacing between braced walls, have been placed in Part 9 of the 2010 National Building Code of Canada (NBCC) [1]. Acceptable materials, fastening and framing details constituting a braced wall are also specified in the code. Although most wood-frame buildings are able to meet the minimum braced wall requirements, there are situations where the required length of braced walls may not be met due to space limitations imposed by architectural requirements. For example, one common feature is the large opening required for multi-car garages. Alternative bracing systems which do not limit open space need to be developed to provide equivalent lateral load resistance to the minimum braced wall requirements in Part 9 of 2010 NBCC.

The wood portal frame system has been identified by engineers and builders as one of the alternative bracing systems that can meet the lateral load demand. This system was initially developed by APA - The Engineered Wood Association in the early 2000s. Full-scale portal frame specimens were tested by APA to show that portal frame has comparable performance to existing bracing requirements stipulated in the US International Residential Code (IRC). More than 25 full-scale cyclic tests in two phases confirmed that the 6:1 height-to-width ratio portal frame system performed approximately equal to or better than the 4:1 height-to-width ratio wall segment permitted in IRC [2,3]. The test results formed the basis for the acceptance of portal frame system in IRC since 2006.

There is a need to study and evaluate the performance of wood portal frame systems with different construction details and explore potential improvements to demonstrate that light wood frame structures with portal frames has the same level of performance as bracing walls with traditional designs. In this study, results are presented from a testing program focused on evaluating the performance of portal frame systems with different corner details and boundary conditions. Full-scale portal frame assemblies were tested under monotonic and reversed cyclic loading. The effect of variables such as metal strap type and location, sheathing placement and hold-downs on the performance of portal frame were evaluated.

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2 PORTAL FRAME TESTS

2.1 SPECIMEN DETAILS

The test matrix of the portal frames is given in Table 1. The corner details of the tested portal frame are shown in Figure 1. All the portal frames are 3.66 m in length and 2.44 m in height, with a 406 mm wall segment at each end of the portal frame. The wall framing was constructed with 38 mm x 89 mm NLGA No.2 and better Spruce-Pine-Fir lumber. The average moisture content of the lumber was 13% at the time of fabrication and testing and the average specific gravity of the lumber was 0.43. Except specimen No. 8 in which 38 mm x 286 mm (1.5” x 11.25”) No.2 and better Spruce-Pine-Fir lumber was used as header, the headers of the other walls were built up with 45 mm x 302 mm (1.75” x 11.875”) 1.5E laminated strand lumber (LSL). CSA 0325 oriented strand board (OSB), with a thickness of 12.7 mm (0.5”) and a span rating of 2R32, was used as sheathing panels.

Table 1: Test matrix of portal frames

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>Hold-down Connection details at corner</th>
<th>Load protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No</td>
<td>Ramp &amp; cyclic</td>
</tr>
<tr>
<td>2</td>
<td>HTT 16 C1</td>
<td>Ramp</td>
</tr>
<tr>
<td>3</td>
<td>HTT 16 C2</td>
<td>Ramp &amp; cyclic</td>
</tr>
<tr>
<td>4</td>
<td>HTT 16 C2</td>
<td>Cyclic</td>
</tr>
<tr>
<td>5</td>
<td>HTT 16 C3</td>
<td>Cyclic</td>
</tr>
<tr>
<td>6</td>
<td>HTT 16 C4</td>
<td>Cyclic</td>
</tr>
<tr>
<td>7</td>
<td>No</td>
<td>Cyclic</td>
</tr>
<tr>
<td>8</td>
<td>Steel rods C5</td>
<td>Cyclic</td>
</tr>
<tr>
<td>9</td>
<td>HTT 16 C6</td>
<td>Cyclic</td>
</tr>
</tbody>
</table>

Figure 1: Corner details of portal frames

The 8d common nails (3.3 mm in diameter and 63.5 mm in length) were used to attach sheathings to framing members. The following nailing schedule is used: a) one row of nails spaced at 75 mm on centre on top and bottom plates, b) two rows of nails spaced at 75 mm on centre on double end-studs, and c) grid pattern of nails spaced at 75 mm on centre on header.

For specimen No. 1 to No. 4, the top plate and the header were connected with two rows of 16d nails (4.2 mm in diameter and 89 mm in length) at 75 mm on centre. The end stud and the end of header were connected with two rows of the same type of nails at
For specimen No. 5 to No. 9, screws (5 mm in diameter and 90 mm in length) were used to connect the top plate and the header as well as the end stud and the end of header.

For specimen without hold-downs (No. 1 and No. 7), 12.7 mm diameter anchor bolts were used to fasten the bottom plate to the test frame. The anchor bolts were placed in the same locations as the bolts to connect the hold-down devices to the test frame. Where hold-downs were used, the bolts used to connect hold-down devices to the test frame were also used as anchor bolts to resist the shear force of the portal frame.

Two types of hold-down devices were used: a) Simpson Strong Tie HTT16 which is attached to end studs with 16-16d sinker nails, b) 12.7 mm diameter continuous steel rods. For specimen No. 2, No. 4, No. 5 and No. 9, Simpson Strong Tie HTT16’s were installed at the ends of portal frame (no hold-downs at the opening). For specimen No. 3 and No. 6, Simpson Strong Tie HTT16’s were installed at the ends of portal frame and around opening. For specimen No. 8, steel rods were installed at the ends of portal frame and around opening.

A Simpson Strong Tie LSTA 21 (1000 lb capacity) was used to connect the header and end studs to provide vertical continuity and moment resistance at the corner of portal frame. Where wall frame is sheathed with panels, metal straps are installed over wall sheathing. In areas where metal straps are installed over wall sheathing, wall sheathing is not fastened to the frame.

2.2 TEST SET-UP

A photo of the test setup is shown in Figure 2. The lateral load was applied through steel spreader bar attached to the top of the specimen. The spreader bar had lateral guides to ensure a steady and consistent unidirectional movement of the specimen. Besides the load and actuator stroke, a string displacement transducer was placed at the top of the specimen to measure the lateral deflection of the assembly. Two displacement transducers were placed at the bottom of the header around the frame corners to measure the relative vertical movements between end studs and the header. Four transducers were used at the end-studs to measure the uplift of studs from the foundation.

2.3 LOAD PROTOCOL

Monotonic and reversed cyclic displacement schedules were used in the test program. For monotonic tests, the displacement rate was 10.2 mm (0.4”) per minute. The reversed cyclic displacement schedule followed the ISO 16670 Standard [4], in which the cyclic protocol consisted of the following reversed cycles: one cycle at each displacement level of 1.25%, 2.5%, 5%, 7.5% and 10% of the reference ultimate displacement, and three cycles at each displacement level of 20%, 40%, 60%, 80%, 100% and 120% of the reference ultimate displacement, as shown in Figure 3. Based on the monotonic tests of specimen No. 1, No. 2 and No. 3, the reference ultimate displacement was taken as 55.8 mm (2.198”) for specimen without hold-downs and 88.9 mm (3.5”) for specimen with hold-downs. A displacement rate of 20.3 mm (0.8”) per second was used for reversed cyclic tests.

3 RESULTS OF PORTAL FRAME TESTS

Analysis of test results was carried out in accordance with the Equivalent Energy Elastic-Plastic (EEEP) curve in ASTM Standard E2126 [5]. A summary of the test results under monotonic and cyclic loading is provided in Tables 2 and 3. For specimen under cyclic loading, the values are the average of positive and negative envelope curves. Figure 4 shows a typical load-displacement curve under cyclic loading. The envelop load-displacement curves of portal frames with panels sheathed on one and both sides are shown in Figures 5 and 6.
The notations in the tables are as follows: \( K_y \) is the initial (yield) stiffness; \( F_y \) is the yield load; \( \Delta_y \) is the yield deflection; \( F_{\text{max}} \) is the maximum load; \( \Delta_{F_{\text{max}}} \) is the deflection at which the maximum load was reached; \( \Delta_u \) is the ultimate deflection in post maximum load region where the load dropped to 80% of the maximum load.

**Table 2: Test results of portal frames under monotonic loading**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( K_y ) [kN/mm]</th>
<th>( F_y ) [kN]</th>
<th>( \Delta_y ) [mm]</th>
<th>( F_{\text{max}} ) [kN]</th>
<th>( \Delta_{F_{\text{max}}} ) [mm]</th>
<th>( \Delta_u ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.45</td>
<td>9.01</td>
<td>20.0</td>
<td>10.93</td>
<td>55.4</td>
<td>55.8</td>
</tr>
<tr>
<td>2</td>
<td>0.38</td>
<td>12.88</td>
<td>33.8</td>
<td>14.77</td>
<td>67.5</td>
<td>80.2</td>
</tr>
<tr>
<td>3</td>
<td>0.62</td>
<td>14.91</td>
<td>24.0</td>
<td>16.37</td>
<td>158.7</td>
<td>163.5</td>
</tr>
</tbody>
</table>

**Table 3: Test results of portal frames under cyclic loading**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( K_y ) [kN/mm]</th>
<th>( F_y ) [kN]</th>
<th>( \Delta_y ) [mm]</th>
<th>( F_{\text{max}} ) [kN]</th>
<th>( \Delta_{F_{\text{max}}} ) [mm]</th>
<th>( \Delta_u ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.42</td>
<td>9.00</td>
<td>21.5</td>
<td>10.27</td>
<td>52.4</td>
<td>71.8</td>
</tr>
<tr>
<td>2</td>
<td>0.56</td>
<td>14.00</td>
<td>24.9</td>
<td>15.74</td>
<td>87.6</td>
<td>124.7</td>
</tr>
<tr>
<td>3</td>
<td>0.54</td>
<td>14.31</td>
<td>26.5</td>
<td>16.54</td>
<td>64.2</td>
<td>90.2</td>
</tr>
<tr>
<td>5</td>
<td>0.63</td>
<td>18.67</td>
<td>29.6</td>
<td>21.37</td>
<td>80.2</td>
<td>140.0</td>
</tr>
<tr>
<td>6</td>
<td>0.68</td>
<td>18.78</td>
<td>27.7</td>
<td>21.21</td>
<td>63.1</td>
<td>121.2</td>
</tr>
<tr>
<td>7</td>
<td>0.61</td>
<td>15.76</td>
<td>25.9</td>
<td>18.04</td>
<td>65.0</td>
<td>82.9</td>
</tr>
<tr>
<td>8</td>
<td>0.51</td>
<td>20.46</td>
<td>39.8</td>
<td>23.24</td>
<td>136.0</td>
<td>153.4</td>
</tr>
<tr>
<td>9</td>
<td>0.99</td>
<td>23.07</td>
<td>23.4</td>
<td>25.84</td>
<td>61.2</td>
<td>71.8</td>
</tr>
</tbody>
</table>

**3.1 FAILURE MODES**

For all the portal frames, dominant failure modes were generally associated with failure in the OSB sheathing in tension at the corner of the portal frame. In all specimens where metal straps were used, failure of the metal strap due to load cycling (i.e., fatigue) was observed, following the failure of the OSB panels. For specimens without hold-downs (No. 1 and No. 7), separation of end studs from bottom plate was observed. In some cases (No. 1, No. 7 and No. 9), panel chip-out, nail withdrawal and bottom plate split were also observed. Photos of failure modes are shown in Figure 7.

a) Panel shearing and metal strap broken

b) Nail withdrawal and sill plate split
4 DISCUSSION OF TEST RESULTS

4.1 PORTAL FRAME WITH PANELS SHEATHED ON ONE SIDE OF FRAMING

For portal frames with panels sheathed on one side of the framing, the maximum lateral load capacity is in the range of 11 to 21 kN, with portal frame without hold-downs (No. 1) having the lowest lateral load capacity and portal frames with corner configuration 3 and 4 (No. 5 and No. 6) having the highest lateral load capacities. For portal frame without hold-down (No. 1), the lateral load carrying capacity is approximately 75% of the capacity of identical frames with hold-down (No. 2). For portal frames with hold-downs, the support contributes to the resistance of bending moment, and as a result the bending moment at the corner is smaller than that of portal frame without hold-downs under the same lateral load. As the lateral load capacity is governed by the bending moment at the corner, higher lateral load capacity is obtained for portal frame with hold-downs. No significant differences were found between specimens No. 2, No. 3 and No. 4 in terms of lateral load resistances. This indicates that hold-downs around opening are not critical and can be omitted. Similarly, two metal straps installed side-by-side do not significantly increase the lateral load resistance of portal frame. However, significant increase is observed when metal straps are installed on both sides of the portal frame. The increase of lateral load capacity may also be due to the installation of metal straps on both ends of the studs (C3 and C4) for specimen No. 5 and No. 6, while for specimen No. 2, No. 3 and No. 4 metal straps were only installed at the inner corners of the specimens (C1 and C4). With metal straps installed on both ends of the studs, the metal straps in both wall segments provide resistance to the bending moment at corners. For metal straps installed only at the inner corners of the specimen, only the metal strap in tension provides resistance to the bending moment at corner.

4.2 PORTAL FRAME WITH PANELS SHEATHED ON BOTH SIDES OF FRAMING

For portal frames with panels sheathed on both sides of the framing, the maximum lateral load capacity is in the range of 18 to 26 kN. For portal frames without hold-downs, the portal frame with sheathing on both sides of the framing has approximately 75% higher lateral load capacity than the portal frame with sheathing on one side of the framing. For portal frame with hold-downs, the lateral load capacities of portal frame with sheathing on both sides of the framing are 20 to 40% higher than the portal frame with sheathing on one side of the framing. Difference performance was noticed for portal frames with steel rod (No. 8) and HTT16 tie-down (No. 9). While the lateral load capacity is slightly lower for specimen No. 8, the displacement at ultimate lateral load is much larger. In fact, specimen No. 8 did not reach the ultimate displacement as the test was stopped due to reaching the actuator’s displacement limit. Unlike portal frames with HTT16 tie-downs, the failure modes in specimen No. 8 were nail withdrawal or break. Panel failure in tension was not observed. Figure 8 shows the failure mode of specimen No. 8.

4.3 COMPARISON WITH BRACED WALLS

Results of braced walls are used to compare the lateral load resistance of portal frames. Table 4 summarises the test results of braced walls under different boundary conditions [6]. The wall framing was constructed with Spruce-Pine-Fir (SPF) 1650f-1.5E 38 × 89 mm machine
stress rated lumber. Canadian softwood plywood (CSP) panels, 9.5 mm thick and 1.22 m × 2.44 m in size, were vertically sheathed to the framing. Power-driven spiral nails, 2.5 mm diameter and 63.5 mm long, were used to attach CSP to the framing. The nails were spaced at 150 mm along the perimeter of the panels and 300 mm elsewhere. Comparison shows that for a portal frame without hold-downs, the lateral load capacity of a portal frame with panels sheathed on one side of the framing is equivalent to a 2.44 m braced wall without hold-downs. As can be seen in Tables 2 and 3 above, the lateral load capacity of a 3.66 m long portal frame with hold-downs can be as high as the lateral load capacity of a 4.88 m braced wall without hold-downs.

**Table 4: Test results of braced walls under monotonic loading**

<table>
<thead>
<tr>
<th>Wall Length</th>
<th>Wall No.</th>
<th>Hold-down</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$\Delta_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.44 m</td>
<td>C1-1 No</td>
<td>8.5</td>
<td>44.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1-2 No</td>
<td>9.5</td>
<td>40.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1-3 No</td>
<td>10.8</td>
<td>39.2</td>
<td></td>
</tr>
<tr>
<td>4.88 m</td>
<td>CR2 Yes</td>
<td>34.6</td>
<td>112.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2-1 No</td>
<td>21.4</td>
<td>66.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2-2 No</td>
<td>26.3</td>
<td>56.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2-3 No</td>
<td>31.0</td>
<td>51.7</td>
<td></td>
</tr>
</tbody>
</table>

5 CONCLUSIONS AND RECOMMENDATIONS

There is a need to develop alternative bracing systems in light frame wood buildings that would allow for the increasing demand for more open space structures. This paper presents results of portal frame tests with different corner details and boundary conditions. Lateral load capacity, initial stiffness and ultimate displacement were determined from tests. From the test results of full-scale portal frame, it is evident that the corner joint between the header and braced wall segment dominates the lateral load carrying capacity and ultimate displacement of the portal frame. The installation of metal straps considerably increases the lateral load carrying capacity of the portal frame assemblies. Straps placed directly on the lumber framing showed increased resistance compared to those installed on the OSB. Portal frames with hold-downs have greater lateral load carrying capacity and ultimate displacement than those without hold-downs.

Comparison of lateral load capacity is made to a braced wall sheathed with 9.5 mm wood-based panel and with 8d nail spaced at 150 mm along the perimeter of the panels. Results show that for a portal frame without hold-downs, it has equal or greater lateral load capacity than a 2.44 m braced wall without hold-downs. The lateral load capacity of a portal frame with hold-downs can be as high as the lateral load capacity of a 4.88 m braced wall without hold-downs.

More work is needed to develop design provisions for portal frames used in engineered building.

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REFERENCES

PORTAL FRAME BRACING SYSTEMS USING FIBER REINFORCED POLYMER (FRP) IN LIGHT-WOOD FRAME BUILDINGS

Abdullah Al Mamun¹, Ghasan Doudak², Chun Ni³, Mohammad Mohammad⁴

ABSTRACT: Although most wood-frame buildings are able to meet the minimum wall bracing requirements to resist seismic and wind loads, there are situations where the required length of braced walls may not be sufficient due to openings desired by architectural requirements such as large openings for multi-car garages. Hence, developing alternative bracing systems that can either replace or be used in conjunction with conventional shear wall segments without compromising the structural integrity of the building is desired. Wood portal frame systems have been identified as a viable option to meet the lateral load requirements. However, there is a need to explore the potential for improvements to the portal frame systems to make sure they can be integrated into light wood frame structures as the main lateral load resisting system in combination with conventional shear walls. Test observations have shown that the moment capacity of the corner joint between the header and narrow braced wall segment dominates the portal frame behavior. The current study focuses on optimizing the corner details using full scale joint tests. This paper reports on tests results and failure modes from corner tests on portal frame corner joints retrofitted with Fiber Reinforced Polymer (FRP). Modeling technique to develop numerical model using nonlinear finite element software SAP 2000 to validate the test results is also presented in the paper.

KEYWORDS: Portal Frame, Lateral load bracing system, Alternative bracing system, FRP, Finite element modeling

1 INTRODUCTION

Wood portal frames have been investigated with the purpose of incorporating such systems into light frame wood buildings in order to allow for more open space and larger openings. International Residential Code (IRC) in the US has allowed the use of prescriptively designed portal frames as lateral bracing systems in light frame wood building since 2006 [1]. Although research on portal frame bracing systems has been undertaken by APA- The Engineered Wood Association [2] to include the system in the IRC, very few research work [3] have been conducted to establish their performance using a mechanics-based approach. The current research work is attempting to establish and enhance the performance of portal frames so that they can be used in conjunction with or as a substitute to light frame conventional shear walls with wood sheathing panels.

Since the connection of header and narrow braced wall segment is detrimental to the lateral load resistant capacity of portal frame bracing systems, Fiber Reinforced Polymer (FRP) was used to strengthen the corner joint due to its superior tensile strength. The work reported here focuses on the investigation and comparison of the moment carrying capacity and stiffness for specimens with and without FRP application at the corner joint.

Numerical modelling with non linear finite element software SAP 2000 had been performed and output from model results compared with the full scale corner tests conducted.

2 CORNER JOINT TEST

To simulate the corner joint of a portal frame an L-shaped test assembly was developed. Although the boundary conditions of the test assembly differed from those found in the portal frame, the test was intended to quantify the potential changes in strength, stiffness and ductility due to the application of the FRP material.

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Both standard portal frame technique developed by APA [2] and midply shear wall technique developed by Forintek [4] were evaluated. In midply shear wall detailing, the studs are rotated 90-degree about the longitudinal axis as shown in Figure 1, so the number of stud become double relative to traditional light frame shear walls. The detailing allows for higher lateral load resisting capacity due to engagement of the nails in double or triple shear [4].

2.1 TEST ASSEMBLY DETAILS

Six different configurations of portal frame had been tested as shown in Figure 4. Portal frame corner joint assembly C1, C2, FRP-1 and FRP-2 had been assembled as per standard portal frame technique and consisted of 1760 mm high and 405 mm wide wall segments connected to nominal 2x12 Laminated strand lumber (LSL) header. The narrow wall segment was constructed of 2x4 SPF No. 2 & better studs and sheathed with Oriented Strand Boards (OSB). Two headers were jointed as a built-up section to form an 89 mm (3.5") thick header. The sheathing to lumber connection consisted of 8d common nails with a nailing schedule of 75 mm o.c. and the sheathing to the LSL header was nailed using a nail spacing grid of 75 mm.

Corner joint assembly C3 and FRP-3 had been assembled as per midply shear wall technique and consisted of 1720 mm high and 405 mm wide wall segments connected to a header consisting of a two-ply nominal 2x12 Spruce-Pine-Fir (SPF) members for test specimen C3 and nominal 2x12 Laminated veneer lumber (LVL) header for test specimen FRP-3. The 12.5 mm thick OSB, with a span rating of 2R32, was used as middle and exterior sheathing panels. 16d common nails (4.06 mm in diameter and 90 mm in length) were used to attach the sheathing to the studs. The nailing schedule used was: one row of nails spaced @100 mm on each face of stud on wall segment, and a grid pattern of nails spaced @50 mm on the header.

Six 5.12 mm x 88 mm wood screws were used to connect exterior stud with header. Simpson Strong Tie LSTA 21 (1000 lb capacity) was used to connect the header and end studs in some configurations of the corner joint. Average moisture content of the wood members in the test assemblies was 16.5% and the maximum moisture content did not exceed 19%.

The FRP membrane was applied vertically at the top of the exterior OSB. Both face of OSB sheathing was covered vertically with single sheet of FRP. Horizontal FRP membrane was applied at the end of vertical FRP to avoid delamination. Tyfo SCH-41S-1 reinforcing fabric with Tyfo S Epoxy was used. Tyfo SCH-41S-1 is a custom weave, unidirectional carbon fabric with glass crosses. Fibres are orientated in the 0° direction and glass fibres are orientated at 90°

2.2 TEST SETUP

The test set-up is shown in Figures 2 below. Cyclic load was applied at the top of the assembly through a 9.0 mm thick metal plate connected to an actuator. The metal plate was fastened to the top wood plates using four Ø12.7 mm lag screws. The header was secured to a heavy glulam beam using a combination of metal brackets and 12.7 mm lag bolts on both sides of the headers. The glulam beam was attached to a steel I-beam, which in turn, was anchored to the concrete foundation of the laboratory. Three displacement transducers LVDTs were used to measure the horizontal and vertical movements of the vertical section relative to the header/base.
2.3 LOAD PROTOCOL

ASTM E2126-08 [5] CUREE protocol was used to conduct reversed cyclic loading test as shown in Figure 3. Frequency of cyclic tests was 0.25 Hz and the acquisition of data was captured at a frequency of 10 Hz. Based on the monotonic tests, the reference ultimate displacement was taken as 60 mm (2.36 inch) at the top of the wall segment which was used to develop the CUREE loading protocol.

2.4 TEST MATRIX

For all of the configurations with FRP single specimen was tested while all of the configurations without FRP two specimens were tested. Metal straps were nailed above the sheathing and on top of the FRP membrane. Figure 4 shows all the test configurations of corner joints.
3 EXPERIMENTAL TEST RESULTS

The test results were analysed according to the Equivalent Energy Elastic-Plastic (EEEP) curve in ASTM Standard E2126 [5]. Maximum lateral load capacity and corresponding displacement, initial stiffness, maximum moment and corresponding rotation as well as rotational stiffness were calculated to compare the performance of the corner joint assemblies. For moment resistance calculations, the distance between the center of the header and vertical wall segment corner to the top of bottom plate connected to the load distribution metal plate (i.e. 1.58m) was taken as the moment arm. Transducer 2 (see Figure 2) was used to measure the deflection of the vertical stud to calculate the rotation angle. A summary of the analysis results for corner joint with and without FRP with similar configurations is given in Table 1.

Table 1: Experimental results of Portal Frame Corner joints

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Corner Joint Type</th>
<th>Header</th>
<th>Max. Load (KN)</th>
<th>Max. Moment (KN-M)</th>
<th>Rotation @ Max. Moment (Rad.)</th>
<th>Rotational Stiffness (KN-M/Rad.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Standard</td>
<td>LSL</td>
<td>19.2</td>
<td>15.0</td>
<td>0.0471</td>
<td>6.10</td>
</tr>
<tr>
<td>FRP-1</td>
<td>Standard</td>
<td>LSL</td>
<td>14.29</td>
<td>23.9</td>
<td>0.0793</td>
<td>540</td>
</tr>
<tr>
<td>% Increase</td>
<td></td>
<td></td>
<td>54.0</td>
<td>88.6</td>
<td>0.0807</td>
<td>11.5</td>
</tr>
<tr>
<td>C2</td>
<td>Standard</td>
<td>LSL</td>
<td>10.98</td>
<td>19.7</td>
<td>0.0452</td>
<td>721</td>
</tr>
<tr>
<td>FRP-2</td>
<td>Standard</td>
<td>LSL</td>
<td>17.46</td>
<td>29.2</td>
<td>0.0607</td>
<td>715</td>
</tr>
<tr>
<td>% Increase</td>
<td></td>
<td></td>
<td>54.0</td>
<td>89.4</td>
<td>0.0812</td>
<td>11.5</td>
</tr>
<tr>
<td>C3</td>
<td>Midply</td>
<td>SPF</td>
<td>10.67</td>
<td>18.8</td>
<td>0.0545</td>
<td>599</td>
</tr>
<tr>
<td>FRP-3</td>
<td>Midply</td>
<td>LVL</td>
<td>17.99</td>
<td>28.4</td>
<td>0.0629</td>
<td>728</td>
</tr>
<tr>
<td>% Increase</td>
<td></td>
<td></td>
<td>66.5</td>
<td>88.5</td>
<td>0.0804</td>
<td>12.0</td>
</tr>
</tbody>
</table>

3.1 MAXIMUM MOMENT RESISTANCE

The maximum moment resistance was calculated as the average of maximum positive and negative moment of each specimen. Figure 5 shows that for all the configurations FRP membrane lead to an increase of the moment resisting capacity. Compared to FRP-1, addition of 3 metal straps on each face (FRP-2) increased the moment resisting capacity of standard portal frame with FRP by 22.6%. Midply portal frame with FRP (FRP-3) has also 23.5% higher moment resisting capacity than standard portal frame with FRP (FRP-1).

3.2 ROTATIONAL STIFFNESS

Rotational stiffness was calculated using the slope of the secant line passing between 0% and 40% of the maximum moment measured from the envelope curve. Figure 6 shows that FRP membrane had no significant effect on stiffness properties. The addition of 3 metal straps on each face with FRP (FRP-2) had a slightly more profound effect and increased the rotational stiffness of standard portal frame by 32% compared to FRP-1. Midply portal frame with FRP (FRP-3) has 37% higher rotational stiffness than standard portal frame with FRP (FRP-1).

4 FAILURE MODE

Failure modes of all the corner joint configurations with and without FRP were recorded. In specimens FRP-1, the FRP started to tear at both the outermost corner at the top of header and propagated towards the centre. No failure of exterior OSB was observed.

Metal straps did not show noticeable visual elongation or rupture in specimen FRP-2. Bottom plate failed to transfer load and split at the peak load. There was no sign of exterior OSB failure. In FRP-3 failure of the test specimen was due to the rupture of outside OSB...
followed by failure of middle OSB sheathing. Figure 7 shows typical failure modes of corner joint configurations with FRP observed during the test.

In C1 failure occurred due to rupture of exterior OSB sheathing on both face. In C2 failure was due to the tearing of metal strap followed by the rupture of exterior OSB sheathing. In C3, the middle OSB sheathing ruptured first followed by a strength increase until rupture of exterior OSB.

![Failure of FRP membrane](image1)

(a) Failure of FRP membrane in FRP-1

![Failure of bottom plate](image2)

(b) Failure of bottom plate in FRP-2

![Failure of exterior OSB](image3)

(c) Failure of exterior OSB in FRP-3

**Figure 7**: Typical failure mode of Corner joints with FRP

### 4.1 NUMERICAL MODELING

A non-linear numerical model was developed using SAP 2000 finite element software to validate the test results and behaviour of portal frame corner joint with FRP. In the numerical model, framing member of the narrow braced wall segment was modelled as a beam element, header and OSB sheathing was defined as shell element. Linear elastic behaviour was assumed for framing members and header, and the modulus of elasticity was taken from Canadian timber design code CSA O86. OSB was assumed to be orthotropic shell element and the properties were taken from work by [6].

For sheathing to framing and framing to framing nailed connection, a 3D link element was assigned where component 1 represent withdrawal load, component 2 representing joint loaded parallel to the grain of the framing member and component 3 representing joint loaded perpendicular to the grain of the framing member. The metal strap was modelled as a one dimensional link element. Load slip curve of nail and metal strap was also taken from [6].

In order to accurately predict the failure mode of the portal frame corner joint it was assumed that metal strap, placed over the exterior OSB, carried the entire load after rupture of the exterior OSB. Since SAP 2000 cannot remove the component automatically when OSB reached maximum capacity, splitting of OSB sheathing was assigned manually in the numerical model. It was predicted that ultimate lateral load carrying capacity of corner joint increased until the rupture of metal strap.

Test results from specimens C1 and C2 were used to validate the numerical model developed by SAP 2000. Figure 8 shows that the numerical model can effectively predict the behaviour of corner joint assembly. The same numerical model was used to validate the test result of corner joint test with FRP.

![Validation of FE model](image4)

**Figure 8**: Validation of FE model with corner joint test results

From the corner joint test and failure mode it was observed that only small portion (25mm) of FRP membrane was active to take the tensile stress induced by the lateral load. As an example, during pulling of the braced wall segment, FRP membrane at the top of interior stud was active only. Thus the FRP membrane was defined in the FE model as non linear circular cable element. Diameter of the assigned FRP cable was calculated from the equivalent active area of FRP membrane from corner joint experiment. Monotonic load was applied to the numerical model to identify the
behaviour of corner joint on that specific direction. The FRP properties were assigned from paper [7]. One-dimensional link was used to represent epoxy used to connect FRP to exterior OSB. To connect FRP to FRP another one-dimensional link with the same tensile properties of the circular FRP cable was used. Figure 9 shows the connection detail of FRP at the top corner of the header and narrow wall segment joint.

Figure 9: FE model of Corner joint

Figure 10 shows that FE model was able to capture the maximum capacity of the specimens with FRP. However the prediction of the initial stiffness is not very accurate.

Figure 10: Comparison of FE model with FRP corner joint

Table 3 shows the percentage of variation of maximum load, displacement at maximum load and the initial stiffness of FE model compared to corner joint test with FRP. It is clear from this table that, where the model is able to reasonably predict the maximum capacity, the initial stiffness has not been well-predicted well. This is not uncommon for light frame wood structures, as initial stiffness is a function of complex mechanisms involving, amongst others, friction, gaps and other redundancy issue, which the model does not take into account. This could considerably affect the load path, stress distribution and ultimately, the assembly stiffness.

Table 3: Variation of FE model and Corner joint with FRP

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Corner Joint Type</th>
<th>Max Load (KN)</th>
<th>Displacement @ Max Load (MM)</th>
<th>Initial Stiffness (KN/MM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP-1</td>
<td>Standard</td>
<td>14.26</td>
<td>36.6</td>
<td>6.728</td>
</tr>
<tr>
<td>FE Model-1</td>
<td>Standard</td>
<td>14.09</td>
<td>41.3</td>
<td>1.237</td>
</tr>
<tr>
<td>% Variation</td>
<td></td>
<td>1.2</td>
<td>12.8</td>
<td>76.4</td>
</tr>
<tr>
<td>FRP-2</td>
<td>Standard</td>
<td>17.45</td>
<td>26.0</td>
<td>6.961</td>
</tr>
<tr>
<td>FE Model-2</td>
<td>Standard</td>
<td>17.54</td>
<td>31.6</td>
<td>1.179</td>
</tr>
<tr>
<td>% Variation</td>
<td></td>
<td>0.5</td>
<td>13.0</td>
<td>22.7</td>
</tr>
</tbody>
</table>

5 CONCLUSION

The research work confirmed that it is possible to increase the lateral load resistance capacity of portal frame by minimum 50% with the use of FRP at top of the exterior OSB. The strength can be further increased by using midply shear wall technique.

Nonlinear finite element model had been developed and validated by the test results. The same model was used to predict the behaviour of corner joint test and found that the structural behaviour of FE model resembled well with corner joint with FRP. The FE model will be used in future to predict the behaviour of full scale portal frame with different configurations of FRP at the corner joint. The optimum corner joint configuration will be tested with full scale portal frame to identify the capacity and behaviour to develop high capacity portal frame.

REFERENCES

EXPERIMENTAL ANALYSIS OF TIMBER DIAPHRAGM'S CAPACITY ON TRANSFERRING HORIZONTAL LOADS IN TIMBER-STEEL HYBRID STRUCTURE

Zhong Ma¹, Minjuan He²

ABSTRACT: Timber-steel hybrid structure with steel moment resisting frame and timber floor is a good and novel type of construction in seismic zone because of its light weight. In seismic analysis, timber diaphragm is classified as either flexible or rigid diaphragm for purpose of horizontal shear force distribution depending on its in-plane stiffness, which influences structural load bearing capacity and deformation and the method by which lateral load is transferred from diaphragm to steel frame. The diaphragm shear forces shall be distributed to the vertical resisting elements based on tributary area when it is defined as flexible and on lateral stiffness of the vertical resisting elements of the bottom frame when it is rigid. Aiming to understand in-plane stiffness of timber diaphragm and its capacity on distributing horizontal loads to single frame in timber-steel hybrid structures, an pseudo-static experiment about a one-story two-span steel frame with light frame wood diaphragm under lateral force was conducted. This paper presents the test details and analysis results, which would be beneficial to studying seismic performance of timber-steel hybrid structure.

KEYWORDS: timber-steel hybrid structure; steel frame; timber diaphragm; horizontal loads; in-plane stiffness; seismic performance

1 INTRODUCTION

This timber-steel hybrid structure composed by steel moment resisting frame and light frame wood diaphragm is a good and novel structure style especially in seismic zone, because of its lightweight, dissipative performance, ductility, energy-saving and rapid construction. The crucial issue for timber-steel hybrid structure is in-plane stiffness of the timber diaphragm. Code for design of concrete structures[1] considers wood floor as flexible diaphragm. The ASCE 7-05[2] defined that diaphragm is permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical element of the seismic force-resisting system of the associated story and as rigid when its deflection less than two times the average story drift. At the same time, it also points out that the structure analysis shall consider the relative stiffness of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with the code’s provision, the structure analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semi-rigid modelling assumption). Experiment study on in-plane stiffness of the wood diaphragm and its influence on transferring horizontal loads in whole structure were investigated by a lot of people. James Wescott Bott [3] performed multiple non-destructive experimental tests on six full-scale wood diaphragms of varying sizes, aspect ratios, and load-orientations to study the in-plane stiffness of it. Each test of each specimen involved a different combination of construction parameters including blocking, foam adhesive, presence of designated chord members, corner and centre sheathing openings, and presence of walls on top of the diaphragm. Andre Filiatrault etc.[4] investigated fourteen different wood diaphragm in order to acquire some parameter’s influence on the in-plane flexibility/rigidity of the diaphragm. Some conclusions were reached that the installation of diaphragm blocking at unsupported plywood panel edges and (or) sub-floor adhesive caused a significant increase in the diaphragm shear rigidity and the diaphragm nailing, blocking, and gluing had little influence on the diaphragm flexural rigidity. Eng. Christian Baldessari[5] tests the behaviour of in-plane timber floors which are differently refurbished to ascertain the stiffness of the different solutions and to study the influence on the global behaviour of the building. David F. Peralta etc.[6] constructed three diaphragm specimens with elements and connection details typical of pre-1950’s construction.

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in unreinforced masonry (URM) buildings found in the Central and Eastern regions of the United States. Specimens, using different rehabilitation methods such as unblocked and blocked plywood overlays on top of the sheathing, a steel truss attached to the bottom of the joists and so on, were tested under quasi-static reversed cyclic loading to evaluate their in-plane stiffness. The result was compared with the provisions for wood diaphragm in the FEMA 273[7] and FEMA 356[8] and shows that FEMA 273 tended to overpredict the stiffness, while FEMA 356 tended to underpredict it. Shuo Li etc.[9] presented a structural test of a one-story hybrid structure with concrete frame and light frame wood diaphragm and experimental data show us that the light frame wood diaphragm could be treated as semi-rigid diaphragm in timber-concrete hybrid structure, maintaining high in-plane stiffness in the elastic and plastic range, which have a large contribution on the distribution of lateral load.

The greater majority of document involving hybrid structure are wood diaphragm with unreinforced masonry (URM) buildings or concrete structure, there are few literature with respect to structure with steel moment resisting frame and timber floor. This paper describes the detail of the pseudo-static experiment about a one-story two-span steel frame with light frame wood diaphragm under horizontal force, aiming to study timber diaphragm’s capacity on transferring horizontal loads in timber-steel hybrid structure.

2 EXPERIMENT
2.1 DESCRIPTION OF THE TEST STRUCTURE
The completely symmetrical test structure, composed by steel frame and light frame wood diaphragm, is a single-story two-span frame construction with 6m long by 3m wide by 2.8m height, as depicted in figure 1 and 2.

![Figure 1: Floor plan of the specimen](image1.jpg)

![Figure 2: Wood diaphragm-steel frame joint](image2.jpg)

Table 1 lists construction details of the steel frame

<table>
<thead>
<tr>
<th>Column</th>
<th>H150×150×7×10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>H148×100×6×9</td>
</tr>
<tr>
<td></td>
<td>Q235B</td>
</tr>
<tr>
<td>Joint</td>
<td>Flange welded</td>
</tr>
<tr>
<td></td>
<td>Web bolted</td>
</tr>
</tbody>
</table>

Table 2 lists construction details of the wood diaphragm.

<table>
<thead>
<tr>
<th>Panel</th>
<th>15mm thick OSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing</td>
<td>Nail</td>
</tr>
<tr>
<td></td>
<td>Spacing: perimeter, 150/75mm, intermediate, 300/150mm</td>
</tr>
<tr>
<td>Joist</td>
<td>38×140SPF@300mm</td>
</tr>
<tr>
<td>Blocking</td>
<td>Double 38×140 SPF</td>
</tr>
</tbody>
</table>

2.2 EXPERIMENT SETUP
The top of structure’s column was connected with two distribution beam by horizontal hinge and the load was applied to the structure by two jams fixed on the resistant frame in one end and hinged on the middle of the two distribution beam by horizontal hinge in the other end, which guarantees the load is 2 times in the middle column and 1 times each in the side. The experiment setup is shown in figure 3.

2.3 LOADING METHOD
Table 3 lists five test cases.

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Only steel frame</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Steel frame + Wood diaphragm</td>
<td>Monotonic load</td>
</tr>
<tr>
<td></td>
<td>Nail spacing: perimeter, 150mm, intermediate, 300mm</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Steel frame + Wood diaphragm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nail spacing: perimeter, 75mm, intermediate, 150mm</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Steel frame + Wood diaphragm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nail spacing: perimeter, 75mm, intermediate, 150mm</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>The same as Case 4</td>
<td>Cyclic load</td>
</tr>
</tbody>
</table>

2.3.1 Monotonic load
The case 1–4 shown in figure 3~5 were investigated in monotonic load with maximum load 50KN equally divided into 5 grades, aiming to study influence of the wood diaphragm’s in-plane stiffness to shear force distribution of the structure in elastic range.
2.3.2 Cyclic load

The case 5 seen in figure 5 was investigated in cyclic load controlled by force in elastic range and by displacement of the middle column’s top in plastic phase, purposing on study the horizontal load transfer capability of the wood diaphragm in plastic range and obtain ductility and dissipative performance of the hybrid structure. The cyclic loading method is shown in figure 6.

2.4 DATA MEASUREMENTS

Ten transducers were used to measure displacements, six recording the top displacement of the column and four the middle points of the beam, as depicted in figure 7.

Strain gauges were used to measure shear forces and bending moments of single steel frame as depicted in figure 8. For example, S3,S4,S5,S6,S15,S16,S17,S18 were attached to elastic zone of the column in the first single steel frame, which ensure shear force of the single steel frame can be measured even though the whole structure was damaged. Other strain gauges arranged on the second and third were the same as the first.
The column’s shear force $Q$ can be obtained by the following equations.

$$
\frac{N}{EA} + \frac{M}{EW} = \varepsilon_{\text{max}} \\
\frac{N}{EA} - \frac{M}{EW} = \varepsilon_{\text{min}}
$$

(1)

(2)

(3) is derived from (1) - (2):

$$
M_t = \left( \varepsilon_{\text{max}} - \varepsilon_{\text{min}} \right) EW
$$

(3)

In the same way, (4) is acquired:

$$
M_b = \left( \varepsilon_{b\text{max}} - \varepsilon_{b\text{min}} \right) EW
$$

(4)

Substituting (3), (4) into (5), $Q$ can be deduced:

$$
Q = \frac{M_t - M_b}{L} = \left( \varepsilon_{\text{max}} - \varepsilon_{\text{min}} - \varepsilon_{b\text{max}} + \varepsilon_{b\text{min}} \right) EW
$$

(5)

Where $N =$ Axial force, $M =$ Bending moment, $Q =$ Shear force. $\varepsilon =$ strain, $E =$ Young’s modulus, $1.9\times10^5$ N/mm$^2$ obtained by material test. $A =$ Section area, $W =$ Bending modulus, $L =$1000mm, the distance between S3 and S5.

Shear forces of the second and third single steel frame can be received in the same way.

2.5 TEST OBSERVATIONS

2.5.1 Monotonic load

In the case 1, displacement of the middle column is obviously large than the side and in the rest cases the difference of displacement between the column is small. In addition, no damage was observed in all 4 cases.

2.5.2 Cyclic load

As displacement of the middle column increase, some nails in shear wall were cut off and pulled out and OSB panels were dislocated and tore off by nails. The welding seams of the main beam-column joint in all three single steel frames were absolutely abrupt and the bottom of the column was flexed. No damage was seen in the wood diaphragm and its connection to steel beam in addition to small dislocation of the OSB panels. Figure 10 shows the destrucional forms.

2.6 TEST DATA ANALYSIS

2.6.1 Monotonic load

Displacement of the column’s top measured by the transducer and shear forces of three single steel frames computed by the formula (5) in the case 1-4 are shown in figure 11~14.

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*(Figures and diagrams related to test observations and data analysis are not included in the text.*

---

*Figure 10: Destructional forms in test*

*Figure 11: Case 1*

*Figure 12: Case 2*
Figure 15 shows the curve of load transfer capability $b$ to the ratio $a$ of wood diaphragm’s in-plane stiffness to the vertical resisting elements.

$$b = \frac{F}{2P - \frac{P + 2P + P}{3}} = \frac{3F}{2P} \quad (6)$$

Where $F$ is the shear force transferred from the middle vertical resisting elements to two sides, which could be computed by strain value. The load applying on the middle vertical resisting elements is $2P$ and on two sides is $P$ each as depicted in figure 7. When the wood diaphragm is absolutely rigid, three single vertical resisting elements cooperate with each other completely and total load acting on the structure is passed on them uniformly leading to that shear forces of them are equal and $b$ is 1. On the contrary, when wood diaphragm is absolutely flexible, the load $2P$ imposed on the middle is failed to be transferred to two sides so that $b$ is 0. In the remaining cases, wood diaphragm possessing certain in-plane stiffness, neither rigid nor flexible, may transfer horizontal load from the middle to two sides to a certain extent and $b$ is between 0 and 1. The load transfer capability of the link beam is about 0.04, which is too small to be neglect here.

In the case 1–4, the structure was in elastic range and shear force proportion between three vertical resisting elements should be consist with displacement proportion between the three column’s top. Figure 11 reveals that displacement and shear force of the side are both about half of the middle and $b$ is about 0 in the case 1, indicating that the load transferred only by link beam from the middle single steel frame to two sides is so small as to be ignored. When wood diaphragms were installed in the structure in the case 2, displacement and shear force of the side are both close to 90% of the middle as depicted in figure 12 and $b$ is near to 0.9 too, illustrating that the wood diaphragm possessing certain in-plane stiffness has much influence on load transfer capability compared with the case 1. When the wood diaphragm’s nail spacing was reduced to half contrasted with the case 2 in the case 3, displacement and shear force of the side are both slightly large than 90% of the middle as shown in figure 13 and $b$ is also a bit large than 0.9, revealing that reducing nail spacing on the basis of the case 2 has not notable effect on shear force distribution. It could be explained that decreasing nail spacing raises in-plane stiffness of the wood diaphragm,
however, $a$ is large than 3 while $b$ is close to 0.9 and then $b$ increases very slowly as $a$ ascends, as shown in figure 15. In the case 4, owing to that shear force distributed on wood shear wall is proportion to it on single steel frame in elastic range, the proportion of shear force distributed on three single vertical resisting elements is equal to it on three single steel frames. So in figure 14(b), x axis shows only the shear force of three single steel frames. Displacement and shear force of the side are both in the vicinity of 75% of the middle and b is about 0.75 too, which could be attribute to that wood shear wall raises in-plane stiffness of the vertical resisting elements so that $a$ and $b$ are both in reduction. In addition, on account of the wood shear wall contacting with steel frame not tightly at the beginning, the curve flexes seriously at the initial stage in figure 14, out of accord with the case 1~3, however, along with increasing load, the curve becomes straight little by little.

2.6.2 Cyclic load
Because of symmetry, hysteric curves of the load-displacement of the middle column and only the first side column are shown in figure 16.

![Figure 16: Hysteretic curves of load-displacement](image)

It can be seen that the displacement of the side column has smaller difference with the middle even though when the structure get into plastic stage, proving that the wood diaphragm had little stiffness degrading and performed well on distributing horizontal loads all the time, which meets the test observation that the wood diaphragm had no obvious damage. Moreover, the hysteric loop is plump and symmetrical and the ratio of ultimate displacement to yield displacement is relatively large, showing that the hybrid structure exhibits excellent dissipative and ductility performance.

3 CONCLUSIONS
The pseudo-static experiment about a one-story two-span steel frame with light frame wood diaphragm under horizontal force allows a better understanding of the timber diaphragm’s in-plane stiffness and its capacity on transferring horizontal loads in timber-steel hybrid structure. Based on the experimental results obtained, conclusions can be drawn that

a) The ratio $a$ of wood diaphragm’s in-plane stiffness to the vertical resisting elements has much influence on horizontal load transfer capability $b$. when $a$ is between 0~3, $b$ varies rapidly and $a$ increases to 3 then, $b$ is near to 0.9. Thereafter, $b$ goes up very slowly while $a$ keep on growing, so it is consider that when $a$ is large than 3, the wood diaphragm is fully rigid and carrying on raise its in-plane stiffness will be nonsense. In the paper, nail spacing has little influence on load transfer capability of the wood diaphragm because of $a$ is large than 3 just then.

b) Light frame wood diaphragm, having certain in-plane stiffness and strength, performs well on transferring horizontal loads when the hybrid structure was in elastic and plastic phase. So it should be treated as semi-rigid diaphragm in many cases, neither traditionally flexible diaphragm in China.

c) The timber-steel hybrid structure, in addition to lightweight, rapid construction and renewable feature, shows excellent dissipative and ductility performance, which makes it advantageous in seismic zone.

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REFERENCES
EVALUATION OF EARTHQUAKE RESPONSE OF WOOD FRAME WITH DIAGONAL BRACE FASTENED BY NEW BRACE FASTENER

Tomoki Furuta¹ and Masato Nakao²

ABSTRACT: A new type brace fastener which fastens the end of a diagonal wood brace to the end of column is developed for Post and Beam construction wooden houses. Since the brace fastener contains high damping rubber, it absorbs a displacement between the end of a wood brace and a column, damage of wood around wood screws is able to be prevented. On shake table test of a wood frame with a wood brace fastened by the new brace fasteners, the new brace fastener showed higher seismic performance in comparison with normal brace fastener especially under large earthquake motion. Moreover, by means of earthquake response analysis, it was confirmed that the new brace fastener showed good seismic performance under several earthquake motions.

KEYWORDS: Wooden house, Brace fastener, High damping rubber, Shake table test, Earthquake response analysis

1 INTRODUCTION

Wooden houses play an important role to preserve human life and property. They need to bear repeated large earthquakes and to be in use even after the earthquakes. However, once wood is subjected to a force, a decline of stiffness occurs. Therefore, shear stiffness of shear walls subjected to an earthquake is considered to decline, which means wooden houses are hard to resist the repeated large earthquakes.

From the expressed reason, authors have been developed a new brace fastener which fastens the end of a diagonal wood brace to the end of column for Post and Beam construction wooden houses[1, 2]. Since the brace fastener contains high damping rubber, it absorbs a displacement between the end of a wood brace and a column, damage of wood around wood screws is able to be prevented. Therefore, the brace fastener minimizes the decline of stiffness, moreover high damping force by the high damping rubber is produced.

In this paper, to evaluate basic performance of the new type brace fastener, shake table test of a wood frame with a wood brace fastened by the new brace fasteners firstly. Wood braces fastened by the new type brace fastener presented in this paper is able to be considered as one of shear walls in Building Standard Law in Japan. Secondly, considering the result of the shake table test,
earthquake response analysis is conducted to evaluate the performance of the new brace fastener in ordinary Post and Beam wooden houses.

2 OUTLINE OF THE NEW BRACE FASTENER

The new type brace fastener, as shown in Figure 1, consists of a L-shaped normal brace fastener, a steel plate and a high damping rubber with 5mm thick. The high damping rubber glues a L-shaped normal brace fastener and a steel plate together. The L-shaped normal brace fastener is fastened to the end of a column with nine 75mm wood screws and the steel plate is fastened to the end of a wood brace with six 45mm wood screws. In addition to the wood screws, four 45mm wood screws which fasten the L-shaped fastener and a wood brace directly are added for fail-safe considering exfoliate of high damping rubber. Picture 1 shows a wood frame with wood braces which are fastened by the new brace fastener at the ends of braces as shown in Picture 2. The new brace fastener is able to be used in place of a normal brace fastener to connect the end of a wood brace whose cross section is 90mm x 45mm to the end of a column.

3 SHAKE TABLE TEST

Shake table test of a wood frame specimen with a wood brace fastened by the new brace fasteners to evaluate its seismic performance. Figure 2 shows a wood frame for shake table test.

Three stages of excitation were performed, namely the specimen with fail-safe wood screws(Stage 1), the one without fail-safe wood screws(Stage 2) and the one with only fail-safe wood screws(Stage 3) as shown in Table 1. In each stage, one wood brace, whose cross section is 90mm x 45mm, was fastened by the brace fasteners at the both ends of the wood brace. In Stage 3, the high damping rubber did not glue a L-shaped normal brace fastener and a steel plate, and there was no wood screw which fasten the steel plate to wood brace.

Wood species of column, sill and brace of the specimen

![Picture 2: New brace fastener installed at the end of brace]

Table 1: Arrangement of wood brace on each stage

<table>
<thead>
<tr>
<th>Stage1</th>
<th>Stage2</th>
<th>Stage3</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Stage1: 6 standard screws and 4 fail-safe screws" /></td>
<td><img src="image2" alt="Stage2: 6 standard screws" /></td>
<td><img src="image3" alt="Stage3: 4 fail-safe screws only" /></td>
</tr>
</tbody>
</table>

![Figure 2: Wood frame for shake table test]

![Figure 3: Shear force-drift relationship under 20% of JMA Kobe wave on Stage 1](image4)
was Tsuga heterophylla and beam was Dugrus fur. Since weights on the top of the specimen was approximately 20kN, total weight of the upper half of the specimen was 24.5kN. On each stage, approximately 5%, 10% and 20% of JMA Kobe wave were input to the specimen in one direction only. Before starting each stage, to have the natural period and the damping ratio, small random wave and impulse wave were input.

In the case of the new brace fastener with fail-safe wood screws(Stage 1), a maximum response drift under approximately 20% of JMA Kobe wave was 3.4%, where principal motion of the JMA Kobe wave was input to the wood brace as tensile force. Figure 3 shows the shear force-drift relation ship. Though deformation at the corner of the L-shaped fastener and embedment on the surface of the column were observed, fatal damage to the fastener was not detected.

In the case of the fastener without fail-safe wood screws(Stage 2), the principal motion of JMA Kobe wave was input to the wood brace as compressive force. Figure 4 shows the shear force-drift relationship under approximately 20% of JMA Kobe wave. Maximum response drift was 2.4%, compressive buckling of the wood brace occurred. Slight bending deformation of the fastener due to the buckling of the wood brace was observed. Little difference of the result of the specimen with and without the fail-safe wood screws was detected as shown in Figure 5 with respect to shear force-drift relationship when the wood braces were subjected to tensile force. More over, additional wood screws as fail-safe caused no brittle failure on the wood brace such as...
splitting failure.

Figure 6 shows the results of a wood frame with the brace fasteners whose high damping rubber did not glue a L-shaped fastener and a steel plate under approximately 10% of JMA Kobe wave (Stage 3). Only four fail-safe wood screws fasten the L-shaped fasteners and the end of brace.

Initial shear stiffness of the specimen with only fail-safe wood screws (Stage 3) was approximately 50% of the one with fail-safe wood screws (Stage 1) as shown in Figure 7. It is considered that friction force between the surfaces of the high damping rubber and the two steel parts, namely the L-shaped fastener and the steel plate was produced due to a compressive force by the wood screws. It was confirmed that the fail-safe wood screws show moderate performance even the exfoliation of the high damping rubber occurs.

An additional test of a wood frame with two wood braces fastened by the new brace fasteners was conducted. The fastener was attached using six standard and four fail-safe wood screws the same as Stage 1. In this test, 10%, 30% and 50% of JMA Kobe waves were input to the specimen. Maximum drift and shear force under 50% of JMA Kobe were 3% and 13kN, respectively as shown in Figure 8. In a series of this test, a test of a wood frame with two wood braces fastened by normal brace fasteners was also performed. Figure 9 shows the result under 50% of JMA Kobe wave. The maximum drift was 5.6%, it is 1.9 times as much as the drift with the new brace fastener. Figure 10 shows maximum drift of the tests with the new fastener and with normal fastener. In the tests under 10% and 30% of JMA Kobe wave, the maximum drifts of the two kinds of fasteners were almost the same. Therefore, the use of the new brace fastener is effective especially under relatively large earthquake motion.

## 4 EARTHQUAKE RESPONSE ANALYSIS

Earthquake response analysis was carried out to examine the seismic performance of the new brace fastener under several earthquake motion.

### 4.1 Earthquake Response Analysis

Earthquake response analysis was carried out to examine the seismic performance of the new brace fastener under several earthquake motion.
NCL model[3] was adopted for hysteresis model of a wood frame with a wood brace fastened by the new brace fasteners. Figure 11 shows the hysteresis model calibrated by the test results. A wood frame with a wood brace fastened by normal brace fasteners and the one with nailed plywood were also modeled as shown in Figure 12 and Figure 13, respectively. Shear forces presented here are shear force per wall length.

Three amounts of shear wall, minimum amount of required wall length in the Building Standard Law in Japan(100% wall quantity), 1.5 times of the minimum(150% wall quantity) and 2 times of the minimum(200% wall quantity) were set to the analysis model.

Input earthquake to the analysis models were El-Centro NS and BCJ L2(Level 2 simulated earthquake wave for structural design by The Building Center of Japan) in addition to JMA Kobe NS. Figure 14 shows the unscaled response spectra of the three earthquake waves. They were scaled to 980 gal(1G) on peak acceleration of elastic response as listed in Table 2. To calculate the input level, 0.5% drift secant shear stiffness of 100% wall quantity model was regard as elastic shear stiffness.

From the analysis result, as shown in Figure 15, the maximum drifts of a wood frame with wood braces fastened by the new brace fasteners are conservative in comparison with the one fastened by normal brace fasteners. The new brace fastener showed relatively good seismic performance especially under large earthquake motion. It is considered that the high damping rubber enhanced deformation capacity of the brace fastener.

5 CONCLUSIONS
A new type brace fastener which fastens the end of a diagonal wood brace to the end of column is developed for Post and Beam construction wooden houses.

On shake table test of a wood frame with a wood brace fastened by the new brace fasteners, it was found that fail-safe wood screws cause no brittle failure on wood braces such as splitting failure. In the case of the specimen with only fail-safe wood screws, even if the high damping rubber did not glue a L-shaped fastener and a steel plate, a decline of shear stiffness was approximately 50%.

From the additional test, the new type brace fastener showed higher seismic performance in comparison with normal brace fastener especially under large earthquake motion.

Moreover, by means of earthquake response analysis, it was confirmed that the new brace fastener showed good seismic performance under several earthquake motions.

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NUMERICAL ANALYSIS ESTIMATION OF HORIZONTAL RESISTANT FORCES OF VARIOUS MUD PLASTERED WALLS IN JAPANESE TRADITIONAL WOODEN STRUCTURES

Koji Yamada¹, Yoshiyuki Suzuki², Masami Goto³, Hiroyuki Nakaji⁴

ABSTRACT: In this report, to verify a numerical analysis modelling mud plastered walls, we compare the restoring force characteristics of actual test and the ones by the numerical analysis on eight types of mud plastered walls in wooden frames. The numerical model of mud plaster wall is composed of frame member for both post and beam, the end springs for the connector of frames, rectangular solid model for mud plaster and contact analysis model between mud plaster and frame. As a result, the proposed numerical method gives approximate restoring force characteristics.

KEYWORDS: Mud plastered wall in wooden frame, Numerical analysis, Horizontal resistant force characteristic

1 INTRODUCTION
The main earthquake resistant element of a Japanese traditional wooden structure is a mud plastered wall within a wooden frame. The horizontal resistant force of a mud plastered wall within wooden frame is influenced by mud plaster strength, lath, batten, and column. In former report [1][2], we report the relation between the mud plaster strength and the horizontal resistant force of a mud plaster wall for a Japanese traditional wooden structure by finite element method. In this report, we report that we are able to calculate various types of mud plastered walls within a wooden frame by the proposed method.

2 VARIOUS MUD PLASTERED WALLS IN WOODEN FRAME
We tested 8 types of mud plastered walls in wooden frame in Figure 1. The height of every wall is 2730mm. The length of wall is 1820mm or 3640mm. Other features are as follows: Type 1 wall consists of a 910mm length mud plastered wall within wooden frame with 3 horizontal battens. Type 2 wall consists of a 1820mm length mud plastered wall with 3 horizontal battens. Type 3 wall consists of a hanging mud plastered wall and a breast mud plastered wall. Type 5 wall consists of 2 910mm length mud plastered walls, a hanging mud plastered wall and a breast mud plastered wall. Type 6 wall consists of 2 910mm length mud plastered walls and a hanging mud plastered wall. Type 7 wall consists of a 910mm length mud plastered wall and a long hanging mud plastered wall. Type 8 wall consists of a 910mm length mud plastered wall, a long hanging mud plastered wall and a long breast mud plastered wall. The section of column is 120 x 120 mm. The section of batten is 15 x 120 mm. Every specimen has 20kN weight on its top, and all columns of a specimen are not fixed on the ground at 1st test. But the column goes up in some specimens during a loading test, then specimens are rotated by their lateral load. So we fixed the columns on the ground. The load schedule for full-scale wall test is 2 times iteration at 10 deformation angles: 1/480, 1/240, 1/120, 1/90, 1/60, 1/45, 1/30, 1/20, 1/15, 1/10 rad.

3 STRENGTH OF MUD PLASTER
Two material tests are executed against mud plaster of 1st and 2nd layer. The mud plaster is mined at Kyoto in Japan. These material tests are compression test and splitting test in Figure 2. The specimen is a cylinder with 50mm diameter and 100mm height. The results of compression tests and splitting test are shown in Figure 3. The test results show that the maximum strengths of compression tests are 0.5 - 0.6 N/mm², and the maximum strengths of splitting tests are 0.12 - 0.15 N/mm²

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springs for the connector of frames, rectangular solid model for mud plaster and contact analysis model between mud plaster and frame. Wooden frames are supposed to be linear materials. A bending fracture and the drop out at a joint are simulated by member end springs. Member end springs are adopted on an axial force, a shear force, and a bending moment respectively. The restoring force characteristics on an axial force and a bending moment are shown in Fig. 4. The restoring force characteristic on a shear force is supposed as linear system. The restoring characteristics of end springs are set using examples from element tests.

A finite element for mud plaster is a rectangular element that is a smeared crack model. The smeared crack model has 12 evaluation axes. The restoring force in Fig. 5 is calculated in each evaluation axis respectively. The representative axis for an element is the closest axis to the principal strain. The restoring characteristics of mud plaster solid model are set using examples from both the compression test and the splitting test of mud plaster cylinders.

A contact element is the rectangular element including 2 springs in Fig. 6. This element transfers axial forces and frictional forces. The compression stiffness is the product of the 1st Young’s ratio of mud plaster (Ec1), the half-length of a frame member, and the thickness of mud plaster element. A friction factor is 0.4.

The proposed calculation method is shown in reference [1] and [2]. The resistant system of mud plaster wall is defined as follows:

1) A wooden frame is linear material except the bending fracture
2) Inroad of wooden frame to mud plaster is represented a crush of mud plaster
3) The rotation resistance at wooden frame connection is represents a non-linear spring.

The numerical model of mud plastered wall is composed of frame member for both post and beam, the end

**Figure 1: Tested mud plastered walls**

**Figure 2: Material tests**

**4 ASSUMPTION OF NUMERICAL MODEL**

The proposed calculation method is shown in reference [1] and [2]. The resistant system of mud plaster wall must express a crush of mud plaster, inroad of wooden frame (especially batten), the rotation resistance at wooden frame connection, and the contact between mud plaster and a wooden frame. Therefore the modelling policy of mud plaster wall is defined as follows:

1) A wooden frame is linear material except the bending fracture
2) Inroad of wooden frame to mud plaster is represented a crush of mud plaster
3) The rotation resistance at wooden frame connection is represents a non-linear spring.

The numerical model of mud plastered wall is composed of frame member for both post and beam, the end

**Figure 3: Result of compression test, split test and their mechanism model**

**Figure 4: Restoring force characteristics on an axial force and bending moment**
5 COMPARISON BETWEEN TEST AND CALCULATION

In the test, cracks are firstly occurred on the battens in all specimens. Then the second coat mud plaster on the batten falls down from the mud plaster wall. The destroyed specimens are shown in Figure 8. The restoring force characteristics by actual test with fixed columns and their skeleton curve by calculation are shown in Figure 9. Almost all skeleton curves are close to the upper envelope curve of actual test.

6 CONCLUSIONS

In this report, we compare the restoring force characteristics of actual test and the ones by the numerical analysis on 8 types of mud plastered walls in wooden frames. As a result, it is found that the proposed numerical method can estimate well the restoring force characteristics.
FIGURE 9: Restoring force characteristic of mud plastered wall and its skeleton curve by calculation

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REFERENCES
LOAD-DISPLACEMENT RELATIONSHIP OF ACM WOODEN-HOUSE RETROFITTING BRACE

Tomiya Takatani¹

ABSTRACT: In this paper, a load-displacement relationship of ACM (Advanced Composite Material) brace was described. Horizontal resistant force of the ACM brace installed on a timber frame, which consists of timber beams, columns and puncheons, was investigated through some experiments in order to apply the ACM bracing method to a wooden house seismic retrofitting work. In general, a lot of old wooden houses in Japan do not have enough seismic resistant force against a large earthquake. Therefore, the ACM bracing method has been developed for a seismic retrofitting work of wooden house. It was found from some experiments that seismic retrofit work using ACM brace to wooden house has a more significant horizontal resistant strength in comparison with other retrofitting works.

KEYWORDS: ACM Bracing Method, Timber-Frame Experiment, Seismic Retrofit, Wooden-House

1 INTRODUCTION

It is well known in Japan that a lot of RC buildings and wooden-houses built before 1981 by some old earthquake resistant design codes were destroyed in the 1995 Hyogo-ken Nanbu Earthquake. Therefore, Japanese Government has been adopted several significant politics concerning this issue since 1995 in order to reduce a lot of earthquake damages for RC buildings and wooden houses built before 1981 as quickly as possible. As some seismic retrofitting policies adopted by Japanese Government had been not carried out smoothly, Japanese Government has been demanding a numerical target for many local self-governments in Japan to quickly improve a seismic retrofitting ratio for all buildings [1]. This numerical target of seismic retrofitting ratio including a lot of wooden houses as well as RC building structures is up to 90% until 2017.

The author has already proposed a seismic retrofitting work for RC building structure using an advanced composite material (referred to as ACM) bracing method, which consists of a carbon fiber reinforced plastic (referred to as CFRP) material, steel sleeves and anchors, in order to save a lot of residents’ lives against a large-damaged earthquake[2]. CFRP material has several advantages of strong and light-weight feature, good durability, and wide applicability in comparison with the steel material.

In this paper, a seismic retrofitting work of a wooden house using the ACM bracing method is proposed in order to aim at both low cost and short construction period in comparison with the traditional bracing method conducted in Japan. The ACM brace for a wooden house consists of CFRP plate (referred to as e-plate) and steel one. An e-plate has Young’s modulus, 156kN/mm², and its tensile strength is 2,400N/mm²[3]. The width and the thickness of the e-plate for the ACM bracing method are 25mm and 1.2mm, respectively. An advanced epoxy resin [4], E390, was employed for bonding between an e-plate and three steel plates.

The purpose of this paper is to investigate a load-displacement relationship of the ACM brace for a seismic retrofit for wooden house.

2 LOAD-DISPLACEMENT RELATIONSHIP OF ACM BRACE

Figure 1 shows a loading equipment system to investigate a load-displacement relationship of the ACM wooden-house retrofitting brace. A timber frame shown in Figure 2 is installed in the loading equipment system.
as shown in Figure 1. A typical timber frame consists of three columns, two puncheons and two beams as indicated in Figure 2. The timber frame has 2,730mm height and 1,810mm width, and is a typical size of wooden house in Japan. Cross section of the timber member is a square with 105mm by 105mm, and a timber puncheon is a rectangular with 105mm by 30mm.

Figure 3 illustrates three ACM brace models proposed in this paper. Model 1 shown in Figure 3(a) is a typical ACM brace model, which is applied to the intersection surface between timber column and beam, and is consist of an e-plate and three steel plates. Model 2 in Figure 3(b) is fixed on the inside surface between timber column and beam. Model 3 in Figure 3(c) is fixed on the outside wall of wooden house, and is more economical than other models from a view of retrofitting cost.

Photo 1 shows three ACM brace models for the retrofitting of wooden house. An e-plate and three steel plates for the ACM brace are fixed by an advanced epoxy resin, E390, which has been developed by Konishi Co., Ltd.
[4]. The epoxy resin left over on the steel plate of ACM brace can be seen in Photo 1(c). Photo 2 shows a jack system with a load cell in the horizontal loading experiment of a timber frame with ACM brace. A relationship between horizontal loading cycle and deformation angle, $R$, of timber frame is illustrated in Figure 4. Deformation angle, $R$, can be defined by the following equation.

$$ R = \frac{\delta}{H} $$

where, $\delta$ and $H$ are the horizontal displacement and the height of timber frame, respectively. According to the relationship shown in Figure 4, horizontal load is charged to timber frame by the jack system shown in Photo 2. A measurement system with PC and data recorders used in this experiment is indicated in Photo 3. Two displacement gauges shown in Photo 4(a) are employed in order to measure a large-scale horizontal displacement at the top part of timber frame, and also horizontal displacement at the bottom part of timber frame is obtained by a displacement gauge shown in Photo 4(b). In this paper, load-displacement relationships for three ACM wooden-house retrofitting brace models in Figure 3 were obtained by the loading equipment and the measurement systems described above.

Figure 5 shows a hysteresis loop, that is, a load-displacement curve, for Model 2 in Figure 3(b) during static loading cycle condition shown in Figure 4. Only tensile load was operated to investigate a load-displacement relationship for tensile characteristics of the ACM brace. As a reference, the hysteresis loop for a timber frame without ACM brace is illustrated in Figure 5, too. It can be observed from this figure that the timber frame reinforced by ACM brace (Model 2) has a significant high resistant strength in comparison with the timber frame without ACM brace. The maximum tensile load is 12.6kN at the failure of timber frame with ACM brace, when some nails came off from the steel plate of ACM brace and the timber beam. When the standard resistant force of timber frame is...
1.275kN at the deformation angle $R = 1/120$ of the timber frame (wall) with 1.0m length, the wall ratio is defined 1.0 in the Building Standard Law in Japan. Namely, an allowable resistant force for the wall ratio of 1.0 is 1.275kN per 1.0m frame (wall) length. According to this definition in the Building Standard Law in Japan, the tensile force of timber frame with the ACM brace in Figure 5 is 4.5kN at the horizontal displacement 25mm of timber frame, which can be obtained from the deformation angle $R = 1/120$ of timber frame used in this experiment.

Figure 6 indicates a load-displacement curve for Model 3 shown in Figure 3(c). The maximum tensile load is 12.2kN at the failure of timber frame, when an e-plate came off from ACM brace steel plates. Tensile force is 7.2kN at the horizontal displacement 25mm of timber frame.

Photo 5 shows a finished wall of wooden house retrofitted by the ACM brace shown in Figure 3(c). The outward appearance of this wooden house with the ACM brace may look fine on the outside.

3 CONCLUSIONS

In this paper, a load-displacement relationship of the ACM brace was obtained from some timber frame experiments in order to apply the ACM bracing method to wooden-house seismic retrofitting work. A timber frame reinforced by the ACM brace has a significant high resistant strength and also has a possibility to apply the ACM bracing method to seismic retrofitting work for a lot of old wooden houses in Japan, which have a weak seismic performance against a large earthquake. Moreover, an e-plate of the ACM brace has several advantages of strong and light-weight feature and good durability in comparison with other materials.

This kind of seismic retrofitting work of wooden house using the ACM bracing method will lead to reduce a large earthquake damages in Japan and also improve a seismic retrofitting ratio for a lot of wooden houses with doubtful seismic performance as quickly as possible.

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REFERENCES


Study on Improvement of Performance of Lattice Shear Wall using Domestic Timber
- Experiment on rotational stiffness of Half-lap Joint Inserted Dowel -

TAMURA Yui¹, FURUKAWA Tadatoshi², FUJIMORI Shigeru³, YOSHIDA Takuya¹

ABSTRACT: To improve the initial stiffness of the lattice wall, several experiments using cruciform specimen with unit joint were conducted. In the first experiment, Application of metal dowel to the half-lap joints of the lattice shear wall were studied by experimental approach. As the result, the effect of dowel was not able to be checked clearly. In the second experiment, it experimented only about the joint which inserted dowel, and investigated only the effect by dowel. As the result, the effect of dowel has been checked. In Section 4, the calculation formula in consideration of shear deformation which is renewed was proposed. The third experiment was conducted to verify the validity of the proposed calculation formula. As the result, there was no big difference in the calculation formula proposed as the present calculation formula, the value and experimental value of the formula were comparatively well in agreement.

KEYWORDS: Lattice shear wall, Half-lap joint, Initial stiffness

1 INTRODUCTION
A wooden lattice is an architectural style which is commonly used in traditional tradesmen's house in Japan. The bulletin of The Ministry of Land, Infrastructure and Transport was revised in 2003, and then multiplier of bearing wall was given to the lattice shear wall according to the specification [1]. However, the multiplier of the lattice wall is 1.0 or less, the same level as plasterboard, and it is very small as compared with that of structural plywood. It is because the initial stiffness of the lattice shear wall is low, though it shows very tenacious characteristics. Therefore, in this study, to improve the initial stiffness of the lattice wall, application of metal dowel to the half-lap joints of the lattice shear wall are studied by experimental approach. Several experiments using cruciform specimen with unit joint are conducted. In this study, the influence of the presence and the shape of the dowel on the rotational stiffness of the connected joint are examined.

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2 FIRST EXPERIMENT
2.1 SPECIMENS
Several cruciform specimens were made of Japanese cedar with 90×90×1000mm and the joint gap set to two levels (0mm and 0.5mm). These cruciform specimens were divided into six types (TYPE-A to E and N) by presence, shape of the dowel, and joint size. The outline of the cruciform specimen is...
shown in Figure 2~4 and the types of the cruciform specimen is shown in Table 1.

![Figure 2: Outline of the cruciform specimen](image)

![Figure 3: Dimension of the cruciform specimen](image)

![Figure 4: Dowel](image)

![Figure 5: The experimental apparatus](image)

![Table 1: The types of the cruciform specimen](image)

<table>
<thead>
<tr>
<th>TYPE</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>N</th>
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<tr>
<td>Joint gap/</td>
<td>0.05/</td>
<td>0.05/</td>
<td>0.05/</td>
<td>0.05/</td>
<td>0.05/</td>
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<tr>
<td>Joint size [mm]</td>
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<td>90.5</td>
<td>90.5</td>
<td>90.5</td>
<td>90.0</td>
</tr>
<tr>
<td>Dowel</td>
<td>No dowel</td>
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2.2 EXPERIMENTAL PROCEDURE

Figure 5 shows the experimental apparatus. The cruciform specimen was set up in the frame device connected by the pin joints, the lateral deformation was given to the upper end of the column, force of the jack, displacement of the upper end of the column, and the rotational deformation of the connection joint were recorded.

Force was controlled by displacement of the upper end of the column. Among three specimens, two specimens were loaded until displacement was set to 36 mm monotonously. One specimen was repeatedly loaded until displacement was reached 2, 3, 4.5, 6, 9, 12, 18 mm. The number of repetitions in the same displacement is 2 times. And it was finally loaded until displacement was reached 36 mm. This way almost corresponded to the procedure provided in the building law of the standard act correspondingly.

![Figure 6: The relation between load and story drift](image)

2.3 RESULTS and DISCUSSION

Figure 6 shows the relation between load and story drift. Figure 7 shows the relation between moment and rotation angle of the joint.

In the present study, the possibility that the initial stiffness improves was shown by inserting the dowel in the connected joint. However, advantage by using dowel was still a little unclear. It should be needed the test methodology to be able to judge the effect of the dowel more clearly to evaluate the difference depending the type of the dowel.

![Figure 7: The relation between moment and rotation angle of the joint](image)
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3 SECOND EXPERIMENT

3.1 SPECIMENS

To evaluate the effect of the dowel quantitatively, the second experiment was carried out. The cruciform specimen was made of Japanese cedar with 90×45×1000mm. It was lattice shear wall without half-lap joints, and the dowel was inserted in the joints and the center was fixed by the pin. TYPE-C, D and E dowels at the first experiment were used in this experiment. Moreover, one TYPE-N which is a type without the dowel was also experimented. The outline of the cruciform specimen is shown in Figure 8 and the case of the cruciform specimen is shown in Table 2.

Experiment was carried out using the same experimental apparatus with the first experiment. All specimens were loaded until displacement was reached 36 mm.

Table 2: The case of the cruciform specimen

<table>
<thead>
<tr>
<th>TYPE</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
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<tbody>
<tr>
<td>dowel</td>
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3.2 RESULTS and DISCUSSION

Figure 9~12 shows the relation between moment and rotation angle of the joint of specimens. Figure 13 shows the average value of the moment of the joint at 1/120 radian. It should be noted that these values are adjusted by deducting the value of TYPE-N from the observed ones.

About TYPE-C, variation appeared greatly. However, TYPE-C and TYPE-E dowel achieved greater effectiveness to improve the rotational stiffness.

Figure 7: Moment-rotation angle of the joint

Figure 8: The outline of the cruciform specimen

Figure 9: Moment-rotation angle (TYPE-C)

Figure 10: Moment-rotation angle (TYPE-D)

Figure 11: Moment-rotation angle (TYPE-E)
4 THEORY

Now, the calculation formula of the relation between load and story drift is given to the lattice shear wall \(^2\).

\[
K = \frac{1}{\left(\frac{1}{K_f} + \frac{1}{K_j} + \frac{R_0}{P_y}\right)} \quad (1)
\]

\[
K_f = \frac{12EIu^2v^2}{H(uL+vH)} \quad (2)
\]

where \(K\) = total stiffness, \(K_f\) = frame stiffness, \(K_j\) = rotational stiffness at joint, \(P_y\) = yield load, \(R_0\) = rotation angle in initial slip, \(E\) = Young’s modulus, \(I\) = moment of inertia, \(H\) = Height of lattice wall, \(L\) = Length of lattice wall, \(u\) = the number of horizontal lattice and \(v\) = the number of vertical lattice.

However, this is the formula which evaluated only flexural rigidity and ignored the shear deformation. In this calculation formula, the moment of inertia should be reduced the half value of that calculated by full section of the members. The influence of shear deformation at the members of the lattice shear wall seems to be taken into account implicitly by the above manner. Therefore, the new formula containing shear deformation explicitly is proposed as follows.

\[
K_f = \frac{1}{H(uL+vH) + \frac{uL+vH}{12EIu^2v^2} + GALuv} \quad (3)
\]

where \(G\) = shear modulus and \(A\) = area.

In order to verify the validity of the proposed calculation formula, the experiment for comparing the influence of both the size of the cruciform specimen and the joint gap was conducted.

5 THIRD EXPERIMENT

5.1 SPECIMENS

The cruciform specimen with the half-lap joint was made of Japanese cedar with square section of 105×105 and length of 600mm and 1000mm. The joint size set to three levels (105, 106, and 107mm) which correspond joint gap of 0, 1, and 2mm. Four specimens for each type were prepared.

The outline of the cruciform specimen is shown in Figure 16~17 and the types of the cruciform specimen are shown in Table 3.
The experiment was carried out using the same experimental apparatus with the first and second experiments. All the specimens were repeatedly loaded until displacement was reached 2, 3, 4.5, 6, 9, 12, 18, 36 mm.

In Figure 18 and 19. The value of the proposed calculation formula is not mostly different from the present calculation formula. The initial slip had errors with the calculation formula and the experimental value. However, inclination of the graph of the calculation formula was almost near to the experimental value.

5.2 RESULTS and DISCUSSION

Figure 18 shows the relations between load and story drift of TYPE-L series. Figure 19 shows those of TYPE-S series. Figure 20 shows the relation between load and story drift of TYPE-L1. The calculated one by the proposed formula is also shown in Figure 20. Figure 21 shows the relation between load and story drift of TYPE-S1 with the calculated one.

Actual measured joint size of TYPE-x1 specimens were almost the same as those of TYPE-x0. Therefore, differences between x0 and x1 types could not be funded
6 CONCLUSIONS

To improve the initial stiffness of the lattice wall, several experiments using cruciform specimen with unit joint are conducted.

In the first experiment, Application of metal dowel to the half-lap joints of the lattice shear wall are studied by experimental approach. As the result, the effect of dowel was not able to be checked clearly.

In the second experiment, it experimented only about the joint which inserted dowel, and investigated only the effect by dowel. As the result, the effect of dowel has been checked about Type-C, E.

In Section 4, the calculation formula of the lattice shear wall was improved, and the calculation formula in consideration of shear modulus which is renewed was proposed.

The third experiment was conducted to verify the validity of the proposed calculation formula. The experiment for comparing the influence which the size of the cruciform specimen and a clearance has on the joint was conducted. As the result, there was no big difference in the calculation formula proposed as the present calculation formula, the value and experimental value of the formula were comparatively well in agreement.

Therefore, at a present stage, there is no telling which calculation formula can be evaluating the performance of the lattice shear wall clearly.

It is required to carry out experiment with more realistic lattice shear wall to verify about a calculation formula from now on.

REFERENCES


SEMIELMERIC METHOD TO PREDICT THE DISPLACEMENT CAPACITY AND RESISTANCE OF COLD-FORMED STEEL FRAME WOOD-PANEL SHEAR WALLS

Seyed Ali Moayed Alaee1, Timothy Sullivan2, Colin A. Rogers3, Roberto Nascimbene4

ABSTRACT: The cold-formed steel frame / wood panel shear wall system is a relatively new construction method, rooted from light wood framing concepts. Tests have shown that the seismic behaviour of these shear walls depends on their configuration and components. To date, design methods have largely been developed using a force based philosophy which relies on full-scale cyclic experimental tests. Conducting full-scale tests for various wall configurations is expensive and the experimental tests conducted so far have been respectively very limited. This study focuses on predicting the behaviour of cold-formed steel frame / wood panel walls using a semi-empirical method based on data obtained from fastener connection experimental tests. The semi-empirical model is applied to a series of shear walls with different aspect ratios and fastener schedule. It is shown that the predicted wall displacement and resistance agrees well with experimental test results. A brief discussion is also provided on how this simplified method for the prediction of the wall response can be utilised within a direct displacement-based seismic design procedure.

KEYWORDS: Cold-formed steel, Wood-panel, Shear wall, Direct Displacement Based Seismic Design

1 INTRODUCTION

Over the past decades, cold-formed steel frame / wood-panel (CFSFWP) systems (Figure 1) have seen increased usage as the structural framing elements for low-rise buildings due to their competitiveness in relation to conventional construction systems and the increased design and construction flexibility they offer. The walls in such systems have the primary functions of carrying vertical and lateral loads. They are composed of cold-formed steel profiles (studs and tracks) and wood based panels, such as Oriented Strand Board, OSB, and Plywood, which are connected to the steel frame by fasteners (usually screws).

Current seismic design procedures for CFSFWP shear walls [1,2] rely on force-based methods and on relatively few results of full-scale tests on walls with different details. The tests undertaken have illustrated that the seismic behaviour of these shear walls depends on several factors such as the panel thickness, type and properties, the fastener spacing and the steel properties [4-9]. The panels that have been used in most of the tests to date are specific to the North American market and were constructed of wood species that may not be

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available in other countries. Therefore, the validity of using specific experimental results for countries where the wall systems are realised with different wood panel types, for instance, is questionable. Costly full-scale shear wall experimental campaigns could be deemed necessary in every country wishing to utilise this framing system. The use of analytical models in place of wall tests in the development of design provisions could reduce the number of full-scale tests needed to evaluate wall behaviour when new panel types or wall configurations are being considered.

In this study a number of key points that characterise the force-displacement response of CFSFWP wall systems were identified using the results of wood panel to steel frame fastener connection tests, without needing to undertake costly tests of whole wall specimens. To do this, a semi-empirical method that relies on the connection data is presented for CFSFWP walls and predictions of their force-displacement response are provided. The validity of the model is illustrated by comparing the predictions with experimental shear wall results obtained from the literature.

2 CYCLIC BEHAVIOUR OF CFSFWP SHEAR WALLS

Past tests [4-9] showed that the overall behaviour of a steel-frame – wood-panel shear wall can be considered to be a function of the connection performance if other structural components are protected with proper design. As such, the hold-downs, hold-down anchors, shear anchors, tracks, field fasteners and the steel-to-steel framing connections rarely suffer any type of permanent damage.

A nonlinear resistance vs. deflection behaviour and a substantial amount of pinching in the resulting hysteresis graphs is typically observed for reversed cyclic tests. This behaviour is attributed to the permanent bearing – pull-through type damage to the wood sheathing at the connection locations. The ability of the screws to tilt back and forth in the thin steel framing is considered as the main reason why only limited damage occurs in the fasteners themselves and in the steel frame.

The height to length aspect ratio is the next important factor governing the behaviour of these walls. The maximum shear capacity of walls ranging in length from 1220 mm to 2440 mm is developed at similar lateral displacement levels, while the 610 mm long walls require significantly larger displacements prior to reaching their ultimate shear resistance. The shear strength appreciably decreases when the aspect ratio increases from 2 to 4.

The fastener spacing plays an important role where installing the sheathing screws at closer distances significantly increases the shear strength, and stiffness both in monotonic and cyclic loading. Another important observation is that the wall framing deforms into a parallelogram and the deformation of the wood panels is mainly due to a rigid body rotation. Negligible shear deformation of the wood panels has been observed to occur.

The dissipated energy significantly increases with the displacement up until the peak load. After this point, the dissipated energy remains relatively unchanged for higher displacements up until the onset of failure. The energy dissipation is approximately proportional to the number of perimeter screws, with the exception of walls which fail by local buckling of the compression chord rather than by bearing – tilting – pullout of the sheathing connections. At each displacement level, the dissipated energy decreases in subsequent loops to the same peak displacement, reflecting some strength and stiffness degradation caused by cyclic loading.

It is observed that prior to ultimate failure, the wall specimens can sustain large inelastic deformation cycles with limited strength degradation. However, it is important to preclude failure of chord studs as an unfavourable failure mode because, in many situations, in addition to resisting a lateral load the wall also supports gravity loads. Therefore, in the cases in which dense fasteners schedules are required, thicker back-to-back coupled end studs for the walls should be used to allow the shear strength of the panel-to-frame connections to be fully developed. Moreover, a limitation should be respected for some parameters such as steel thickness. For instance, 4.2 mm screws should be limited to 1.09 mm thick framing since the desirable behaviour (local bearing failure of the wood at the connections) of the screws may change to a brittle failure mode of fracture in shear when 1.37 mm thick frames are used.

Okasha and Rogers conducted tests on fastener connections of CFSFWP walls [9] and concluded that the sheathing type had a significant effect on the connection strength and energy dissipation. It was observed that the connection strength and displacement capacity increased with increasing panel thickness and that the connections with higher edge distance provided higher strength and ductility. They concluded that the connections with edge distances less than 12.5 mm suffer from significant degradation in strength and energy dissipation due to the change in failure mode from bearing to edge tear out.

3 THE SEMI-EMPIRICAL 3-POINT FORCE -DISPLACEMENT CURVE

Different analytical models have been developed to predict the behaviour of the light wood framed shear walls based on the results of fastener connection tests [10-13]. Also to a more limited extent there are analytical models proposed for CFSFWP shear walls [1, 2, 13]. In these analytical models the shear modulus of the sheathing panel, G, should be known a priori by conducting tests. Since the expressions were established based on the full-scale tests of a selected number of sheathing types, for other types of plywood or OSB adjustments to the coefficients based on material tests may be necessary.

This study aims to provide a simple method to predict the yield displacement and the post elastic performance of CFSFWP shear walls constructed using the platform.
framing technique, including the deformation at the maximum lateral resistance.

It is necessary that the analytical model account for the most important factors that define the seismic behaviour of CFSFWP shear walls, i.e. the aspect ratio, the fastener spacing and the sheathing characteristics (type, thickness & material properties), as well as the less influential parameters including the screw size, the steel thickness and the steel grade.

Many of the assumptions made by McCutcheon [10] for wood framed walls can be considered appropriate for the deformed frame in a CFSFWP shear wall. Some of the important assumptions made in the considered wall distortion model (Figure 2) and in determining the wall resistances are noted below.

- The frame becomes a parallelogram while the shape of the sheathing panel remains unchanged. The panel will, rotate in its own plane. The studs and tracks retain their section shape and straightness at least until reaching the peak resistance of the wall, (Post peak behaviour of the walls is not addressed in the proposed model).
- The load-deflection curve is nonlinear for a single screw.
- The shear wall is assumed to be designed and constructed on a good engineering basis. As such, the wall is anchored to the supporting structure. No significant slip and rotation in the bottom track or significant uplift of the chords is expected.
- The contribution of the bending of the panel and panel shear deformation to the wall total deformation may be neglected (as opposed to McCutcheon (1985) assumption [10] for reasons explained in Moayed Alaee et al. [14]).
- The hysteretic energy is principally provided by the distortion of the sheathing screw connections.
- The sheathing-to-frame connections have the same capacity and stiffness in all in-plane directions (for reasons explained in Moayed Alaee et al. [14]).
- The screws are spaced symmetrically and the panel is parallel to the frame.
- The lower corner fastener at the compression side of the panel distorts along the line of the sheathing’s diagonal and both lower corner fasteners have the same relative displacements (Figure 2).

A comprehensive discussion of the above assumptions is provided in the thesis by Moayed Alaee [14].

By considering the above assumptions and distortion model, Equation (1) is proposed for predicting the wall displacement at the desired limit state, LS:

$$\Delta_w = 2\rho \frac{D}{B} a_0 = 2\rho \Delta_i \sqrt{1 + \frac{H^2}{B^2}}$$

(1)

where $\Delta_w$ is the in-plane lateral displacement of the wall at the desired limit state, $D$ is the diagonal length of the sheathing panel, $B$ and $H$ are the width and height of the sheathing panel, respectively, $\Delta_i$ is the relative displacement of the lower corner screws in the panel at the desired limit state and $\rho$ is a correction factor calibrated, using the existing tests [4-9], to take into account the effect of the dense fastener schedule (Table 1). By using Equation (1) the lateral displacement of the wall ($\Delta_w$) is related to the relative displacement between the frame and the panel at the lower corner of the wall ($\Delta_i$).

The displacements corresponding to important limit states, which are later used as pivot points to form the predicted force-deflection curve of the wall, are identified in the following sections.

**Figure 2: Distorted shape of the frame and the sheathing of the shear wall**

**Table 1: Correction factor, $\rho$, for fastener schedule**

<table>
<thead>
<tr>
<th>Fastener schedule</th>
<th>Plywood</th>
<th>OSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 mm</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>100 mm</td>
<td>1.1</td>
<td>1.05</td>
</tr>
<tr>
<td>75 mm</td>
<td>1.15</td>
<td>1.1</td>
</tr>
<tr>
<td>50 mm</td>
<td>1.15</td>
<td>1.15</td>
</tr>
</tbody>
</table>

**3.1 ULTIMATE LIMIT STATE, ULS**

It is assumed that the wall reaches its maximum strength when the corner fastener fails. Failure of a single fastener connection is defined as the point where the resistance of the fastener connection drops to 2/3 of the peak resistance, i.e. $0.67f_u$, where $f_u$ is the maximum resistance of the fastener connection obtained from relevant experimental fastener connection cyclic test data (as shown, for example, in Figure 3).

The wall displacement corresponding to the maximum lateral resistance of the wall is considered here as the wall displacement for the ultimate limit state (ULS), $\Delta_w_{uls}$.

For a force-displacement curve obtained from a fastener experimental test, $\Delta_{t,uls}$ is defined as the displacement corresponding to failure of the connection. $\Delta_{t,uls}$ should be obtained from the average of the envelopes of the negative and positive quadrants of the cyclic force-displacement test curve of the sheathing-to-framing fastener connection.
Therefore, the displacement corresponding to the maximum strength of the wall, which is also considered to be the displacement of the wall at the ULS, can be estimated from Equation (1) as:

\[ \Delta_{w,ULS} = 2\rho \frac{D}{b} \Delta_{t,ULS} = 2\rho \Delta_{t,ULS} \cdot \sqrt{1 + \frac{H^2}{b^2}} \]  

(2)

3.2 DAMAGE LIMITATION LIMIT STATE, DL LS:
Displacement at the wall corner fastener, \( \Delta_i \), for the damage limitation (DL) limit state (LS), \( \Delta_{i,DL} \), is defined here as the displacement corresponding to maximum strength of the single fastener connection, \( f_u \), that should be obtained from the average of the peak force envelope from the positive and negative quadrants of the force-displacement curve obtained from cyclic test of a fastener connection. As such, the wall displacement at the DL LS is determined from Equation (1) as:

\[ \Delta_{W,DL} = 2\rho \frac{D}{b} \Delta_{t,DL} = 2\rho \Delta_{t,DL} \cdot \sqrt{1 + \frac{H^2}{b^2}} \]  

(3)

3.3 THE WALL (INITIAL) YIELD DISPLACEMENT:
Unlike steel structures, a precise yield point is not distinguishable in CFSFWP shear wall systems. However, it is important to have a consistent method of defining the yield point of the walls for seismic design since most seismic design methodologies require estimates of ductility demand to relate elastic spectral ordinates to inelastic response.

The point in which a major decline in stiffness is observed in a load-displacement curve of CFSFWP shear walls is considered as the (initial) yield point here. It will be the first pivot point, out of three pivots, that define the tri-linear load-displacement curve of the wall. To obtain this point it is assumed that the displacement, \( \Delta_i \), of the corner fastener is equal to the yield displacement of the idealized bilinear load-displacement curve of the sheathing fastener connection tests, \( \Delta_{y,c} \), that is obtained based on EEEP method, which approximates the nonlinear load-displacement curve of the connection by an idealised bilinear equivalent energy elastic-plastic (EEEP) curve [13]. This fastener yield displacement is then inserted into Equation (1) to determine the yield displacement of the wall, \( \Delta_{w,y} \).

\[ \Delta_{w,y} = 2\rho \frac{D}{b} \Delta_{y,c} \]  

(4)

3.4 Predicting the wall resistance at the limit states
By knowing the wall displacement at each limit state from Equation (1) the geometrical parameters of the distorted wall demonstrated in Figure 2 can be determined and then the relative displacement between the panel and the frame for each screw of the wall is determined as shown in Equation (5).

\[ \Delta_z = \frac{E \Delta_w}{D} \left[ (\frac{X}{B} - 0.5) \frac{H}{D} \right]^2 + \left( (\frac{Y}{B} - 0.5) \frac{B}{D} \right)^2 \]  

(5)

From the load-displacement curve of the relevant fastener experimental test the connection forces can also be found. Subsequently, an energy approach is used to determine the wall resistance corresponding to the wall drift of \( \Delta_n \) as explained next. The envelope curve of the force-displacement hysteresis loops of a sheathing-to-framing fastener connection cyclic test is linearized into 4 or 5 lines (as deemed necessary to obtain a good fit of the hysteresis loops envelope) and the average area of this linearized envelope curve in both the positive and negative quadrants should be considered as the energy absorbed by the single fastener, \( I_s \), which is determined from Equation (6):

\[ I_s = \frac{1}{2} \int_{-\Delta_k}^{\Delta_k} (F \cdot d \Delta) \]  

(6)

\( F \), the wall resistance at wall displacement of \( \Delta_w \) is then determined from Equation (7):

\[ F = \frac{N}{2 \Delta_w} \sum_{i=1}^{n} I_{s,i} \]  

(7)

where \( n \) is the number of the screws in the wall, \( N \) is number of panels in the shear wall, \( \Delta_{s,i} \) is the displacement in the \( i^{th} \) screw, calculated from Equation (5), and \( F_{s,i} \) is the force at displacement \( \Delta_{s,i} \), determined from the force-displacement curve of the fastener connection test. The internal work is determined from the numerical integration of the average of positive and negative quadrants area of force-displacement curve of all of the screws.

4 PERFORMANCE OF THE PROPOSED METHOD
The three points defined in the previous section for the wall displacement at yield, the displacement at the damage limitation state and the displacement at the ultimate limit state are considered as pivot points that can be used to predict the load-displacement curve of a shear wall. This section compares values predicted by the simplified approach described in the previous section, with values obtained from experimental data.
Table 2 compares the ratio of values predicted by the proposed model and the displacement and resistances obtained from experimental test data (obtained from various sources cited at the foot of the table).

<table>
<thead>
<tr>
<th>Test</th>
<th>Disp. Ratio At Wall Peak Resistance</th>
<th>Wall resistance Ratio DL LS</th>
<th>Wall resistance Ratio ULS</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 M*</td>
<td>0.89</td>
<td>0.99</td>
<td>0.92</td>
</tr>
<tr>
<td>8 M*</td>
<td>0.90</td>
<td>0.99</td>
<td>0.94</td>
</tr>
<tr>
<td>10 M*</td>
<td>0.97</td>
<td>0.97</td>
<td>0.80</td>
</tr>
<tr>
<td>16 M*</td>
<td>0.99</td>
<td>1.00</td>
<td>0.94</td>
</tr>
<tr>
<td>20 M*</td>
<td>0.95</td>
<td>0.97</td>
<td>0.94</td>
</tr>
<tr>
<td>22 M*</td>
<td>0.96</td>
<td>0.93</td>
<td>0.96</td>
</tr>
<tr>
<td>24 M*</td>
<td>1.13</td>
<td>0.87</td>
<td>0.94</td>
</tr>
<tr>
<td>26 M*</td>
<td>1.17</td>
<td>0.86</td>
<td>0.89</td>
</tr>
<tr>
<td>28 M*</td>
<td>0.97</td>
<td>0.87</td>
<td>0.83</td>
</tr>
<tr>
<td>30 M*</td>
<td>0.88</td>
<td>0.97</td>
<td>0.85</td>
</tr>
<tr>
<td>32 M*</td>
<td>0.93</td>
<td>0.94</td>
<td>0.79</td>
</tr>
<tr>
<td>34 M*</td>
<td>0.88</td>
<td>0.87</td>
<td>0.73</td>
</tr>
<tr>
<td>1 F*</td>
<td>1.02</td>
<td>0.91</td>
<td>0.99</td>
</tr>
<tr>
<td>14 C*</td>
<td>1.13</td>
<td>0.84</td>
<td>0.84</td>
</tr>
<tr>
<td>16 C*</td>
<td>1.09</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>17 C*</td>
<td>1.20</td>
<td>1.00</td>
<td>0.98</td>
</tr>
<tr>
<td>19 C*</td>
<td>1.08</td>
<td>1.21</td>
<td>1.04</td>
</tr>
<tr>
<td>A1&amp;A2 S*</td>
<td>0.80</td>
<td>0.92</td>
<td>0.76</td>
</tr>
<tr>
<td>A3&amp;A4 S*</td>
<td>0.82</td>
<td>1.16</td>
<td>0.94</td>
</tr>
<tr>
<td>A5&amp;A6 S*</td>
<td>1.00</td>
<td>0.98</td>
<td>0.87</td>
</tr>
<tr>
<td>A7&amp;A8 S*</td>
<td>0.76</td>
<td>1.21</td>
<td>0.88</td>
</tr>
<tr>
<td>B1&amp;B2 S*</td>
<td>0.91</td>
<td>0.85</td>
<td>0.78</td>
</tr>
<tr>
<td>OSBI FD*</td>
<td>1.22</td>
<td>0.90</td>
<td>0.87</td>
</tr>
<tr>
<td>Average</td>
<td>0.98</td>
<td>0.96</td>
<td>0.88</td>
</tr>
<tr>
<td>STD. DEV.</td>
<td>0.13</td>
<td>0.12</td>
<td>0.08</td>
</tr>
</tbody>
</table>


The results presented in Table 2 indicate that the proposed analytical model predicts the experimentally observed forces and displacements well. In making comparisons with experimental data, care was taken to ensure that the experimental fastener connection test that has similar characteristics of the wall used in full-scale test, in terms of panel type and thickness, screw type and size, edge distance, etc. More discussion of this point can be found in Moayed Alaae [14]. In Table 2, the higher deviation of the ratios for the tests carried out by Serrette et al. [8] and Fulop et al. [6] can be attributed to the fact that there were some differences for sheathing type and test protocol between the fastener test used for the model and the actual full-scale test, due to the limited number of existing fastener connection tests for CFSFWP walls.

Figure 4 compares the experimental test results of test no. 8, carried out in McGill University, with the predicted tri-linear load-displacement curve, developed using Okasha no. 38 fastener connection test data. The test characteristics of the full-scale and fastener test were similar. In both tests the CSP plywood is 12.5mm thick and the steel is of grade 230 having 1.1mm thickness.

5 POTENTIAL APPLICATIONS FOR DISPLACEMENT-BASED SEISMIC DESIGN

For what regards seismic design, the importance of deformation, rather than strength, in assessing seismic performance is apparent. As argued by Priestley et al. [15] if the design objective is to control the damage under a given level of seismic attack, it would be more reasonable to design the structures to meet a desired displacement under the design seismic intensity, and this has motivated the development of the Direct Displacement-Based Design approach (DDBD) [15].

To develop the DDBD procedure for cold-formed steel-frame / wood-panel shear wall structures, three important parameters are needed: (i) a means of determining the displacement corresponding to important limit states for different wall configurations; (ii) expressions for the equivalent viscous damping, EVD, of CFSFWP shear walls and (iii) expressions for the design displacement profile and general design procedure for CFSFWP shear wall systems. In this work, significant progress has been made towards satisfaction of the first of these tasks.

In particular, note that within the DDBD procedure the design displacement, Δy, for the performance level under consideration, can usually be based either on code-specific drift limits or material strain limits. In a structure composed of CFSFWP shear walls, due to the very complicated response at the level of sheathing-to-frame fastener connections, it is not easy to define a material strain limit. However, as it was proposed here, it is possible to estimate important drift limits of the walls and the yield displacement, Δy, using the relative displacement between the frame and the sheathing-to-frame corner fasteners and the associated fastener connection test load-displacement data and consequently determine the design displacement.
6 CONCLUSIONS

A simple analytical approach has been proposed for the prediction of the displacements corresponding to important limit states of CFSFWP shear walls. The proposed approach uses sheathing-to-framing fastener connection test results and relates the wall displacement to the displacement of the corner fastener connection. The analytical procedure has been developed to predict critical pivot points of the CFSFWP wall force-displacement curve, using cyclic tests on corresponding sheathing-to-framing screw connections. The critical points were taken to be the yield, damage limitation and ultimate limit state of the shear walls. The wall yield displacement is assumed to correspond to the displacement at which the corner connection yields, obtained using an Equivalent Energy Elastic Plastic (EEEP) bilinear representation of the load-displacement curve of the fasteners. The displacement when the maximum resistance of the fastener is reached was taken as the fastener pivot displacement for determining the displacement at the damage limitation state of the walls. The displacement when the fastener resistance falls to 2/3 of the peak value was considered as the pivot displacement corresponding to the displacement at the wall peak resistance. These pivot points were used to predict the tri-linear idealized load-displacement curve of the wall. Good agreement was observed between the experimentally obtained displacements at the peak resistance of CFSFWP walls and the displacements predicted by the proposed method. Based on an energy approach the resistance of the wall corresponding to each limit state was obtained and predictions were found to correlate well with experimental results.

The proposed method for the calculation of critical force and displacement points can be used to determine the behaviour of shear walls with different configurations, i.e. aspect ratio and fastener spacing, and even other less influential aspects such as steel thickness and grade, because the effect of these aspects on displacement capacity and resistance of the wall is reflected in the fastener connection tests. A benefit of this approach is that the need for extensive full scale shear wall experiments can be significantly reduced. Conducting fastener connection tests is much less expensive; furthermore it is possible to use existing fastener test results.

In closing, it is considered that the presented method could be very useful for the Direct Displacement Based design of CFSFWP shear walls by offering a means of determining the displacement corresponding to important limit states for different wall configurations.

ACKNOWLEDGEMENT

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REFERENCES

SEISMIC PERFORMANCE OF MUD-WALLS WITH SILL BASED ON FULL-SCALE CYCLIC LOADING TESTS

Hiroyuki Nakaji¹, Teruo Kamada², Masami Gotou³, Koji Yamada⁴, Yoshiyuki Suzuki⁵

ABSTRACT: In this study, static shear loading tests on twelve types of mud-walls and ten types of timber frameworks were conducted as a part of a research project to establish structural design method for traditional wooden buildings. Based on the test results and numerical analyses, the superposition principle was considered to examine the restoring force characteristics of frames combining structural elements. The P-Delta effect was also considered in the restoring force characteristics. It was found that in some combination cases of structural elements, the superposition principle could be applied in both linear and non-linear region of mud-walls’ restoring force characteristics.

KEYWORDS: Mud-walls, Restoring force characteristics, Superposition principle, Structural design

1 INTRODUCTION

In Japan, buildings or houses have been built with timber materials for a long time. Structural design method for the ordinary wooden houses was revised after Hyogoken-Nanbu Earthquake in 1995. On the other hand, structural design method for traditional wooden buildings is difficult to use for most of the structural designers because of lack of experimental and analytical researches.

In such wooden buildings built with traditional construction method, mud-plastered walls are used as bearing walls against strong earthquakes. Through recent researches after Hanshin-Awaji disaster in 1995, the mud-wall’s seismic capacity has been examined again [1] and it is now known that these traditional bearing walls have large deformability while load capacity is low. This characteristic of energy dissipation must be considered in seismic design of traditional timber buildings.

In this study, static cyclic shear loading tests on twelve types of mud-walls and ten types of timber frameworks were conducted as a part of a research project to establish structural design method for traditional wooden buildings. Numerical analyses were also executed on the restoring force characteristics of mud-walls.

2 FULL-SCALE TESTS OF MUD-WALLS WITH SILL

2.1 SPECIMENS

In 2010, twelve types of mud-walls were prepared and tested to investigate the seismic performance of them. Figure 1 shows the outline of mud-wall specimens. The mud-wall specimens MWD-1, MWD-2, and MWD-3 are mud-walls without opening. MWD-4, 5 and 6 are with opening. And MWD-7, MWD-8, MWD-9, MWD-10, MWD-11 and MWD-12 are variations of combination of MWD-1, MWD-4, MWD-5 and MWD-6. All specimens have Dodai (a sill) to connect the bottom ends of columns. The types of length of mud-walls are 1P (910 mm), 2P (1820 mm) and 4P (3640 mm). MWD-1 is 1P, MWD-2, 4, 5 and 6 are 2P, and others are 4P.

Mud-walls were plastered with mud produced in Kyoto. Komaidake (bamboo) was tied with rice straw ropes, and then Arakabe (rough wall, the first layer of mud-wall) and Nakanuri (a middle layer of plaster applied between the base and finish coats of a mud-plastered wall) were applied. The inside measurement between each Komaidake was 45mm. The thickness of mud-wall was 60 mm. In 2011, in addition to those mud-walls, the timber framework specimens were also tested. The timber framework specimens were MWDF-0, MWDF-1,
MWDF-2, MWDF-3, MWDF-6, MWDF-7, MWDF-8, MWDF-9, MWDF-11 and MWDF-12, MWDF-0, 1, 2, 6 are 2P and MWDF-3, 7, 8, 9, 11 and 12 are 4P. Figure 2 shows timber framework specimens. The specimen numbers are corresponding to those of MWD. MWDF-0, which is built without Nuki, was prepared to examine the effect of Nuki on the restoring force characteristics of timber framework. MWDF-4 and MWDF-5 were not tested because it was expected that their restoring force characteristics are not so different from that of MWDF-6. The common settings for both MWD and MWDF specimens were as follows. Column joints were made using Nagahozo (mortise-tenon) and strengthened by Komisen (cotter-pin). Figure 3 shows the detail of column joints. The frameworks were made with Sugi (Japanese cedar). The section size of all cedar columns and a sill was 120 mm × 120 mm. The section size of Nuki (tie beam) was 18 mm × 105 mm.

**Figure 1:** Mud-wall specimens with sill

MWDF-0  MWDF-1  MWDF-2  MWDF-6  MWDF-3

MWDF-7  MWDF-8  MWDF-9  MWDF-11  MWDF-12

**Figure 2:** Timber framework specimens with sill

MWDF-0

MWDF-1

MWDF-2

MWDF-6

MWDF-3

MWDF-7

MWDF-8

MWDF-9

MWDF-11

MWDF-12

**Figure 3:** Detail of joints between column and sill

2.2 SHEAR CYCLIC LOADING TESTS

Shear cyclic loading tests of the 12 types of mud-walls and 10 types of timber frameworks were carried out in Tottori University of Environmental Studies, Fukuyama University and Prefectural University of Kumamoto. To avoid a pull-out failure at the column-sill joints, the axial force was applied to columns by using about 20kN of steel weight. Horizontal cyclic force is applied at the level of the upper beam of a specimen. The deformation angle was increased in stages from 1/480 rad to more than 1/10 rad, and each deformation was cyclically produced twice. Load and displacements were measured to obtain the load-deformation curves of specimens.

2.3 TESTS RESULTS

In further discussion, so-called P-Delta effect is removed from the load using following equation.

\[ P' = P + W \times \tan \theta \]

Here,
- \( P \): measured load (kN)
- \( W \): weight of steel (kN)
- \( \theta \): deformation angle (rad)

Figure 4 and 5 show the load-deformation envelope curves. The horizontal axis is the deformation angle \( \theta \) (rad) and the vertical axis is \( P' \) (kN). The envelope curves of MWDF-0 and MWDF-2 are almost same, and
it is recognized that Nuki has few effect on the restoring force characteristics of timber frameworks.

From Figure 4(a), the maximum load $P_{\text{max}}$ of MWD-2 is higher than that of others but the declination of load is also large. From Figure 4(b), MWD-3’s deterioration is larger than others.

From Figure 5(a), MWD F-1’s restoring force is larger than others because of the number of columns. MWD F-1 has 3 columns while others have two columns. From Figure 5(b), the envelope curves are classified into two groups: low and high bearing load. In the case of higher bearing force specimens, the number of columns is more than that of lower ones.

2.4 Failure patterns

The failure or break of the MWD specimens was that the mud layer was delaminated after many cracks in the wall. Photo 1 and 2 show the final damage of some MWD and MWDF specimens respectively.

3 CONSIDERATION ON SUPERPOSITION PRINCIPLE

To evaluate the restoring force characteristics of frames combining structural elements, a superposition principle is often assumed in wooden structural design code in Japan. The restoring force of combined structural elements is obtained by adding simply up each element’s restoring force. In this section, based on the test results described above, it is examined whether the superposition principle is applicable on the restoring force characteristics of traditional mud-wall frames variously combining mud-wall and timber framework elements. Assuming that MWDF-4 and MWDF-5 are equal to MWDF-6, the following seven patterns are to be discussed:

(1) MWD-2 = (MWD-1 – MWDF-1) × 2 + MWDF-2
(2) MWD-3 = (MWD-3 – MWDF-3) × 2 + MWDF-3
(3) MWD-6 = (MWD-4 – MWDF-6) + (MWD-5 – MWDF-6) + MWDF-6
(4) MWD-7 = (MWD-1 – MWDF-1) × 2 + (MWD-6 – MWDF-6) + MWDF-7
(5) MWD-8 = (MWD-1 – MWDF-1) × 2 + (MWD-4 – MWDF-4) + MWDF-8
(6) MWD-9 = (MWD-6 – MWDF-6) × 2 + MWDF-9
(7) MWD-12 = (MWD-4 – MWDF-6)× 2 + MWDF-12

Figure 6 shows the comparison of envelope curves in above seven cases. Table 1 shows the comparison of
maximum load ($P_{\text{max}}$) of them. From Table 1 and Figure 6, the superposition principle seems applicable even in the inelastic region. In five cases except (2) and (6), $P_{\text{max}}$ of test results are more than summations, that is, the use of superposition principle in structural design can be safer. In the case of (6), the envelope curves agree well.

**Table 1: Comparison of maximum loads of specimens (kN)**

<table>
<thead>
<tr>
<th>Test result</th>
<th>Superposition principle</th>
</tr>
</thead>
<tbody>
<tr>
<td>MWD-2</td>
<td>11.85 (1) 10.73</td>
</tr>
<tr>
<td>MWD-3</td>
<td>18.82 (2) 22.90</td>
</tr>
<tr>
<td>MWD-6</td>
<td>6.47 (3) 6.09</td>
</tr>
<tr>
<td>MWD-7</td>
<td>18.48 (4) 13.73</td>
</tr>
<tr>
<td>MWD-8</td>
<td>14.75 (5) 11.12</td>
</tr>
<tr>
<td>MWD-9</td>
<td>11.28 (6) 11.58</td>
</tr>
<tr>
<td>MWD-12</td>
<td>6.12 (7) 5.68</td>
</tr>
</tbody>
</table>

To complement and modify the superposition principle, numerical analyses were also examined. A modified superposition principle is presented to evaluate the restoring force characteristics of combined mud-walls. Figure 7 shows some analytical results. In the case of MWD-12, the load is calculated by twice of MWD-4’s analytical result. It is also recognized the modified superposition principle can be applied.

4 CONCLUSIONS

The static cyclic shear loading tests on twelve types of mud-walls and ten types of timber frameworks were carried out. From the test results, it was recognized that the restoring force characteristics of mud-walls could be estimated by using superposition principle. This estimation method will be useful in the structural design of traditional wooden buildings. It is important to continue the shear loading tests on various mud-walls in order to make sure the structural design of the traditional wooden buildings.

ACKNOWLEDGEMENTS

This study was carried out as a part of a research committee supported by the Japanese Ministry of Land, Infrastructure, Transport and Tourism. Authors express our gratitude for its support, members and staffs of the committee, and students to help so many experiments.

REFERENCES

DYNAMIC PROPERTIES OF A TRADITIONAL COMPLEX BRACKET SETS IN AN ORIENTAL TEMPLE

Wen-Shao Chang1, Takehiro Wakita2, Akihisa Kitamori3, Kohei Komatsu4, Yasuo Kataoka5, Min-Fu Hsu6

ABSTRACT: The aim of this paper is to investigate the dynamic properties of traditional complex bracket sets in an oriental timber temple by shaking table tests. The experimental programme considered the roof weight and the intensity of ground excitation. The results showed that the whole system can be simplified as a lump mass system with simple degrees of freedom, and the system performed significant decrease in natural frequency under the condition of combined heavy roof weight and strong ground acceleration.

KEYWORDS: Instructions to authors, Proceedings, WCTE 2012

1 INTRODUCTION

The complex bracket set in an oriental temple is an important architectural characteristic of traditional timber buildings in Asia. Limited research work has been done on the dynamic characteristics of this type of construction. Static and dynamic tests have been carried out by Fujita et al. to investigate the impact of Japanese complex bracket set on the entire structure and observed that the displacement will be reduced if the natural frequency of the complex bracket set is similar or close to that of the entire structure [1]. Fujita et al. [2] studied the structural behaviour of complex bracket set itself, and create analytical model. The connections were modeled as linear spring, and stiffness was obtained from theoretical model. Pang et al. have conducted ambient vibration measurement on a traditional Chinese timber building and use the vibration measurement data to verify the Finite Element Model (FEM) [3, 4]. The ways that structural members are connected vary from place to place. This paper aims at investigation of dynamic characteristics of a complex bracket set in a 100-year-old Taiwanese temple. The parameters considered include the roof weight, peak ground acceleration.

2 MATERIALS AND METHODS

To investigate the dynamic properties of a complex bracket set in a Taiwanese temple, the specimens were prepared in and shipped from Taiwan and re-assembled by Taiwanese traditional carpenters in Kyoto University, and the shaking table tests were carried out at Chubu University, Japan. To enable us to establish a model to analyze the structural behavior of the complex bracket set, static tests were carried out to obtain the rotational stiffness of the connections (10).

2.1 STATIC TESTS

2.1.1 SPECIMENS

To obtain the stiffness of individual complex unit, a complex bracket unit was fabricated as shown in Figure 1. The vertical members used were recycled from a 100-year-old temple, whilst the horizontal members are newly fabricated by traditional carpenters.
2.1.2 TEST PROCEDURE
To investigate how vertical load affects the lateral stiffness of the structure, vertical loads of 12 and 16kN were applied on the structure to simulate the roof weight. Horizontal cyclic load was then imposed on the structure until 1% of relative displacement of the structure was reached.

2.2 DYNAMIC TESTS
2.2.1 SPECIMENS
The complex bracket set tested formed the entrance hall of the temple, which was built in early 1910s and dismantled in 1990s. After the temple was dismantled, the components were collected and preserved in a conditioned room before they are re-assembled for the tests. The material used in the old temple is Taiwan red cypress (Chamaecyparis formosensis). The newly made components with same species were used to replace those lost components to form a whole specimen.

2.2.2 TEST PROCEDURE
Firstly the system identification were carried out with different roof weight (0, 4.9kN, 9.8kN and 14.7kN) under different level of white noise (20, 40 and 60 gal); secondly the shaking table tests were conducted using seismic records of Chi-Chi earthquake that occurred in Taiwan in 1999. The time-history record is shown in Figure 2, and the setup of the specimen is shown in Figure 3.

The connection stiffness was calculated from the load-displacement relation of static tests, i.e. the stiffness of the entire complex bracket set. Commercialised structural analysis programme, ROBOT, was used to obtain the stiffness of the connection. The results show that the rotational stiffness of the connection increases with the increase of the roof weight. The rotational stiffness of the connection under 14.7 kN roof weight, 42.7 kNm/rad/, was obtained from interpolation of 12 and 16 kN roof weight static tests, which will be used in the analytical model.

3 RESULTS AND DISCUSSION
3.1 THE CONNECTION STIFFNESS VERSUS ROOF WEIGHT
The connection stiffness was calculated from the load-displacement relation of static tests, i.e. the stiffness of the entire complex bracket set. Commercialised structural analysis programme, ROBOT, was used to obtain the stiffness of the connection. The results show that the rotational stiffness of the connection increases with the increase of the roof weight. The rotational stiffness of the connection under 14.7 kN roof weight, 42.7 kNm/rad/, was obtained from interpolation of 12 and 16 kN roof weight static tests, which will be used in the analytical model.

3.2 DYNAMIC PROPERTIES
The vibration of the specimens were recorded by data acquisition system, then calculate the natural frequency of the structure by Fast Fourier Transform (FFT). A typical FFT spectrum is illustrated in Figure 4.
Table 1 shows the results of system identifications, and the damping ratios of the structure under different roof weight and excitation. Figure 5 shows the relationship between natural frequency and number of steel plates, we can learn that the trend that the natural frequency decreases with increase of steel plate (roof weight).

Table 1: The dynamic properties of complex bracket set

<table>
<thead>
<tr>
<th>Roof weight (kN)</th>
<th>20 gal</th>
<th>40 gal</th>
<th>60 gal</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>8.94</td>
<td>8.94</td>
<td>8.94</td>
</tr>
<tr>
<td>4.9</td>
<td>8.59</td>
<td>8.94</td>
<td>5.06</td>
</tr>
<tr>
<td>9.8</td>
<td>6.51</td>
<td>6.81</td>
<td>6.81</td>
</tr>
<tr>
<td>14.7</td>
<td>6.28</td>
<td>6.03</td>
<td>6.03</td>
</tr>
</tbody>
</table>

* The number in the brackets represent damping ratio.

As natural frequency and damping ratio are vibration dependent parameters of a structure, it can be seen in above table that the damping ratios of the structure under same roof weight increase with the vibration intensity.

where \( f \), \( k \), \( m \), are respectively the natural frequency, stiffness and mass of system. Hence if we compare different systems with different number of steel plates, we can use the following relationship:

\[
\frac{f_1}{f_2} = \sqrt{\frac{k_1 m_2}{k_2 m_1}}
\]  

(2)

If the weight of the complex bracket set is negligible, assume the weight of steel plate is \( m \), and then we can calculate the stiffness of complex bracket set with 2 and 3 steel plates as:

\[
\frac{8.06}{6.81} = \sqrt{\frac{k_2 m}{k_1 m}}
\]  

(3)

\[
\frac{8.06}{6.05} = \sqrt{\frac{k_3 m}{k_1 m}}
\]  

(4)

where \( k_2 \) and \( k_3 \) are respectively stiffness of system with two and three steel plates, and the following figure depicts the relationship between system stiffness versus number of steel plate. From Figure 6, it appears that the stiffness of the system shows linear correlation with number of steel plate.

Figure 6: The relationship between roof weight and stiffness

Although has been explained in previous section, increase of roof weight results in higher stiffness in connections and leads to stiffer structure, the stiffness of the entire structure is traded off by the roof weight. This explains the reason the natural frequencies of the structure decreases with the increase of the roof weight.

3.3 ULTIMATE STRENGTH OF THE SPECIMEN

Figure 7 below shows the natural frequencies of the specimens under different roof weight (4.9, 9.8 and 14.7 kN) and peak ground acceleration (110, 220 and 330 gal). It appears that failure occurred at combination of large PGA (330 gal) and roof weight (14.7 kN). The natural frequencies of the specimens dropped to 4.12 Hz from 6.05, this means that the stiffness of the whole system decreased to around 40% of its origin and can be considered as failure condition.
3.4 ANALYTICAL MODEL

To analyse the structure, an analytical model was created using ROBOT. The model is illustrated in Figure 8, the horizontal and vertical members are assumed as rigid bars, whilst the connections are assumed as rotational spring with stiffness of 42.7 kNm/rad. The surface connects the two parallel complex brackets are assume as rigid with no in-plan deformation. The roof weight was assume as 14.7 kN, the natural frequency of the structure obtained from the mode analyses was 6.05 Hz. Acceptable agreement between results from dynamic tests (6.26 from 20gal white noise and 6.05 Hz from 40 and 60 gal white noise) and that from analytical model (6.05 Hz) can be found.

4 CONCLUSIONS

The shaking table tests have been carried out to investigate the dynamic properties of traditional complex bracket set in an oriental temple. The following conclusions can be drawn:

1. System identification shows the natural frequencies decrease with the increase of roof weight. The damping ratio will increase with the increase of the intensity of excitation.

2. The whole system can be simplified as a lump mass system with single degree of freedom.

3. The natural frequencies of the specimens dropped significantly representing a reduction of 46% of stiffness at 330 gal PGA and 14.7 kN roof weight. General trend can be found that natural frequency of the system will decrease with combined increase of PGA and roof weight.

4. Analytical model was created to analyse the structure, the spring stiffness of the connection was verified by static tests and comparison with the system identification results.

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REFERENCES


SEISMIC RETROFIT METHOD FOR EXISTING WOODEN HOUSES WITH LATTICE BEARING WALL USING THICK PLYWOOD

Ji-young PARK¹, Toshihiro KUWANO¹, Kei TANAKA¹, and Masafumi INOUE¹

ABSTRACT: In Japan, whenever the severe earthquake occurred, many residents were injured or died by the collapse of old wooden houses with poor seismic strength. However, the seismic strengthening for existing wooden house is not carried out adequately in Japan. The reasons why the residents do not carry out the seismic strengthening, are high cost, long work period and so on for retrofit works. In this study, we developed the new seismic strengthening method by the thick plywood with openings to reduce high cost and long work period for existing wooden houses. We can obtain a good appearance and open atmosphere in the wooden house using the thick plywood with openings. In this study, the horizontal loading tests for some kinds of bearing wall made of thick plywood with openings were carried out.

KEYWORDS: Wooden house, Seismic retrofit, Latticed bearing wall, Wall strength factor

1 INTRODUCTION

We propose the new type of wooden wall composed of the thick plywood with openings and the new seismic retrofit system for the wooden post and beam houses. By attaching the thick plywood with openings to the frame of post and beam houses, the good appearance and open atmosphere in the house can be obtained. In this paper, we propose for new column-sill connection method. The tension tests of column-sill connection are carried out. Based on their results, the horizontal loading tests are carried out to research the structural performance of the thick plywood wall with openings.

2 TENSION TEST OF COLUMN-SILL CONNECTION

2.1 TEST SPECIMENS OF COLUMN-SILL CONNECTION

In this retrofit method, it is important that the hardware of column-sill connection is not visible from the surface of the wall. Therefore, three specification strengthening unit for column-sill connection are proposed in this study. The following describes the three specification strengthening unit for column-sill connection. The first specification of test specimen is shown Figure 1. The first specification is not using hardware. The plywood is fixed to 45mm square cross section timbers by wood screw of 65mm in length. The square cross section timbers are fixed to column or sill by wood screw of 90mm in length. The second specification of test specimen is shown Figure 2. The second specification is added L shaped hardware to the first specification of test specimen. The wood screw for column-hardware and sill-hardware joint is 90mm in length. The third specification of test specimen is shown Figure 3.

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Figure 1: The First specification (Unit in mm)
2.2 TEST PROCEDURE OF COLUMN-SILL CONNECTION

The experimental equipment is shown Figure 4. The loading test is carried out according to the rule specified by the testing manual (Allowable stress design method for wooden post and beam house) 1). Tensile force is applied monotonously to specimens by oil jack. Load speed is less than 0.025kN/sec.

2.3 TEST RESULTS AND DISCUSSIONS OF COLUMN-SILL CONNECTION

Photo 1 shows the typical failure modes. The failure mode of the first specification is pulling-out failure of column-sill connection. The typical failure mode of the second specification and third specification is pull-out failure of column-sill connection, and pull-out failure of wood screw.

Figure 5 shows the ultimate load of test specimens. In this test, the third specification of test specimen is shown the highest ultimate load of all.

3 HORIZONTAL LOADING TEST OF BEARING WALL

3.1 TEST SPECIMENS OF 1P(910mm in width)

Bearing walls for the horizontal loading test are adopted column-sill connection method from former section. The wooden frame of test specimen is 910mm in width, 2730mm in height. Thick plywood with openings 36mm in thickness is used.

The ST type specimen is shown in Figure 6. The ST type specimen is adopted the first specification of column-sill connection in former section. that is, the ST type is reinforced by only square timber and wood screw in column-sill connection. The ST type is installed corner hardware for column-beam connection.

The HWST type specimen is shown Figure 7. The HWST type specimen is adopted the second specification of column-sill connection. The HWST is added L shaped hardware to the ST type specimen. The HWST type is also installed corner hardware for column-beam connection.
The T1HHS type specimen is shown Figure 8. The T1HHS type specimen is adopted as the third specification of column-sill connection. The T shape cross section hardware is installed to column-sill connection for strengthening, and plywood are fixed to column by lag screw from the opening, and fixed to hardware by wood screw. The T1HHS type specimen is used L shaped hardware for column-beam connection.

3.2 TEST SPECIMENS OF 2P(1820mm in width)
The wooden frame of test specimen is 1820mm in width, 2730mm in height. Thick plywood 28mm in thickness is using. The 45mm square timbers are fixed to column or sill by wood screw of 90mm in length. The plywood are fixed to 45mm square timbers by wood screw of 65mm in length. The VP type specimen is shown Figure 9. The VP type specimen is added to retrofit beam and column. Two sheet of plywood with openings are inserted vertically. The HP type specimen is shown Figure 10. The HP type specimen is added to retrofit beam only. The three sheet of plywood are horizontally inserted.

3.3 TEST PROCEDURE
The loading test is carried out according to the rule specified by the testing manual [Allowable stress design method for wooden post and beam house] 1). Loading program and horizontal loading test set-up are shown in Figure 11 and 12. Photo 3 shows the test specimens of
2P. Horizontal cyclic load is applied to specimens by oil jack. Load speed is less than 0.03mm/sec. Bracing is installed to prevent the slant to out-of-plane. Experiment is controlled by the deformation angle. The repeated loadings are carried out at same deformation three times.

3.4 TEST RESULTS AND DISCUSSIONS
Photo 4, 5, 6, 7 and 8 shows the typical failure modes. Photo 4 shows the splitting failure of timber in ST type. Photo 5 shows the deformation of hardware in HWST type. Photo 6 shows the splitting failure of sill in T1HHS type. Photo 7 shows the splitting failure of sill in VP type. Photo 8 shows the gap between plywood in HP type.

![Photo 4: Splitting failure of timber (ST type)](image)

![Photo 5: Deformation of hardware (HWST type)](image)

![Photo 6: Splitting failure of sill (T1HHS type)](image)
Figure 13 shows the relation between load and deflection angle of the typical specimens of each series specimens in this test. The load in VP type specimens and HP type specimens are fall down due to the sudden destruction at 6th cycle.

$$K = \frac{P_1}{\delta_1}$$  \hspace{1cm} (1)

Here,
K: Initial stiffness
P1: Load for 1/120rad
\(\delta_1\): Deformation angle (1/120rad)

Figure 14 shows the initial stiffness (K) of all specimens. The initial stiffness mentioned above is determined by Equation (1).
As shown in Figure 14, initial stiffness of 1P is almost the same value.
The initial stiffness of VP type is higher than HP type. It is reason that the HP type occur large gap between plywood.

Figure 15 shows the ultimate load of test specimens. In this test, the T1HHS type is obtained the highest ultimate load of 1P. The average of ultimate load of T1HHS type is about 8.3kN. Comparing the HWST type and ST type, despite using the hardware, HWST type is not high ultimate load.
The ultimate load of HP type is higher than VP type.
It is reason that column-sill connection of VP type is given serious damage due to high stiffness of plywood.
The average of ultimate load of VP type is about 10kN.
The average of ultimate load of HP type is about 14kN.
Figure 15: Ultimate load

Figure 16 indicates a schematic explanation of perfect bi-linear approximation. At first, an envelope curve of P-δ relationship should be prepared at least for the data belonging in the quadrant where last loading was done. Using 0.1P_{max}, 0.4P_{max}, and 0.9P_{max}, a yield load P_y and stiffness K are determined. An ultimate strength P_u is determined so as to make the trapezoid area surrounded by line V, VI, and δ_s equivalent to that of S surrounded by envelop curve. The ductility factor μ is defined as δ_u/δ_v. A factor D_s, which characterizes equivalent strength of the structure in the aspect of energy dissipation, is also determined as D_s=1/(2μ -1). The straight line which connects (P_y,δ_v) to the starting point is set to V straight line, and this is determined as initial stiffness (K).

Using these values, the following equation (2) to calculate the wall strength factor. The wall strength factor is fundamental value used for structural design of wooden house in Japan.

Difference coefficient= 1-CV \times k
(3)

Here,
CV: Coefficient of variation
k: Factor for determining the permissible limits in the lower 50% confidence coefficient 75%
(a) Yield load: P_y
(b) Ultimate strength: P_u \times (0.2/D_s)
(c) P_{max} 2/3
(d) Equivalent strength

Table 1 shows the wall strength factor and the data used in calculation of wall strength factor. As shown in Table 1, the wall strength factors are determined by the vales based on the structural characteristics factor D_s.

In this test, the T1HHS type shows the highest wall strength factor of 1P. It is reason that column-sill connection is reinforced by T shape hardware. The wall strength factor of ST type and HWST type are about the same value. The wall strength factor of HP type is higher than VP type. It is reason that column-sill connection of VP type is given serious damage due to high stiffness of plywood.

Table 1: Wall strength factor and the data used in calculation of wall strength factor

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Evaluation item (Unit: kN)</th>
<th>Wall strength factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P_{y}</td>
<td>2/3P_{max}</td>
</tr>
<tr>
<td>ST-No.1</td>
<td>2.88</td>
<td>2.97</td>
</tr>
<tr>
<td>ST-No.2</td>
<td>3.19</td>
<td>3.02</td>
</tr>
<tr>
<td>ST-No.3</td>
<td>3.18</td>
<td>3.14</td>
</tr>
<tr>
<td>Average</td>
<td>3.09</td>
<td>2.94</td>
</tr>
<tr>
<td>Difference coefficient</td>
<td>0.91</td>
<td>0.98</td>
</tr>
<tr>
<td>HWST-No.1</td>
<td>3.97</td>
<td>4.20</td>
</tr>
<tr>
<td>HWST-No.2</td>
<td>3.94</td>
<td>3.30</td>
</tr>
<tr>
<td>HWST-No.3</td>
<td>4.23</td>
<td>4.20</td>
</tr>
<tr>
<td>Average</td>
<td>3.72</td>
<td>3.90</td>
</tr>
<tr>
<td>Difference coefficient</td>
<td>0.91</td>
<td>0.94</td>
</tr>
<tr>
<td>T1HHS-No.1</td>
<td>4.34</td>
<td>5.92</td>
</tr>
<tr>
<td>T1HHS-No.2</td>
<td>4.24</td>
<td>5.66</td>
</tr>
<tr>
<td>T1HHS-No.3</td>
<td>4.56</td>
<td>5.19</td>
</tr>
<tr>
<td>Average</td>
<td>4.59</td>
<td>5.59</td>
</tr>
<tr>
<td>Difference coefficient</td>
<td>0.98</td>
<td>0.97</td>
</tr>
<tr>
<td>VP Type No.1</td>
<td>8.63</td>
<td>6.90</td>
</tr>
<tr>
<td>VP Type No.2</td>
<td>8.01</td>
<td>6.77</td>
</tr>
<tr>
<td>VP Type No.3</td>
<td>8.06</td>
<td>7.13</td>
</tr>
<tr>
<td>Average</td>
<td>8.17</td>
<td>6.93</td>
</tr>
<tr>
<td>Difference coefficient</td>
<td>0.99</td>
<td>0.99</td>
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<tr>
<td>HP Type No.1</td>
<td>6.66</td>
<td>8.02</td>
</tr>
<tr>
<td>HP Type No.2</td>
<td>6.59</td>
<td>8.80</td>
</tr>
<tr>
<td>HP Type No.3</td>
<td>7.17</td>
<td>10.07</td>
</tr>
<tr>
<td>Average</td>
<td>6.81</td>
<td>9.60</td>
</tr>
<tr>
<td>Difference coefficient</td>
<td>0.98</td>
<td>0.97</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS
In this study, the new type of wooden wall composed of the thick plywood with openings and the new seismic retrofit system for the wooden post and beam houses were proposed.

From the test result, the new seismic retrofit system has enough effect of retrofit performance for the wooden post and beam houses.

On the end stage of loading, the splitting failure of sill was seen a lot. Therefore, for obtain the higher seismic performance, it is necessary to examine avoid the splitting failure of sill

ACKNOWLEDGEMENT
Authors thank for the help of the members of the laboratory of timber structure Oita University.

REFERENCES
A Study on Technical Development of Horizontal Diaphragm with Grid Beams of Stairwell

Satoshi Shimura¹, Kenji Miyazawa², Shinji Hikida³ and Yuushou Nakamoto⁴

ABSTRACT: Various technological development has enabled to secure and keep seismic performance of wooden houses. But there is a tendency that amenity of living space has not been over looked. Therefore horizontal diaphragm with grid beams of Stairwell has been developed. First, We made test on elements prior to full-scale testing. Using the element test results, we made spring model of column to use analysis. From the analitical models, four models were selected for full-scale testing. The results of analysis and testing did not agree well. The reason is supported that some of cross-beams cannot transfer force as analysis assumption. Next, we tried analysis again considering this test result, and the result agreed well with the test result. Therefore it is necessary to consider stress distribution in the analysis. As the result of this study, We obtained the technology of grid beams of stairwell, which secure not only seismic performance but also comfortableness.

KEYWORDS: Grid beams, Stairwell, Horizontal diaphragm, Floor strength

1 Study background and purpose

Various technological development has enabled to secure and keep seismic performance of wooden houses. But there is a tendency that amenity of living space has not been over looked. The horizontal diaphragm with grid beams of Stairwell has been developed.

2 Element test

2.1 Purposes of element test

The purpose of the element test is to grasp the moment rigidity of the joint used in the grid beams to prepare the input data for the analysis.

2.2 Specimen

Figure 1 shows, the connection used in the joint, which is named “GOYA”. Six specimens with same specification were prepared. The Specimens are T-shaped with 910 mm in distance between fulcrums and 455 mm in height.

2.3 Test method

The test was carried out the displacement control using actuator. Target deformation angles of each cycle is 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50 and 1/30 rad.

2.4 Test results

Figure 2 shows, degreasing of load was not observed until 1/15 rad.

Figure 1: Specimen and joint

Figure 2: Load-displacement curve

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⁴ Yuushou Nakamoto, WOOD ONE Co, LTD., 1-1 Mokuzaiko-Minami, Hatsukaichishi City, Hiroshima Prefecture, Japan 738-8502, Japan Tel: +81 829 32 3333
3 Analysis

3.1 Purposes of analysis

The purpose of the analysis is to evaluate basic properties of horizontal diaphragm with grid beams of stairwell against shear force, and to elucidate the stress estimation by incremental load analysis.

3.2 Analysis model (3640 by 3640)

Figure 3 shows, analysis models are 3640 mm in width and 3640 mm in height. Total fourteen models were analyzed changing number of beams and connectors.

3.3 Analysis results

Table 2 shows, summary of the analysis results. Floor strength ratio, \( R_f \), is calculated using equation (1).

\[
R_f = \frac{P_{120}}{1960L}
\]

where \( P_{120} \) = load at 1/120 rad., \( L \) = Specimen width, 1960 [N] = Reference value of shear strength.

As the result of analysis, floor shear strength ratio of model 44-0 is 1.47. There is a tendency that floor shear strength ratio reduces as the number of beams reduces.

3.4 Analysis model and results (Other models)

Figure 4 shows, analysis models are 2730 mm in width and 3640 mm in height, 2730 mm in width and 2730 mm in height, 1820 mm in width and 3640 mm in height, 1820 mm in width and 2730 mm in height, 1820 mm in width and 1820 mm in height, 910 mm in width and 3640 mm in height, 910 mm in width and 2730 mm in height and 910 mm in width and 1820 mm in height models.

Figure 5 shows, test models are 3640 mm in width and 3640 mm in height. Test models were selected from analysis models. The standard test model 44-0, Lots of stairwell model 44-11, half of the entire stairwell model 44-11 and smallest strength model 44-12.

As the result of analysis, Floor strength, square models (33 and 22) are higher than 44-0. Rectangle models (34, 24, 23, etc.) are lower than square models (33 and 22).

4 Test

4.1 Purposes of test

Test purposes are to confirm analysis results and to confirm fracture mode.

4.2 Test specimen

Figure 5 shows, test models are 3640 mm in width and 3640 mm in height. Test models were selected from analysis models. The standard test model 44-0, Lots of stairwell model 44-11, half of the entire stairwell model 44-11 and smallest strength model 44-12.
Figure 5: Test specimen

Number of specimen is two for model 44-0. Number of specimen is one for model 44-10. Number of specimen is three for model 44-11 and model 44-12. “GOYA” connections are used at the end of beams in Y-direction.

4.3 Test method

Figure 6 shows, it is test method. The test was carried out the displacement control using actuator. Target deformation angles are 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50 and 1/30 rad.

Table 4: Characteristic values

<table>
<thead>
<tr>
<th>Characteristic value</th>
<th>Pmax (kN)</th>
<th>δ Pmax (mm)</th>
<th>K (kN/m m)</th>
<th>Ds</th>
<th>Pu(kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>44-0-1</td>
<td>49.00</td>
<td>395.69</td>
<td>0.32</td>
<td>0.46</td>
<td>45.39</td>
</tr>
<tr>
<td>44-0-2</td>
<td>51.68</td>
<td>377.10</td>
<td>0.26</td>
<td>0.49</td>
<td>48.50</td>
</tr>
<tr>
<td>44-10</td>
<td>17.92</td>
<td>242.38</td>
<td>0.12</td>
<td>0.61</td>
<td>15.75</td>
</tr>
<tr>
<td>44-11-1</td>
<td>21.72</td>
<td>250.00</td>
<td>0.12</td>
<td>0.69</td>
<td>19.07</td>
</tr>
<tr>
<td>44-11-2</td>
<td>22.52</td>
<td>242.94</td>
<td>0.12</td>
<td>0.72</td>
<td>20.38</td>
</tr>
<tr>
<td>44-11-3</td>
<td>21.04</td>
<td>241.59</td>
<td>0.12</td>
<td>0.71</td>
<td>18.49</td>
</tr>
<tr>
<td>44-12-1</td>
<td>8.98</td>
<td>227.87</td>
<td>0.07</td>
<td>0.58</td>
<td>8.22</td>
</tr>
<tr>
<td>44-12-2</td>
<td>9.88</td>
<td>240.00</td>
<td>0.08</td>
<td>0.59</td>
<td>3.64</td>
</tr>
<tr>
<td>44-12-3</td>
<td>9.52</td>
<td>222.83</td>
<td>0.07</td>
<td>0.55</td>
<td>3.39</td>
</tr>
</tbody>
</table>

Table 4 shows, characteristic values. Average Structural characteristic factor (Ds) of model 44-0 is 0.45. The values of Structural characteristic factor (Ds) for 44-10, 44-11 and 44-12 were not same. The first rigidity of model 44-0 is 0.34. The first rigidity for 44-12 was 0.07, but about 21% of 44-0.

Table 5: Floor strength

<table>
<thead>
<tr>
<th>Shear strength</th>
<th>2/3Pmax (kN)</th>
<th>Pu/0.2/Ds (kN)</th>
<th>Py (kN)</th>
<th>PT120 (kN)</th>
<th>Po (kN)</th>
<th>Pu (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>44-0-2</td>
<td>34.43</td>
<td>34.06</td>
<td>33.21</td>
<td>12.28</td>
<td>12.21</td>
<td>12.40</td>
</tr>
<tr>
<td>44-10</td>
<td>1.19</td>
<td>1.11</td>
<td>1.04</td>
<td>1.32</td>
<td>1.27</td>
<td>1.51</td>
</tr>
<tr>
<td>44-11-1</td>
<td>13.41</td>
<td>12.71</td>
<td>12.64</td>
<td>6.00</td>
<td>4.60</td>
<td>6.08</td>
</tr>
<tr>
<td>44-11-2</td>
<td>15.91</td>
<td>15.28</td>
<td>15.01</td>
<td>6.44</td>
<td>4.94</td>
<td>6.64</td>
</tr>
<tr>
<td>44-11-3</td>
<td>14.03</td>
<td>13.28</td>
<td>13.05</td>
<td>6.37</td>
<td>4.93</td>
<td>6.67</td>
</tr>
<tr>
<td>44-12-1</td>
<td>5.97</td>
<td>5.24</td>
<td>5.11</td>
<td>2.05</td>
<td>1.50</td>
<td>2.10</td>
</tr>
<tr>
<td>44-12-2</td>
<td>6.35</td>
<td>5.66</td>
<td>5.51</td>
<td>2.40</td>
<td>1.80</td>
<td>2.50</td>
</tr>
<tr>
<td>44-12-3</td>
<td>6.59</td>
<td>5.84</td>
<td>5.73</td>
<td>2.60</td>
<td>2.00</td>
<td>2.60</td>
</tr>
</tbody>
</table>

Table 5 shows, floor strength. Floor shear strength ratio was determined as same as the analysis result using P120. Average floor shear strength ratio of model 44-0 is 1.84, which was determined by the strength at deformation angle 1/120 radian, 13.06 kN. The value of Floor strength ratio for 44-10 and 44-11 were almost same, but about 35% of 44-0. The value of Floor strength ratio for 44-12 was 0.38, but about 20% of 44-0.

5 Comparison of analytical and test

Figure 7 shows, Degreasing of load was not observed until 1/10 rad. The variation of strength in each model was small. Table 4 and table 5 shows, characteristic values obtained by the test.

Figure 7: Load- displacement curve

Figure 8: Comparison of analytical and test

Figure 9: Comparison of analytical and test (1/120 rad)
Table 5: Compare analysis results with test results

<table>
<thead>
<tr>
<th></th>
<th>P_{ab}</th>
<th>P_{aa}</th>
<th>R_{1/120}rad</th>
<th>R_{1/115}rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>44-0</td>
<td>10.31</td>
<td>12.58</td>
<td>1.46</td>
<td>1.79</td>
</tr>
<tr>
<td>44-0</td>
<td>8.72</td>
<td>10.52</td>
<td>1.22</td>
<td>1.43</td>
</tr>
<tr>
<td>44-10</td>
<td>65%</td>
<td>84%</td>
<td>84%</td>
<td>85%</td>
</tr>
<tr>
<td>44-10</td>
<td>88%</td>
<td>54%</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>44-10</td>
<td>3.68</td>
<td>4.68</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>44-10</td>
<td>99%</td>
<td>104%</td>
<td>104%</td>
<td>104%</td>
</tr>
<tr>
<td>44-11</td>
<td>3.60</td>
<td>4.60</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>44-11</td>
<td>4.00</td>
<td>4.84</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>44-11</td>
<td>3.80</td>
<td>4.82</td>
<td>0.65</td>
<td>0.65</td>
</tr>
<tr>
<td>44-11</td>
<td>3.80</td>
<td>4.89</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>44-11</td>
<td>4.80</td>
<td>5.32</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>44-11</td>
<td>113%</td>
<td>114%</td>
<td>113%</td>
<td>114%</td>
</tr>
<tr>
<td>44-12</td>
<td>2.20</td>
<td>2.65</td>
<td>0.31</td>
<td>0.37</td>
</tr>
<tr>
<td>44-12</td>
<td>2.20</td>
<td>2.65</td>
<td>0.31</td>
<td>0.37</td>
</tr>
<tr>
<td>44-12</td>
<td>2.20</td>
<td>2.65</td>
<td>0.31</td>
<td>0.37</td>
</tr>
<tr>
<td>44-12</td>
<td>2.28</td>
<td>2.72</td>
<td>0.32</td>
<td>0.38</td>
</tr>
<tr>
<td>44-12</td>
<td>104%</td>
<td>103%</td>
<td>104%</td>
<td>104%</td>
</tr>
</tbody>
</table>

From the comparison of analysis and results, model 44-10 and model 44-12 almost agreed well. The results of model 44-11 between analysis and testing did not agree well. Figure 10 show, the reason is supports that some of cross-beams cannot transfer force as analysis assumption.

Figure 10: Stress intensity distribution

Next, we tried analysis again considering this test result, and the result agreed well with the test result. Therefore it is necessary to consider stress distribution in the analysis.

Table 6: To compare analysis results with test results (Before and after the modification)

<table>
<thead>
<tr>
<th></th>
<th>Before</th>
<th>After</th>
<th>Test</th>
<th></th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1/150</td>
<td>1/120</td>
<td>1/115</td>
<td>1/150</td>
<td>1/120</td>
</tr>
<tr>
<td>44-10</td>
<td>3.88</td>
<td>4.68</td>
<td>4.84</td>
<td>3.52</td>
<td>4.20</td>
</tr>
<tr>
<td>44-11</td>
<td>4.40</td>
<td>5.52</td>
<td>5.52</td>
<td>3.80</td>
<td>4.56</td>
</tr>
<tr>
<td>44-12</td>
<td>2.28</td>
<td>2.72</td>
<td>2.84</td>
<td>2.20</td>
<td>2.64</td>
</tr>
</tbody>
</table>

Before the modification, Model 44-10 is 4.84kN at 1/115rad. Test and analysis is 93%. After the modification, Model 44-10 is 4.36kN at 1/115rad. The ratio is 103% and is Similar to test result. Before the modification, Model 44-11 is 5.52kN at 1/115rad. Test and analysis is 85%. After the modification, Model 44-11 is 4.72kN at 1/115rad. The ratio is 99%. Almost match. Before the modification, Model 44-12 is 2.84kN at 1/115rad. The ratio is 94%. After the modification, Model 44-12 is 2.72kN at 1/115rad. The ratio is 99%. After analysis modification, analytical results agreed well with test results. It seems possible to use this analysis method for various types of horizontal diaphragm with grid beams of stairwell.

6 Summary and study on

From the observation of damage by the test, large section suffered smaller damage than small section. Considering the deformation, Model 44-12 deformed uniformly. But for another models concentration of deformation was observed. So designers should be careful if they use asymmetric models.

7 Development in the future

As the result of this study, we obtained the technology of grid beams of stairwell, which secure not only seismic performance but also the conformableness. Just as well as basic model 44-0 other models can secure Floor shear strength. Although these grid beams are assumed to be used at floor openings, these are as may be canged to the actual floor sheathed with plurood or other floor seathing materials, in the future.

Figure 11: Room arrangement ex

Acknowledgement

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[2] Kanto Chapter Architectural Institute of Japan: Easy to learn the structural design of timber structures design.(in Japanese)
Seismic Reinforcement for Traditional Wooden Frame by Improving Restoring Force due to Column Rocking

Tatsuru Suda¹, Yoshiyuki Suzuki², Yasuhiko Tashiro³, Kyosuke Mukaibou⁴

ABSTRACT: As a seismic reinforcement method for tradition wooden buildings such as Temples, the improvement method of restoring force due to column rocking by making a cross-section of column bottom larger is presented. To verify the effect of the seismic reinforcement, static tests and shaking table tests using a full-scale model were carried out. It is found that the restoring forces due to column rocking is improved especially for large deformation by comparison with that of unreinforced wooden frame. It is then confirmed that the traditional wooden frame has high performances of deformability and restoring force by the seismic reinforcement.

KEYWORDS: Traditional wooden building, Reinforcement method, Restoring force due to column rocking

1 INTRODUCTION

Many traditional wooden buildings such as temples and shrines exist in Japan. They often do not satisfy the required seismic performance because the traditional wooden structures have heavy roofs and their base shear coefficients are usually low. The seismic reinforcement techniques suited to traditional wooden structures are desired. In this study, as a seismic reinforcement method for a temple tradition wooden building, the method improving restoring force due to column rocking is proposed by increasing a cross-section of column bottom by fitting a reinforcement member up a column bottom.

2 OUTLINE OF SPECIMEN

To clarify the restoring force due to column rocking of traditional wooden frame and to verify the effect of the reinforcement method, static lateral loading tests using a full-scale model were conducted. The specimen of traditional wooden frame is shown in Figure 1. The specimen consists of four circular columns, floor tie-beams, mid-wall tie-beams, pillow blocks, girders as shown in Figure 2. The columns are set on base stones without any connectors. Two precast concrete (PC) panels of total weight 109.6kN were set on the girder as weight equivalent to a roof. The roof weight of specimen is increasing from 109.6kN to 148.9kN and 188.1kN by steel panels. The reinforcement members are constituted by four wooden, and the reinforcement members are fitted up the column bottom of the specimen, as shown Figure 3.

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³ Yasuhiko TASHIRO, Nikken Sekkei Ltd, 4-6-2 Kouraibashi, Osaka Chuo-Ku, Osaka, 541-8528, Japan. Email: tashiro@nikken.co.jp
⁴ Kyosuke MUKAIBO, College of Science & Engineering, Ritsumeikan University, 1-1-1 Nojihigashi, Kusatsu, Shiga, 525-8577, Japan. E-mail: mukaibo@fc.ritsumei.ac.jp
reinforcement members were made by a Japanese cypress and Afzelia (Zelkova from Africa).

![Figure 3: Reinforcement members and Situation of reinforced column bottom](image)

**(a) Reinforcement members**

**(b) Situation of reinforced column bottom**

![Figure 4: Component parts of the reinforcement member](image)

**(a) Type A**

**(b) Type B**

The component parts of a reinforcement members are shown in Figure 4(a) and (b). In the test of first period, as shown in Figure 4 (a), the slot cut for steel ring was processed to the reinforcement members. The steel ring was inserted in the slot in order to prevent movement of a reinforcement member. The cross-section size of the column bottom is increasing from 300mm in the diameter to 500mm by this reinforcement method. The reinforcement members were made by the Japanese cypress. Figure 4 (b) shows the component parts of improved reinforcement member. In the test of the second period, in order to prevent movement of a reinforcement component, the slot cut for a wooden dowel was processed in the side of the reinforcement members further. In addition, the split tensile of the lower part of a reinforcement member reinforced on the screw. The cross-section size of the column bottom is increasing from 300mm in the diameter to 400mm and 500mm by this reinforcement method. The
3 OUTLINE OF STATIC LATERAL LOADING TEST

The parameter of a specimen and a test is shown in Table 1. The testing parameters were the weight of dead load, the cross-section size of column bottom, the wooden material of reinforcement members, and the processing method of reinforcement members. The symbol of a specimen indicates the weight of dead load, the wooden material, the cross-section size of column bottom, and type of reinforcement member, respectively. The static lateral loading tests were carried out for the specimen. The static horizontal forces were applied at the level of PC panels by using an actuator under displacement control. As a program of static lateral loading test, the loadings are repeated three times until each specified deformation amplitude in both positive and negative directions. In the test of the first period, the displaced target points were programmed at 90 mm, 180 mm, and 210 mm. In the test of the second period, the maximum displaced target point was programmed at 270 mm.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Weight of dead load (kN)</th>
<th>Cross-section size of column bottom (mm)</th>
<th>Wooden material</th>
<th>Reinforcement member</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-H500A</td>
<td>109.6</td>
<td>500</td>
<td>Japanese cypress</td>
<td>A</td>
</tr>
<tr>
<td>B-H500A</td>
<td>148.9</td>
<td>500</td>
<td>Japanese cypress</td>
<td>A</td>
</tr>
<tr>
<td>C-H500A</td>
<td>188.1</td>
<td>500</td>
<td>Japanese cypress</td>
<td>A</td>
</tr>
<tr>
<td>C-N300</td>
<td>188.1</td>
<td>300</td>
<td>Without reinforcement</td>
<td>B</td>
</tr>
<tr>
<td>A-H400B</td>
<td>109.6</td>
<td>400</td>
<td>Japanese cypress</td>
<td>B</td>
</tr>
<tr>
<td>A-A400B</td>
<td>109.6</td>
<td>400</td>
<td>Afzelia</td>
<td>B</td>
</tr>
<tr>
<td>A-H500B</td>
<td>109.6</td>
<td>500</td>
<td>Japanese cypress</td>
<td>B</td>
</tr>
<tr>
<td>A-A500B</td>
<td>109.6</td>
<td>500</td>
<td>Afzelia</td>
<td>B</td>
</tr>
<tr>
<td>A-N300</td>
<td>109.6</td>
<td>300</td>
<td>Without reinforcement</td>
<td>B</td>
</tr>
</tbody>
</table>

4 RESTORING FORCE DUE TO COLUMN ROCKING

The horizontal load of specimen, the relative story displacement, and the bending moment of the tie-beams were obtained from the static lateral loading test. The relation between the whole restoring force and the story deformation is shown in Figure 5. The bending moments of the two tie-beams were calculated from strains measured on them. The bending moment of the floor tie-beam and the mid-wall tie-beam are shown in Figure 6 and Figure 7. The whole restoring force shown in Figure 5 consists mainly the bending moments of tie-beams and the restoring forces due to rocking columns. The restoring force due to column rocking shown in Figure 8 is evaluated by using test data[1-2].

![Figure 5: Relation between the whole restoring force and the relative story displacement](image)

![Figure 6: Relation between the bending moment and the rotation angle of the floor tie beam](image)

![Figure 7: Relation between the bending moment and the rotation angle of the mid-wall tie beam](image)
5 EFFECTS OF PARAMETERS ON RESTORING FORCE DUE TO COLUMN ROCKING

Figure 9 shows the effect of reinforcement of the restoring force due to column rocking by comparison between with and without the reinforcement. It is found that the deformable performance of specimen increases by the reinforcement. The effect of the weight of dead load about a restoring force due to column rocking is shown in Figure 10. It is found that the maximum restoring force of specimen increases as the weight of dead load increase. The effect of the cross-section size of column bottom about a restoring force due to column rocking is shown in Figure 11. It is found that the deformable performance of specimen increases as the cross-section size of column increase. However, the maximum restoring force of a specimen is almost the same in all the cases about the test as the parameter of cross-sectional size. The effect of the wooden materials of reinforcement member about a restoring force due to column rocking is shown in Figure 12. The Afzelia of performance is more effective than the Japanese cypress.

The Characteristics of restoring force due to column rocking has been proposed by some researchers [3-5]. The characteristics of restoring force due to column rocking based on the past researched results are shown conceptually in Figure 13 [6]. The curve shown in Figure 13 is applied as a characteristic of restoring force due to column rocking for a practical seismic design in Japan, coefficient is $\alpha_1=0.1$, $\alpha_2=0.2$ and $\beta=0.8$. 

---

**Figure 8: Restoring force due to column rocking**

**Figure 9: Effect of reinforcement**

**Figure 10: The effect of the weight of dead load about a restoring force due to column rocking**

**Figure 11: The effect of the cross-section size of column bottom about a restoring force due to column rocking**

**Figure 12: The effect of the wooden materials of reinforcement member about a restoring force due to column rocking**
STRENGTH AND SERVICEABILITY – EXTREME EVENTS

Figure 13: The Restoring force due to column rocking for a seismic design of wooden building

Here, $Q (= W \times h / D)$: restoring force, $R (= h / D)$: deformation angle, $W$: vertical load, $h$: height of the column, $D$: diameter of the column.

It is thought that the cross-section size of a column effective for a restoring force characteristic is the value which averaged the column top and the column bottom. Therefore, the value which averaged the cross-section size of a column top and the cross-section size of a column bottom is indicated on explanatory note of Figure 14. As shown in Figure 14, the coefficients $\alpha_1=0.2$, $\alpha_2=0.4$ and $\beta=0.55$ are proposed from the test result of the tradition wooden building using the circular section column at this study.

6 CONCLUSIONS

From the static lateral loading tests using a full-scale specimen of traditional wooden frame, it is found that the restoring forces due to column rocking is improved especially for large deformation. It is also confirmed that the traditional wooden frame has the high performance of deformability as well as the restoring force by the seismic reinforcement. The restoring force characteristic for a practical seismic design was proposed by the full-scale static test of traditional wooden frame. The proposed reinforcement method is therefore useful as the seismic reinforcement for traditional wooden structures with large diameter columns.

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REFERENCES

EVALUATION OF TWO ANALYTICAL PLASTIC DESIGN MODELS FOR LIGHT-FRAME SHEAR WALLS

Bo Källsner¹, Ulf Arne Girhammar², Johan Vessby³

ABSTRACT: The objective of this paper is to clarify the difference between two analytical models for plastic design of shear walls and evaluate their potential for hand calculation by comparing calculated load-bearing capacities of different wall configurations with the corresponding ones obtained by finite element analyses. The first analytical model is based on a true plastic lower bound concept, i.e. always fulfilling the conditions of equilibrium. The second model is based on the assumption that the full vertical shear capacity of the wall is utilized, considering that the vertical equilibrium is always fulfilled but disregarding that the horizontal equilibrium of the wall is not always satisfied. If the shear capacity of the stud-to-rail joints is sufficiently large, then the second model is also a true plastic lower bound method. The ratios between the calculated load-carrying capacities of the two analytical models are in the range between 1.00 – 1.24 with a mean value of 1.12 for the wall and load configurations studied. Results from FE simulations show that the first analytical method underestimates the load-carrying capacity by about 10 %, but that the method gives very stable capacity values relative to the FE simulations. It is further evident that there is a good agreement between the second analytical model and the results of the FE calculations at the mean level, but that this method has a considerably higher variation in the capacity values relative to the FE-simulations. Performed tests of different wall and load configurations show about 30 % higher measured capacities than calculated ones. The large deviations are mainly due to differences in the manufacturing of the specimens for the sheathing-to-framing joint tests and the specimens for the wall tests.

KEYWORDS: Timber shear walls, Partially anchored, Stabilization, Plastic design models, FE analysis, Test results.

1 INTRODUCTION

1.1 BACKGROUND

The authors of this paper have proposed different analytical models for plastic design of timber frame walls subjected to shear. The main idea has been that the proposed analytical models should be based on the principles of the static theorem i.e., based on the conditions that

• the equilibrium is fulfilled in each point of the structure
• the strength of the joints and the material is not exceeded in any point

By fulfilling these conditions the calculated capacity of the structure will always be lower than the “true” plastic capacity of the structure, i.e. a lower bound value of the plastic capacity is obtained.

1.2 AIM AND SCOPE

The aim of this paper is to evaluate two analytical models for plastic design of walls subjected to shear.

The first model is a “true” plastic lower bound model while the second one may give capacities higher than the “true” plastic capacity.

In order to study the differences between the two models finite element calculations are carried out. Calculated capacities are compared with results obtained from testing of walls.

The proposed models can only be applied on walls where the sheet material is fixed by mechanical fasteners to the timber members. The models are only intended for use in case of static loading.

In this paper only wall configurations without openings are studied.
2 TESTING PROGRAM

The wall and load configurations tested and the system of notation used for the different test series are shown in Figure 1. The main load was applied in two different ways; as a diagonal load \( (D) \) or as a horizontal load \( (H) \). The horizontally loaded walls were often simultaneously subjected to a vertical point load \( (V) \) acting on the leading stud or as point loads \( (V) \) distributed on all studs.

![Wall and load configurations tested](image)

Figure 1: Wall and load configurations tested

All walls were sheathed with hardboard of dimension 1200x2400x8 mm\(^3\). Only one side of each wall was sheathed. The timber members were of dimension 45x120 mm\(^2\) with a stud spacing of 600 mm. For the sheathing-to-framing joints 50x2,1 mm annular ringed shank nails were used with a centre distance of 100 mm along the perimeter and 200 mm along the centre line of the sheets. In each stud-to-rail joint two annular ringed shank nails 90x3.1 mm were applied in the grain direction of the vertical studs.

The different test series are specified in Table 1. The vertical load acting on the leading stud is denoted by \( V_0 \) and the vertical load acting on all studs by \( V_i \). Note that at diagonal loading, the vertical load on the leading stud represents a certain proportion of the horizontal load \( H \). The bottom rails were normally continuously anchored to the foundation with respect to vertical uplift. For two of the test series, D2:NA and H4VV:NA (NA = No Anchorage), the bottom rail was not anchored with respect to vertical uplift but was prevented from sliding by a simple stopping device at the end of the wall. One of the test series, H4:NF (NF = No Framing joints), had no nails in the joints between the vertical studs and the bottom rail. This was simulated by cutting off the nails after the specimens had been manufactured.

![Table 1: Specification of walls tested](image)

<table>
<thead>
<tr>
<th>Test series</th>
<th>Number of tests</th>
<th>( V_0 ) [kN]</th>
<th>( V_i ) [kN]</th>
<th>Anch. rail</th>
<th>Framing joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>4</td>
<td>2H</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>D2</td>
<td>4</td>
<td>H</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>D2:NA</td>
<td>4</td>
<td>H</td>
<td>-</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>D3</td>
<td>4</td>
<td>2/3H</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>D4</td>
<td>4</td>
<td>1/2H</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H1</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H2</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H3</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4:NF</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>H2VV(_1)</td>
<td>4</td>
<td>3.23</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H2VV(_2)</td>
<td>4</td>
<td>6.46</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H2VV(_3)</td>
<td>3</td>
<td>12.9</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_1)</td>
<td>3</td>
<td>3.23</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_2)</td>
<td>3</td>
<td>6.46</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_3)</td>
<td>3</td>
<td>12.9</td>
<td>-</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_4)</td>
<td>4</td>
<td>-</td>
<td>1.29</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_5)</td>
<td>4</td>
<td>-</td>
<td>3.23</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>H4VV(_6)</td>
<td>4</td>
<td>-</td>
<td>6.46</td>
<td>no</td>
<td>yes</td>
</tr>
</tbody>
</table>

3 TEST RESULTS

The results obtained from the testing of the walls are specified in Table 2. The loads \( H \) given in the table represent the measured maximum values. In case of diagonal loading, \( H \) represents the horizontal component of the applied diagonal load \( D \).

![Table 2: Results from testing of light-frame shear walls including dry density \( \rho_{ov} \) and moisture content \( \omega \) of frame members. (COV = coefficient of variation).](image)

<table>
<thead>
<tr>
<th>Test series</th>
<th>Number of tests</th>
<th>( \rho_{ov} ) [kg/m(^3)]</th>
<th>( \omega ) [%]</th>
<th>Load ( H ) [kN]</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>4</td>
<td>404</td>
<td>11.5</td>
<td>16.7</td>
<td>0.13</td>
</tr>
<tr>
<td>D2</td>
<td>4</td>
<td>405</td>
<td>14.1</td>
<td>30.0</td>
<td>0.04</td>
</tr>
<tr>
<td>D2:NA</td>
<td>4</td>
<td>428</td>
<td>12.4</td>
<td>31.7</td>
<td>0.04</td>
</tr>
<tr>
<td>D3</td>
<td>4</td>
<td>398</td>
<td>11.8</td>
<td>42.0</td>
<td>0.05</td>
</tr>
<tr>
<td>D4</td>
<td>4</td>
<td>394</td>
<td>14.4</td>
<td>59.8</td>
<td>0.10</td>
</tr>
<tr>
<td>H1</td>
<td>4</td>
<td>409</td>
<td>10.4</td>
<td>3.84</td>
<td>0.13</td>
</tr>
<tr>
<td>H2</td>
<td>4</td>
<td>401</td>
<td>12.1</td>
<td>13.9</td>
<td>0.10</td>
</tr>
<tr>
<td>H3</td>
<td>4</td>
<td>403</td>
<td>11.2</td>
<td>25.9</td>
<td>0.06</td>
</tr>
<tr>
<td>H4</td>
<td>3*</td>
<td>437</td>
<td>15.3</td>
<td>45.2</td>
<td>0.08</td>
</tr>
<tr>
<td>H4:NF</td>
<td>3*</td>
<td>402</td>
<td>14.2</td>
<td>41.9</td>
<td>0.08</td>
</tr>
<tr>
<td>H2VV(_1)</td>
<td>4</td>
<td>389</td>
<td>11.4</td>
<td>19.4</td>
<td>0.06</td>
</tr>
<tr>
<td>H2VV(_2)</td>
<td>4</td>
<td>415</td>
<td>11.2</td>
<td>21.9</td>
<td>0.06</td>
</tr>
<tr>
<td>H2VV(_3)</td>
<td>3</td>
<td>478</td>
<td>14.7</td>
<td>26.5</td>
<td>0.03</td>
</tr>
<tr>
<td>H4VV(_1)</td>
<td>3</td>
<td>431</td>
<td>14.0</td>
<td>49.3</td>
<td>0.10</td>
</tr>
<tr>
<td>H4VV(_2)</td>
<td>1*</td>
<td>412</td>
<td>10.5</td>
<td>49.3</td>
<td>-</td>
</tr>
<tr>
<td>H4VV(_3)</td>
<td>4</td>
<td>422</td>
<td>12.6</td>
<td>60.4</td>
<td>0.08</td>
</tr>
<tr>
<td>H4VV(_4)</td>
<td>3*</td>
<td>402</td>
<td>12.9</td>
<td>51.9</td>
<td>0.03</td>
</tr>
<tr>
<td>H4VV(_5)</td>
<td>4</td>
<td>383</td>
<td>13.4</td>
<td>54.8</td>
<td>0.04</td>
</tr>
<tr>
<td>H4VV(_6)</td>
<td>3*</td>
<td>426</td>
<td>12.8</td>
<td>58.5</td>
<td>0.03</td>
</tr>
</tbody>
</table>

* Some tests results have been excluded
The dry density $\rho_{\text{dry}}$ and the moisture content $\omega$ of the timber members were determined in those areas where the primary failures took place. Failure was normally initiated due to plastic bending and withdrawal of the nails in the sheathing-to-framing joints. The failure mode was generally of a fairly ductile type.

4 ANALYTICAL MODELS

4.1 GENERAL INFORMATION

4.2 MODEL 1

An analytical plastic model fulfilling the principles of the static theorem is presented in [1]. The basic assumptions are as follows:

- the sheathing-to-framing joints, referring to the vertical studs and the top rail, are assumed to transfer shear forces only parallel to the timber members
- the sheathing-to-framing joints, referring to the bottom rail, are assumed to transfer forces either parallel or perpendicular to the bottom rail
- the framing joints are not assumed to transfer any tensile or shear forces
- compressive forces can be transferred via contact between adjacent sheets and in the framing joints.

Regarding the last point in the list it should be mentioned that contact between adjacent sheets can be replaced by an alternative load path where the force is transferred from one sheet to an adjacent one via the joints in an intermediate stud.

In order to obtain simple expressions for the resistance of the shear walls, the fasteners are assumed to be continuously distributed along the center of the timber members. The load-carrying capacity of the sheathing-to-framing joints is consequently given in force per unit length. It is further assumed that the fastener spacing around the perimeter of the sheets is constant.

The model assumptions are in Figure 2 applied on a wall configuration consisting of three sheets fastened to a timber frame. The external loads $V_i$ acting on top of the wall and the wall geometry are defined in the figure. The assumed force distribution along the lower part of the wall is shown in a section immediately above the bottom rail and represents the forces in the sheathing-to-framing joints. These forces are assumed to act either perpendicular or parallel to the bottom rail. The plastic capacity per unit length of these joints is denoted by $f_p$ and it is assumed that plasticity has been attained along the entire bottom rail. The factor $\mu$ opens for the possibility of using reduced strength properties when the fastener forces act perpendicular to the edges of the sheets and the timber members. The wall is divided into two fictitious wall segments, denoted by 1 and 2, assuming that the plastic shear flow has been attained in the vertical section immediately to the right of stud number $n + 1$, thus, resulting in a pure plastic shear flow within wall segment 2. The vertical forces $V_i$ acting on wall segment 1 are assumed to be transferred via the studs to the sheathing material. It should be noted that a part of the force $V_{n+1}$ is transferred to the bottom rail since the plastic shear capacity $f_p$ would otherwise be exceeded at the vertical section shown. The vertical forces acting on wall segment 2 cannot be transferred via the studs to the sheets but must be transferred via the studs to the bottom rail.

![Figure 2: Assumed force distribution using model 1.](image)

The horizontal capacity $H$ is obtained by studying the equilibrium of the two fictitious wall segments. The horizontal forces transferred to the left and right wall segments are denoted by $H_1$ and $H_2$, respectively. A moment equation for wall segment 1 with respect to its lower right corner can be formulated as

$$H_1 h - \sum_{i=0}^{n} V_i (l_i - x_i) - \mu f_p (l_1 - l_{1,\text{eff}}) \left( \frac{l_1 + h_{\text{eff}}}{2} \right) = 0$$ (1)

Force equilibrium in horizontal direction gives

$$H_1 = f_p l_{1,\text{eff}}$$ (2)

$$H_2 = f_p l_2$$ (3)

In order to simplify the equations two non-dimensional parameters are introduced

$$\alpha = \frac{l}{h}$$ (4)

$$\beta = \frac{\sum_{i=0}^{n} \left( \frac{l_i - x_i}{l_i} \right) V_i}{f_p h} = \frac{V_{\text{eq}}}{f_p h}$$ (5)

It should be noted that if the vertical load $V_0$ acting on the leading stud is larger than $f_p h$ it means that the leading stud is fully anchored with respect to uplift and the capacity of the wall is obtained from Equation (3) setting $l_2$ equal to the total wall length. It should further be noted that the numerator $V_{\text{eq}}$ in Equation (5) represents the equivalent vertical force acting on the leading stud, considering wall segment 1 as a simply supported beam on two supports. This means that the $\beta$-value in Equation (5) never can exceed 1.

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The effective wall length \( h_{\text{eff}} \) can now be calculated from Equations (1) and (2) as

\[
l_{\text{eff}} = l \left( \frac{1}{1 + \frac{1}{\alpha \mu}} + \frac{2 \beta}{\alpha \mu} - \frac{1}{\alpha \mu} \right)^{1/2}.
\] (6)

At this stage it must be checked that the plastic shear flow has not been attained in the vertical section left of stud number \( n + 1 \), resulting in the following condition with respect to the vertical force equilibrium

\[
\sum_{i=0}^{n} V_i + \mu f_p (l_i - l_{\text{eff}}) \leq f_p h
\] (7)

If stud number \( n + 1 \) is not the end stud of the wall it must further be checked that the plastic shear flow has been attained in the vertical section to the right of this stud leading to the condition

\[
R_{n+1} = \sum_{i=0}^{n+1} V_i + \mu f_p (l_i - l_{\text{eff}}) - f_p h \geq 0
\] (8)

If the conditions (7) and (8) are fulfilled the total horizontal capacity of the wall \( H \) is obtained by summation of the force contributions \( H_1 \) and \( H_2 \) from Equations (2) and (3) as

\[
H = f_p l \left( \frac{1}{1 + \frac{1}{\alpha \mu}} + \frac{2 \beta}{\alpha \mu} - \frac{1}{\alpha \mu} \right) + f_p l_2
\] (9)

An advantage of this model is that it is a lower bound model. A weakness is, however, that the capacity \( H \) must be calculated iteratively, since the position of stud number \( n + 1 \), where the vertical equilibrium is attained, is not known from the beginning.

If the bottom rail is not anchored with respect to vertical uplift, i.e. \( \mu = 0 \), \( l_{\text{eff}} \) is obtained from Equation (6) as

\[
l_{\text{eff}} = \lim_{\mu \to 0} l \left( \frac{1}{1 + \frac{1}{\alpha \mu}} + \frac{2 \beta}{\alpha \mu} - \frac{1}{\alpha \mu} \right) = \beta l
\] (10)

and the total horizontal capacity of the wall \( H \) becomes

\[
H = \beta f_p l_1 + f_p l_2
\] (11)

4.3 MODEL 2

This model was developed to make better use of the stud-to-rail joints along the bottom rail, which were not used for any transfer of shear forces in model 1, see reference [2]. The assumed force distribution using model 2 is shown in Figure 3 for the previously shown wall configuration. The horizontal shear flow \( f \) along the bottom rail of the left wall segment is assumed to come from the stud-to-rail joints. This shear contribution is in fact distributed along the entire wall and is transferred via contact between the sheets to the left wall segment. Thus Figure 3 only gives a simplified illustration of the force distribution. The basic idea behind this analytical method is that the full vertical shear capacity of the wall should be attained as close to the leading stud as possible, considering that the vertical equilibrium is always fulfilled but disregarding that the horizontal equilibrium of the left wall segment is not always satisfied.

Vertical force equilibrium with respect to the left wall segment gives

\[
\sum_{i=0}^{n} V_i + \mu f_p l_1 = f_p h
\] (12)

The length \( l_1 \) is obtained from Equation (12) as

\[
l_1 = \frac{h}{\mu} \left( 1 - \frac{\sum_{i=0}^{n} V_i}{f_p h} \right)
\] (13)

A moment equation with respect to wall segment 1 around its lower right corner gives

\[
H_1 h - \sum_{i=0}^{n} V_i (l_i - x_i) - \mu f_p l_1 \frac{l_1}{2} = 0
\] (14)

Using the previously defined non-dimensional parameters in Equations (4) and (5), the capacity \( H_1 \) is calculated from Equation (14) as

\[
H_1 = \left( \frac{1}{2} \mu \alpha + \beta \right) f_p l_1
\] (15)

The resulting capacity \( H \) of the two fictitious wall segments becomes

\[
H = \left( \frac{1}{2} \mu \alpha + \beta \right) f_p l_1 + f_p l_2
\] (16)

An advantage of model 2 is that the distance \( l_1 \) can be determined without iterations making calculations by hand easier to carry out.

It is noted here that for \( \mu = 0 \) Equation (16) becomes identical to Equation (11) that was derived for model 1.

Based on the principles of model 2, a Swedish handbook [3] on plastic design of walls has been developed.
5 FINITE ELEMENT MODEL

5.1 GEOMETRY AND MATERIAL PROPERTIES

Finite element simulations were performed using the commercial software Abaqus. Geometry and material properties were chosen as in paper [4].

No contact been adjacent sheets was assumed to take place in the analyses. In paper [5] it is shown that the influence of this contact between the sheets can be neglected.

5.2 MODELS OF THE JOINTS

5.2.1 Sheathing-to-framing joints

The mechanical properties of the sheathing-to-framing joints are of vital importance for the structural behaviour of shear walls. The properties are based on experiments performed in the direction parallel with and perpendicular to the timber members, see paper [6]. The nonlinear force-displacement curves used in the calculations are shown in Figure 4. In case of unloading, a linear path parallel to the initial stiffness of the considered spring is assumed. In the present study three different models for these joints are used:

- A single spring (SS) model where the joint is represented by only one nonlinear spring using the properties of the perpendicular direction.
- A single spring (SS) model using the corresponding properties of the parallel direction.
- A spring pair (SP) model where the joint is represented by two uncoupled nonlinear springs using the properties of the perpendicular and the parallel direction, respectively.

More information on the different joint models is available in [4].

5.2.2 Stud-to-rail joints

The mechanical properties of a stud-to-rail joint are modelled using two uncoupled springs. One spring acting in the length direction of the rail simulates the shear stiffness of the joint and one spring in the length direction of the stud simulates the tension or compression stiffness of the joint. The force-displacement curves used in the calculations are shown in Figure 5. They are based on experimental results given in paper [7].

![Figure 4: Assumed force-displacement curves for the sheathing-to-framing joints in the parallel and perpendicular to grain direction of the timber members.](image)

![Figure 5: Assumed force-displacement curves for the stud-to-rail joints in shear, tension and compression.](image)

6 COMPARISON OF MODELS

6.1 GENERAL INFORMATION

In order to compare the models previously described in chapter 4 and 5 all wall and load configurations specified in Table 1 and Figure 1 are analysed.

For the two analytical models the distributed plastic capacity $f_p$ of the sheathing-to-framing joints is taken from the force-displacement curves in Figure 4. Using the peak value 994 N and a centre distance between the fasteners of 100 mm means that $f_p = 9.94 \text{kN/m}$ for the parallel direction. The corresponding value for the perpendicular direction is $f_p = 9.64 \text{N/mm}$. This gives the ratio between the two $f_p$ values in the parallel and the perpendicular directions equal to 1.03.

Before starting the comparison of the different models some features of the analytical models and the FE models are highlighted:

- For the analytical models the sheathing-to-framing joints are assumed to be uniformly distributed along the timber members while they for the FE models are located at the real position of each fastener. Since the fasteners were located starting from the corners of each sheet, the FE-models will result in somewhat higher capacity for the walls studied than the analytical models. For the nail spacing chosen
in this study, the capacity obtained by the FE model can be estimated to be 3 – 8 % higher than the capacity given by the analytical model.

- For the analytical models the fasteners are assumed to be located along the outer edges of the sheets while they for the FE models are located at the real position of each fastener. Due to this feature the analytical models will give about 2 % higher capacity than the FE models.
- For the analytical models the assumed force distributions are simplified in relation to the calculated ones obtained by the FE models. The result will be higher capacities for the FE models than the analytical models.
- For the analytical models it is assumed that the peak values of the sheathing-to-framing joints are kept constant for increasing displacements (plastic condition) resulting in higher capacity values for the analytical models than the FE models.
- Analytic model 2 does not always fulfil the conditions of equilibrium giving rise to somewhat too high capacity values for this model compared to analytical model 1 and the FE models.

6.2 COMPARISON OF ANALYTICAL MODELS

Results of calculations, using the analytical models 1 and 2, are presented in Tables 3 and 4. For each model, the influence of two \( f_p \)-values, corresponding to perpendicular and parallel properties of the sheathing-to-framing joints, are studied.

**Table 3: Effective lengths of different walls using the two analytical models (\( b = \) sheet width). Ratios between effective lengths obtained by model 2 and model 1.**

<table>
<thead>
<tr>
<th>Test series</th>
<th>Effective lengths ( L_{ep} )</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test series</td>
<td>Model 1</td>
<td>Model 2</td>
</tr>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>perp</td>
<td>par</td>
<td>perp</td>
</tr>
<tr>
<td>D1</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>D2</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>D2:NA</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>D3</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>D4</td>
<td>4.00</td>
<td>4.00</td>
</tr>
<tr>
<td>H1</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td>H2</td>
<td>0.83</td>
<td>0.83</td>
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<tr>
<td>H3</td>
<td>1.61</td>
<td>1.61</td>
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<tr>
<td>H4</td>
<td>2.47</td>
<td>2.47</td>
</tr>
<tr>
<td>H4:NA</td>
<td>2.47</td>
<td>2.47</td>
</tr>
<tr>
<td>H2V1</td>
<td>1.02</td>
<td>1.01</td>
</tr>
<tr>
<td>H2V2</td>
<td>1.20</td>
<td>1.19</td>
</tr>
<tr>
<td>H2V3</td>
<td>1.53</td>
<td>1.51</td>
</tr>
<tr>
<td>H4V1</td>
<td>2.72</td>
<td>2.71</td>
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<td>H4V2</td>
<td>2.95</td>
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<tr>
<td>H4V3</td>
<td>3.38</td>
<td>3.35</td>
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<td>H4VV1</td>
<td>2.90</td>
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<tr>
<td>H4VV2</td>
<td>3.29</td>
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</tr>
<tr>
<td>H4VV3</td>
<td>3.58</td>
<td>3.57</td>
</tr>
<tr>
<td>H4VV:NA</td>
<td>3.34</td>
<td>3.31</td>
</tr>
</tbody>
</table>

* Not tested

In Table 3 the effective lengths for shear transfer are calculated using Equations (9) and (16). The variable \( b \) denotes the sheet width and is assumed to be equal to \( h/2 \). In case of diagonal loading, corresponding to a fully anchored wall, the effective length is equal to the total length of the wall and is not dependent on whether model 1 or model 2 is chosen.

In the last two columns of the table, the ratios between the effective lengths of model 2 and model 1 are presented for the case of perpendicular and parallel properties of the sheathing-to-framing joints. For the wall configurations studied this ratio is in the range between 1.00 – 1.24. The largest ratios occur when the walls are subjected to horizontal loads only.

A comparison of the effective lengths in Table 3 shows that they are not much affected by the properties of the sheathing-to-framing joints. This is a consequence of the fact that the measured peak values of \( f_p \) are of similar size in the perpendicular and the parallel directions of the timber members.

For the last test series of Table 3 where the bottom rail is not anchored against vertical uplift, i.e. \( \mu = 0 \), the effective lengths are independent of the choice of model which is consistent with the discussions in sections 4.2 and 4.3.

The calculated capacities \( H \) of the wall configurations presented in Table 4 are obtained by multiplying the effective lengths from Table 3 by the plastic shear flow \( \mu f_p \).

**Table 4: Calculated capacities \( H \) of different wall configurations using the two analytical models.**

<table>
<thead>
<tr>
<th>Test series</th>
<th>Calculated capacity ( H )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test series</td>
<td>Model 1</td>
</tr>
<tr>
<td>(1)</td>
<td>perp</td>
</tr>
<tr>
<td>perp</td>
<td>kN</td>
</tr>
<tr>
<td>D1</td>
<td>11.6</td>
</tr>
<tr>
<td>D2</td>
<td>23.1</td>
</tr>
<tr>
<td>D2:NA</td>
<td>23.1</td>
</tr>
<tr>
<td>D3</td>
<td>34.7</td>
</tr>
<tr>
<td>D4</td>
<td>46.3</td>
</tr>
<tr>
<td>H1</td>
<td>2.78</td>
</tr>
<tr>
<td>H2</td>
<td>9.58</td>
</tr>
<tr>
<td>H3</td>
<td>18.6</td>
</tr>
<tr>
<td>H4</td>
<td>28.6</td>
</tr>
<tr>
<td>H4:NA</td>
<td>28.6</td>
</tr>
<tr>
<td>H2V1</td>
<td>11.8</td>
</tr>
<tr>
<td>H2V2</td>
<td>13.9</td>
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<tr>
<td>H2V3</td>
<td>17.7</td>
</tr>
<tr>
<td>H4V1</td>
<td>31.4</td>
</tr>
<tr>
<td>H4V2</td>
<td>34.1</td>
</tr>
<tr>
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<td>H4VV2</td>
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<tr>
<td>H4VV3</td>
<td>41.5</td>
</tr>
<tr>
<td>H4VV:NA</td>
<td>38.6</td>
</tr>
</tbody>
</table>

* Not tested

6.3 COMPARISON OF FE MODELS

The results of performed FE calculations are presented in Table 5 using the three previously described models for the sheathing-to-framing joints. The first three columns (1) - (3) give the capacity \( H \) of the walls and
the two last ones the relative capacity of the walls where
the single spring model with perpendicular properties
for the joints is a reference model with an assigned
relative capacity equal to one.
In section 6.1 it was stated that the ratio between the
two $f_p$ values in the parallel and the perpendicular
directions is equal to 1.03 for the joints tested. This
value should be compared with the relative capacity
values for the wall configurations presented in the
second last column of Table 5 representing the same
two joint models. These relative capacity values are in
the range from 1.03 to 1.14. The main explanation to
these fairly high values is the influence of the shapes of
the force-displacement curves in Figure 4. The force-
displacement curve in the perpendicular direction is
characterised by a post-peak behaviour with a
comparatively steep decrease in capacity in comparison
with the corresponding curve in the parallel direction.
This means that the single spring model with
perpendicular joint properties (SS perp) for the
sheathing-to-framing joints is less effective than the
single spring model with parallel joint properties (SS par) in redistributing the forces at maximum capacity of
the walls studied.

Table 5: Calculated capacities $H$ and relative capacities
of different test series using three FE models for the
sheathing-to-framing-joints.

<table>
<thead>
<tr>
<th>Test series</th>
<th>Calculated capacity $H$</th>
<th>Relative capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
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<td>(2)</td>
</tr>
<tr>
<td></td>
<td>SS</td>
<td>SP</td>
</tr>
<tr>
<td></td>
<td>perp</td>
<td>par</td>
</tr>
<tr>
<td></td>
<td>[kN]</td>
<td>[kN]</td>
</tr>
<tr>
<td>D1</td>
<td>12.4</td>
<td>12.9</td>
</tr>
<tr>
<td>D2</td>
<td>25.1</td>
<td>26.1</td>
</tr>
<tr>
<td>D2:NA</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>D3</td>
<td>37.9</td>
<td>39.4</td>
</tr>
<tr>
<td>D4</td>
<td>50.7</td>
<td>52.8</td>
</tr>
<tr>
<td>H1</td>
<td>3.10</td>
<td>3.20</td>
</tr>
<tr>
<td>H2</td>
<td>10.9</td>
<td>11.8</td>
</tr>
<tr>
<td>H3</td>
<td>20.6</td>
<td>22.9</td>
</tr>
<tr>
<td>H4</td>
<td>31.2</td>
<td>35.5</td>
</tr>
<tr>
<td>H4:NA</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>H2V1</td>
<td>13.5</td>
<td>14.5</td>
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<tr>
<td>H2V2</td>
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<td>H4V1</td>
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</tr>
<tr>
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<table>
<thead>
<tr>
<th>Calculated capacity $H$</th>
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</thead>
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<tr>
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<tr>
<td>H1</td>
<td>1.0</td>
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<tr>
<td>H2</td>
<td>1.0</td>
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<tr>
<td>H3</td>
<td>1.1</td>
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<tr>
<td>H4</td>
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<td>H2V1</td>
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<td>H2V2</td>
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<td>1.0</td>
</tr>
<tr>
<td>H4VV3</td>
<td>1.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calculated capacity $H$</th>
<th>Relative capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>(3)/(1)</td>
<td>(2)/(1)</td>
</tr>
<tr>
<td>(3)/(1)</td>
<td>(3)/(1)</td>
</tr>
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<td>H2</td>
<td>1.1</td>
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<td>H3</td>
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<tr>
<td>H4</td>
<td>1.1</td>
</tr>
<tr>
<td>H2V1</td>
<td>1.1</td>
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<tr>
<td>H2V2</td>
<td>1.1</td>
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<td>H2V3</td>
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<tr>
<td>H4V1</td>
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<tr>
<td>H4V2</td>
<td>1.1</td>
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<td>H4V3</td>
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<tr>
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<td>1.1</td>
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<tr>
<td>H4VV3</td>
<td>1.1</td>
</tr>
</tbody>
</table>

So far only the outcomes of the two single spring
models have been discussed. In the third model called
the spring pair model (SP perp-par) the sheathing-to-
framing joints are modelled with two uncoupled springs
acting perpendicular and parallel to the timber members.
In a non-linear spring pair model of this type the
stiffness and capacity of the joints are overestimated.

Thus it is obvious that this joint model will result in a
wall capacity that is higher than the one obtained by the
SS model with perpendicular joint properties.
A comparison of the load-carrying capacities obtained
by the "SS par" and the "SP perp-par" models shows
that for diagonal load application (D1, D2, D3 and D4)
the "SS par" model gives lower load capacities than the
"SP perp-par" model. The reason is that many of the
fasteners along the perimeter of the sheets are subjected
to displacements both perpendicular and parallel to the
timber members at maximum load (see Figure 6) and
that "SP perp-par" model overestimates the strength in
such situations. In case of horizontal load application
only (H1, H2, H3 and H4) it is seen from Table 5 that
the "SP perp-par" model gives lower wall capacities
than the "SS par" model. The reason is that the fasteners
along the bottom rail on the windward side of these
walls are subjected to substantial vertical uplift (see
Figure 7) giving rise to large force components
perpendicular to the bottom rail. In this situation the "SS
par" model will overestimate the strength since it only
uses the mechanical properties parallel to the timber
members. From Table 5 can further be seen that for the
horizontally loaded wall configurations with a low
portion of vertical load (H2V1, H2V2, H4V1 and
H4VV2) the capacity is overestimated by the "SS par"
model which is in agreement with the wall
configurations subjected to horizontal load only.
Similarly, for the wall configurations with a high
portion of vertical load (H2V3, H4V2, H4V3, H4VV2
and H4VV3) the capacity is overestimated by the "SP
perp-par" model which is in agreement with the wall
configurations subjected to diagonal load.
Provided that the sheathing-to-framing joints are weaker in the perpendicular than in the parallel direction of the timber members, it can be concluded that the “SS perp” model always should result in an underestimation of the wall capacities. It can further be concluded that both the “SS par” and the “SP perp-par” models should result in an overestimation of the capacities implying that the lowest one of them gives the best estimation. For the test series presented in Table 5 this means that the relative capacity of the walls falls within the interval 1.00 – 1.08 where the value 1.00 represents the capacity calculated using the “SS perp” model.

6.4 COMPARISON OF ANALYTICAL AND FE MODELS

In Table 6 a comparison of results obtained by the FE model and the two analytical models of the sheathing-to-framing joints is made. In the FE analysis the single spring model “SS perp” is used. In the analytical analyses, model 1 and model 2 with perpendicular properties for the sheathing-to-framing joints are used. The relative capacities, presented in the last two columns of the table, represent the ratios between the capacities obtained by the analytical models and the “SS perp” model. It is noteworthy that the relative capacity of model 1 is in the range between 0.85 – 0.94 with a mean value of 0.90 and that the relative capacity of model 2 is in the range between 0.91 – 1.12 with a mean value of 1.00. From this information it is obvious that model 1 underestimates the capacity obtained by the FE calculation by about 10 % but that the variation in relative capacity is very low. It is further seen that there is a good agreement between model 2 and the FE model at the mean level but that there is a considerably higher variation in relative capacity for model 2 than for model 1.

Note the differences in the model assumptions, highlighted in section 6.1, and how they may influence the load-carrying capacity of the walls.

7 COMPARISONS BETWEEN MEASURED AND CALCULATED CAPACITIES

In Table 7 the calculated capacities using the “SS perp” model and the two analytical models are presented together with the measured capacities found in the testing of the wall configurations. In the last three columns of the table the relative capacities, here defined as the ratio between measured and calculated capacity, are given. An immediate impression of the presented results is that the measured capacities are considerably higher than the calculated ones. Thus the relative capacities, obtained by the “SS perp” model, fall in the range between 1.11 – 1.45 with a mean of 1.30. The relative capacities, found by model 1, are in the range between 1.21 – 1.64 with a mean of 1.46. Finally the relative capacities, obtained by model 2, fall in the range between 1.12 – 1.52 with a mean of 1.31.

One explanation to the deviations between the measured and calculated capacities is linked to differences in the manufacturing of the sheathing-to-framing joints. At the manufacturing of the specimens for testing of single joints just the hardboard was predrilled. At the manufacturing of the walls, the timber members were also predrilled to some extent since the hardboard was placed on the timber frame when the predrilling took place. In order to study this effect some preliminary tests of sheathing-to-framing joints were carried out with predrilled and non-predrilled timber members. The tests indicate that the predrilling of the timber may increase the capacity of the joints by up to

---

**Table 6: Calculated capacities H and relative capacities of different wall configurations using the FE model (SS perp) and the two analytical models 1 and 2.**

<table>
<thead>
<tr>
<th>Test series</th>
<th>Calculated capacity H</th>
<th>Relative capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mod. 1</td>
<td>Mod. 2</td>
</tr>
<tr>
<td>SS perp</td>
<td>[kN]</td>
<td>[kN]</td>
</tr>
<tr>
<td>D1</td>
<td>12.4</td>
<td>11.6</td>
</tr>
<tr>
<td>D2</td>
<td>25.1</td>
<td>23.1</td>
</tr>
<tr>
<td>D2:NA</td>
<td>-</td>
<td>23.1</td>
</tr>
<tr>
<td>D3</td>
<td>37.9</td>
<td>34.7</td>
</tr>
<tr>
<td>D4</td>
<td>50.7</td>
<td>46.3</td>
</tr>
<tr>
<td>H1</td>
<td>3.10</td>
<td>2.78</td>
</tr>
<tr>
<td>H2</td>
<td>10.9</td>
<td>9.58</td>
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<tr>
<td>H4</td>
<td>31.2</td>
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<tr>
<td>H4:NF</td>
<td>-</td>
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<tr>
<td>H2V2</td>
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</tr>
<tr>
<td>H2V3</td>
<td>20.6</td>
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<td>H4V1</td>
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<td>H4V2</td>
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<td>H4V5</td>
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<td>41.5</td>
</tr>
<tr>
<td>H4V7:NA</td>
<td>-</td>
<td>38.6</td>
</tr>
</tbody>
</table>

* Not tested
20%. Considering also that the “SS perp” model results in somewhat too low capacity values it is not surprising to find that the measured capacities are 30% higher than the calculated ones using the “SS perp” model.

Table 7: Calculated and measured capacities $H$ of wall configurations tested. Relative capacity means here measured capacity divided by calculated capacity.

<table>
<thead>
<tr>
<th>Test series</th>
<th>Capacity $H$</th>
<th>Relative capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1) M.1</td>
<td>(2) M.2</td>
</tr>
<tr>
<td></td>
<td>[kN]</td>
<td>[kN]</td>
</tr>
<tr>
<td>D1</td>
<td>12.4</td>
<td>11.6</td>
</tr>
<tr>
<td>D2</td>
<td>25.1</td>
<td>23.1</td>
</tr>
<tr>
<td>D2:NA</td>
<td>-</td>
<td>23.1</td>
</tr>
<tr>
<td>D3</td>
<td>37.9</td>
<td>34.7</td>
</tr>
<tr>
<td>D4</td>
<td>50.7</td>
<td>46.3</td>
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<td>28.6</td>
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<tr>
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<td>-</td>
<td>28.6</td>
</tr>
<tr>
<td>H2V1</td>
<td>13.5</td>
<td>11.8</td>
</tr>
<tr>
<td>H2V2</td>
<td>16.0</td>
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<td>H4V1</td>
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<td>38.6</td>
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</tbody>
</table>

8 CONCLUSIONS

Two analytical models for design of light-frame walls subjected to in-plane forces are evaluated. The first model (model 1) is a plastic lower bound model implying that the equilibrium conditions are fulfilled for all parts of the walls studied. The second model (model 2) was developed to better utilize the stud-to-rail joints for shear transfer, which were not used for any force transfer in the first model. The basic idea behind this analytical model is that the full vertical shear capacity of the wall should be attained as close to the leading stud as possible, considering that the vertical equilibrium is always fulfilled but disregarding that the horizontal equilibrium is not always satisfied. A weakness of model 1 is that the load-carrying capacity must be calculated iteratively making hand calculations somewhat harder to carry out. A disadvantage of model 2 is that it is not always a lower bound method.

The two analytical models are flexible with respect to load configurations and boundary conditions and can, for example, be applied on walls where the leading stud on the windward side is fully or partially anchored with respect to vertical uplift.

The ratios between the calculated load-carrying capacities of model 2 and model 1 are in the range between 1.00 – 1.24 for the wall and load configurations studied. The largest ratios occur when the walls are subjected to horizontal loads only.

The two analytical models are evaluated by FE calculations using three models for the sheathing-to-framing joints: a single spring model with perpendicular joint properties (SS perp), a single spring model with parallel joint properties (SS par) and a spring pair model with two uncoupled springs (SP perp-par). Provided that the sheathing-to-framing joints are weaker in the perpendicular than in the parallel direction of the timber members, it can be concluded that the “SS perp” model always should result in an overestimation of the wall capacities. It can further be concluded that both the “SS par” and the “SP perp-par” models should result in an overestimation of the capacities implying that the lowest one of them gives the best estimation. For the wall and load configurations studied, this means that the relative capacity of the walls falls within the interval 1.00 – 1.08 where the value 1.00 represents the capacity calculated using the “SS perp” model.

A comparison of the analytical and the FE models shows that model 1 underestimates the load-carrying capacity compared to that obtained by the “SS perp” model by about 10% but that the variation in relative capacity is very low. It is further evident that there is a good agreement between model 2 and the FE model at the mean level but that there is a considerably higher variation in relative capacity for model 2 than for model 1.

Comparing the calculated load-carrying capacities obtained by the “SS perp” model with those obtained in the tests, it is found that the measured values are about 30% higher than the calculated ones. One explanation to this large deviation is that the sheathing-to-framing joints of the wall specimens were predrilled both in the sheets and in the timber while the corresponding specimens of the sheathing-to-framing joints were predrilled only in the sheets. Another explanation is that the “SS perp” is expected to give somewhat too low capacity values.

ACKNOWLEDGEMENT

The authors would like to thank the European Union’s Structural Funds – The Regional Fund for its financial support.

REFERENCES


DEVELOPMENT OF NUMERICAL ANALYSIS METHOD FOR JAPANESE TRADITIONAL WOOD HOUSES CONSIDERING THE SLIDING BEHAVIOR OF COLUMN ENDS

Takafumi Nakagawa¹, Mikio Koshihara², Naohito Kawai³, Yukio Saito⁴, Yoshiyuki Suzuki⁵

ABSTRACT: In Japanese traditional construction method of wooden post-and-beam houses, the columns of the most of houses are just put on the foundation stones and isolated from the foundation. It is said that the sliding behaviour between the column and the foundation stone has the seismic effect against the large earthquake, but it is difficult to estimate and control the sliding behaviour of the column end, because the columns slide independently unlike the general base isolated system. In this study, we developed the numerical analysis method of Japanese traditional wood houses considering the sliding behaviour of the column ends, and the numerical results were verified by the shake table tests of the real-size two storied wood house. The analytical estimation by three dimensional numerical models agreed well with the experimental results.

KEYWORDS: Japanese Traditional Post-and-beam wood house, Shake table test, Distinct element method

1 INTRODUCTION

The one of the main seismic element of Japanese traditional wood houses is regarded as the post and hanging clay walls and moment resistance of joints between frames. In addition, the posts of the most of the old style traditional houses are just put on the foundation stones and isolated from the foundation, so it is said that the sliding behaviour between the posts and stones has the seismic effect against the large earthquake. But it is difficult to estimate and control the sliding behaviour of the post end, because the posts slide independently unlike the general base isolated system. To estimate and design the sliding behaviour of the column ends, some experimental and analytical approaches were made. But the structural design method including the sliding behaviour of column ends have not yet been established.

In this study, we developed the numerical analysis method of Japanese traditional wood houses considering the sliding behaviour of the column ends, and the numerical results were verified by the shake table tests of the real-size two storied wood house. The numerical model is built up by the three dimensional frame, because it is important to trace the sliding and uplift behaviour of the each columns.

2 SHAKE TABLE TEST

The shake table tests of real-size two story wood houses were conducted to clarify the seismic performance of the traditional wood houses. Figure 1 shows the photograph of the specimens. The tests were executed on four specimens, but we focused on the specimen No.4 in this paper. The main seismic components of the specimen were mud plastered walls. The size of the specimen was 10.92x7.28m.

Figure 1: The photograph of shake table specimen

(a) Specimen No.4 (b) Column end and stone

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The input waves are JMA Kobe and BCJ-L2. JMA Kobe was recorded during the South Hyogo prefecture Earthquake in 1995. BCJ-L2 is the artificial wave which acceleration spectrum fit to the large scale earthquake at the 2nd class soil of Japanese building standard requirement.

3 ANALYSIS MODEL

In our analysis, we used the three dimension frame analysis using the software “wallstat” that was developed at Building Research Institute. The calculating theory of wallstat is based on Distinct Element Method (DEM). A detailed explanation of DEM and our theory was given in our previous papers. DEM is a non-continuum analyzing method, so the large deformation analysis of the fracture developing processes is possible. Figure 2 shows the outline of the analysis model. In this analysis, slide control element is added to wallstat. The slider model act as the two dimensional dynamic and static friction force, when the column end contact to the foundation surface, and calculated by the axial force and lateral load of each column at every calculating step.

The plan of this analysis model was based on the specimen used in the shake table test. The wood frames of numerical models were modelled by beam element as indicated in figure 2 (f). The moment of beam elements starts to fall once the maximum bending moment has been exceeded, with transformation into a pinned state occurring at the point in time when 0 is reached, at which point the member is adjudged to have been broken. Vertical shear walls are modelled by the replacement of braces in truss elements as indicated in figure 2 (a). Hysteric shear walls are modelled by the replacement of bi-linear + slip-type models indicated in figure 2(c).

As indicated in figure 2(d), joints are modelled using the rotational spring + elasto-plastic spring (for strong shearing). Hysteric characteristics of the compression / tensile elasto-plastic spring are set as per the one side elastic + one side slip-t

**Figure 2:** The outline of analysis model
4. RESULTS AND DISCUSSIONS

Figure 3 shows the maximum deformation of analytical and experimental results at BCJ-L2 X-direction, Y-direction and JMA Kobe input. Figure 4 shows the time history of displacement of sliding of the corner column end (L1-S1) at BCJ-L2 Y-direction input. Figure 5 shows the aspects of deformation of column ends at the maximum sliding deformation at BCJ-L2 Y-direction input. Calculated column end displacements of analysis model were fairly agreed with experimental results.

**Figure 3a:** The maximum deformation of analytical and experimental results at BCJ-L2 X-direction input.

**Figure 3b:** The maximum deformation of analytical and experimental results at BCJ-L2 Y-direction input.

**Figure 3c:** The maximum deformation of analytical and experimental results at JMA Kobe input.

**Figure 4:** Time history of displacement of sliding of the column end (L1-S1) at BCJ-L2 Y-direction input.

**Figure 5:** The aspects of deformation of column ends at the maximum sliding deformation at BCJ-L2 Y-direction input. (Displacement was multiplied by 20 times)
5. CONCLUSIONS

The numerical analysis results of Japanese traditional wood houses considering the sliding behaviour of the column ends were verified by the shake table tests of the real-size two-story wood house. The analytical estimation by numerical models agreed well with the experimental results. It was found that the analysis results were largely affected by the friction coefficients and shear strength of seismic elements, so the detailed modeling of each seismic element and a lot of parametric studies are required to establish the structural design method of Japanese traditional wood house.

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Seismic behavior and seismic performance improvement on the complicatedly plane shape of wooden house.

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ABSTRACT: After Hanshin-Awaji (Kobe) Earthquake in 1995, the structural design method of the wooden house has been improved by a lot of researches and the aseismic capacity of wooden houses has improved, too. Irregular shape of house is required from the residential quarter circumstances of Japan and the require of a comfortable space. However, neither the earthquake-proof performance evaluation, the design method to answer this, nor the research of the improvement of the seismic capacity are enough yet. In this study, the design problem of the wooden house with a plan of constricted shape is taken up. Seismic behavior of U-shaped house is affected by the patterns of eccentricity of each block.

KEYWORDS: Timber Frames, Plane of Constricted Shape, Stress concentration, Static loading analysis, Eccentricity

1 Introduction

House with U-shaped plan, that is with constricted part of plan as Figure 2 and figure 3 shows, tends to lack of the stiffness of horizontal structure plane and the integrity of the entire building. Therefore, seismic behavior of such building is very complicated and affected by the ratio of shear stress between each block, which may cause the damage of constricted part of the plan.

2 Description of the analytical model

The analytical model is that of U-shaped one story building with 20ton gross weight and natural period of 0.3s. The area ratio of both blocks is "1:1", design strength ratio (Eg.1) for the entire building is constant. By varying the balance between the blocks by adjusting the stiffness of the wall, the following five cases "1:7", "1:4", "2:4", "3:4", "4:4" are examined. Where the block 1 is weak block (the shear stiffness of the wall is low) (see Fig.1, Fig 2). Figure 3 shows a plan view (mm unit). In addition, in the present study one value of eccentricity, 0.3, is assumed for both blocks (see Fig.4, Fig.5). Model is assumed to be within the elastic range, and static loading analysis was executed.

\[
R_d = \frac{Q_d}{q_d} \quad (1) \\
R_r = \frac{Q_r}{q_a} \quad (2)
\]

Rd = design strength ratio \\
Rr = response strength ratio \\
Qd = required shear strength by design \\
Qa = allowable shear strength \\
Qr = shear strength by analytical result

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3 Methods of Loading

Uniformly distributed static force was applied to the floor of the model to be 0.2W(0.2×Weight). There are four cases of the direction of applied force, +X, -X, +Y and -Y(see Fig.6).

4 Description of previous research

Numerical value of Figure 7 shows "Rr (see Eq.2)" of analytical result of each structure plane for the 1:4 model (non-eccentricity). In figure 8 the solid line shows a deformed shape in the model with non-eccentricity. When +X direction loading is applied, it is properties characteristic that the "Rr (see Eq.2)" of each structure plane is uneven. The reason is that the deformation of joints of connecting part is longer for -X loading than +X loading, because the stiffness of joints is higher under compression force than under tensile force. When +Y direction loading is applied, it can be seen that stress state is unbalanced than ±X direction loading. Because consolidation effect was diminished when Y direction loading is applied(see +Y loading,Fig.8). The stress balance of –Y loading revealed almost same tendency with +Y loading.
5 Results of analysis with patterns of eccentricity

5.1 Response strength ratio

5.1.1 X direction loading

Figure 9 shows the maximum "Rr" (see Eq.2) of each structure plane for the Y-axis eccentricity model (see Fig.5) when +X direction loading is applied. In figure 10 the solid line shows the deformed shape in the 3-Y and the 4-Y model (see Fig.5), dotted line in the model with non-eccentricity. It can be seen that the torsional deformation is large compared to the 3-Y model with the model with non-eccentricity (see model 3-Y, Fig. 10). On the other hand, it can be seen that the torsional deformation is small compared to the 4-Y model with the model with non-eccentricity (see Model 4-Y, Fig. 10).

5.1.2 Y direction loading

Figure 11 shows the maximum "Rr" (see Eq.2) of each structure plane for the X-axis eccentricity model (see Fig.4) when +Y direction loading is applied. In figure 12 the solid line shows the deformed shape in the 1-X and the 2-X model (see Fig.4), dotted line in the model with non-eccentricity. It can be seen that the torsional deformation is large compared to the 1-X model with the model with non-eccentricity (see the 1-X model, Fig.12). On the other hand, it can be seen that the torsional deformation is small compared to the 2-X model with the model with non-eccentricity (see the 2-X model, Fig. 12).
5.2 Response strength ratio of the connecting part

5.2.1 X direction loading

$$\beta = \frac{Q_c}{0.2W} \quad (3)$$

\(\beta\) = response strength ratio of the connecting part
\(Q_c\) = existing shear strengh of the connecting part by analytical result
\(W\) = weight

Figure 13 shows the maximum "\(\beta\) (see Eq.3)" of each the connecting part for the Y-axis eccentricity model (see Fig.5) when the +X direction loading is applied. Almost "\(\beta\)" of patterns of eccentricity is higher than "\(\beta\) (see Eq.3)" of the model with non-eccentricity. "\(\beta\)" of the 3-Y model that the opposite phase and the torsional deformation is large is 1.28 times the model with non-eccentricity model. "\(\beta\) (see Eq.3)" of the 3-X model that reduce the torsional deformation is high. Because the block 1 has burden to the block 2 when it is well balanced between the blocks(see the 4-Y model Fig.10).

5.2.1 Y direction loading

Figure 14 shows the maximum "\(\beta\)" of each connecting part for the X-axis eccentricity model (see Fig.4) when +Y direction loading. In figure 15 the solid line shows a deformed shape in the 1-X when the ratio of "\(1/Rd\) (see Eq.1)" is 2:4 model, dotted line in the model non-eccentricity. When the ratio of "\(1/Rd\)" is "1:7", "\(\beta\) (see Eq.3)" of the 3-X model is 1.43 times the model with non-eccentricity. Because the deformed shape of the 3-X model is like "V" shape, it burden on the connecting part. "\(\beta\)" of the 1-X model is lower than the model with non-eccentricity(see Fig.15). Because the torsional deformation is in the same phase and there is no burden to the connecting portion in the deformed shape.

6 Summary

A) The model of patterns with eccentricity has various effects to wood houses with U-shape plan compared with the model with non-eccentricity.
B) In case of the 4-Y and 2-X model, stress concentration can be reduced. Because the location of "center of stiffness" of each block has advantage to reduce the torsional deformation of whole structure.
C) Even if the stress concentration is moderate, it is necessary to increase shear stiffness of connecting part.
E) In future studies, it is necessary to study various patterns of spring stiffness, eccentricity, floor plan and so on.

REFERENCES

ANALYTICAL AND NUMERICAL ANALYSIS OF TIMBER FRAMED SHEAR WALLS

Daniele Casagrande¹, Simone Rossi², Tiziano Sartori³, Roberto Tomasi⁴

ABSTRACT: One of the prominently important performances required for shear walls of wood-framed construction system should be to resist against the lateral forces generated by seismic events. This paper deals with numerical and analytical elastic models for sheathed shear walls. The models are based on the assumption of elastic load-slip relations for both sheathing-to-framing joints and hold-downs or angle brackets, considering only static loads. Both elastic model and analytical formula have been previously experimentally verified through full scale tests. A model suitable to predict the behavior of the wall with different geometry was developed by means of parametric studies. The influence of vertical loads are included in the models. In addition, the contributions of different components are estimated and a simplified formula is presented. The analytical model was amplified in order to deduce the stiffness matrix of a timber walls structure and to understand the real distribution of horizontal loads between the walls: nowadays the wall stiffness in fact is assumed linear proportional to wall length.

A simplified wall numerical model was used in order to run elastic modal analysis of buildings.

KEYWORDS: Wood framed shear walls, seismic performance, wall displacement, elastic model

1 INTRODUCTION

The analysis of a timber framed shear wall subjected to a horizontal load is fundamental in order to understand the behaviour of a building under seismic events. Especially study the interaction among walls becomes necessary to evaluate the seismic forces distribution in an elastic design way of buildings. Nowadays in fact most designs approaches assume that the horizontal force distribution depends linearly on the walls length: sheet shear and sheet nail-slip are considered the uppermost components of wall deformation.

The present work is focused on the analytical and numerical linear elastic analysis of a timber framed shear wall under a horizontal load, characterized by different configurations.

Firstly, an analytical formula to evaluate the horizontal displacement is presented, taking into account all deformation contributions and different configurations (i.e. geometry, type of connections, vertical load). Then a parametric study of wall stiffness is reported: it is demonstrated that a priori linearly proportional to the wall length can not be assumed.

Secondly, a complete linear model of wall is described in order to validate the proposed equation. A simplified numerical model, calibrated on analytical formula, is also reported.

Lastly all results are compared to experimental data obtained by full scale timber frame wall tests performed in the mechanical and structure laboratory of the University of Trento [1].

2 ELASTIC HORIZONTAL DISPLACEMENT THEORETICAL STUDY

The elastic horizontal displacement (Point C, Figure 1) of a timber framed shear wall can be evaluated by adding the different contributions of deformation: sheathing boards, connection system between sheathing boards and timber frame, connection system for lateral forces and hold-down system. The study is conducted considering the presence of an uniformly distributed vertical load q and a concentrated horizontal force F; the wall dimensions are $I \times h$.

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STRENGTH AND SERVICEABILITY - EXTREME EVENTS

2.1 SHEETING BOARD DEFORMATION

The shear deformation component $\Delta_p$ of sheeting boards is independent on vertical load and can be evaluated in the following way. Referring to Figure 2 the shear deformation $\gamma$ is evaluated as:

$$\gamma = \frac{d_x}{d_y} = \frac{F}{G \cdot A} = \frac{F}{(G_p \cdot n_p) \cdot (t_p \cdot l)}$$  (1)

The displacement $\Delta_p$ can be evaluated considering $X=1$:

$$\Delta_p = \gamma \cdot h = \frac{F \cdot h}{G \cdot n_p \cdot t_p \cdot l}$$  (2)

where $n_p$ is the number of sheeting boards (1 or 2) and $t_p$ is the sheeting board thickness.

2.2 SHEET NAIL-SLIP

The sheeting boards are connected to the timber frame by means of a widespread nailing (or stapling). The distance between fasteners along the perimeter studs $s_p$, the top and bottom rails $s_m$ is assumed constant and equal to $s$. Along the intermediate stud the distance is assumed $s/2$.

The horizontal displacement caused by the sheet nail-slip $\Delta_c$ for a single board wall of length $b$ is given by the following equation [2,3]:

$$\Delta_c = \frac{F \cdot h^3}{k} \left[ \frac{1}{\sum_{i=1}^{n_i} x_i^2} + \frac{1}{\sum_{i=1}^{n_i} y_i^2} \right]$$  (3)

where $n$ is the number of fasteners, $k$ is the stiffness of each fastener, and $x_i$ and $y_i$ are the coordinates of each fastener from the centre of the panel.

The deformation due to the nailing is the following:

$$\gamma = \frac{d_x}{d_y} = \frac{F}{G \cdot A} = \frac{F}{(G_p \cdot n_p) \cdot (t_p \cdot l)}$$  (1)

Defining the parameter $\alpha$ as ratio between the height $h$ and the length $b$ of the board, we can write:

$$\sum_{i=1}^{n} x_i^2 = \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha \right) \cdot \frac{h^2}{b^3} = \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha \right) \cdot \frac{a^2 \cdot b^3}{s}$$  (4)

$$\sum_{i=1}^{n} y_i^2 = \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha \right) \cdot \frac{h^2}{b^3} =$$

$$= \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha \right) \cdot \frac{a^2 \cdot b^3}{s}$$  (5)

Defining:

$$\frac{1}{6} \cdot (1 + 3 \cdot \alpha) = \eta$$  (6)

$$\frac{\alpha^2 \cdot (6 + \frac{5}{2} \cdot \alpha)}{12} = \xi$$  (7)

and

$$\sum_{i=1}^{n} x_i^2 \approx \eta \cdot \frac{b^3}{s}$$  (8)

$$\sum_{i=1}^{n} y_i^2 \approx \xi \cdot \frac{b^3}{s}$$  (9)

the displacement $\Delta_c$ is given by

$$\Delta_c = \frac{F \cdot h^3}{k} \cdot \frac{a^2 \cdot \left(1 + \frac{1}{\eta} \cdot \frac{1}{\xi} \right) \cdot \frac{s}{b^3}}$$  (10)

Or, defining
\[ \lambda = \alpha^2 \cdot \left[ \frac{1}{|l|} + \frac{1}{b} \right] \]  

(11)

by:

\[ \Delta_e = \frac{F \cdot b^2}{k \cdot \lambda(\alpha)} \cdot \frac{S}{b^3} \]  

(12)

Extending the result to a \( l \) long wall (sheeted with more \( b \) long panels) then we have:

\[ \Delta_e = \frac{F}{k} \cdot \lambda(\alpha) \cdot \frac{S}{l} \]  

(13)

The parameter \( \alpha \) assumes the role of a board shape coefficient. For \( \alpha > 1 \) the Equation 11 can be simplified and rewritten as:

\[ \lambda = 0.81 + 1.855 \cdot \alpha \]  

(14)

For the typical value \( \alpha = 2 \), \( \lambda \) is 4.52, as proposed by Kälisner et al.

The \( \lambda \) vs \( \alpha \) curve is reported:

\[ \text{Figure 4: } \lambda \text{ vs } \alpha \text{ curve} \]

2.3 RIGID BODY TRANSLATION

The wall is connected to the foundation by means a connection system (angle brackets, screws) in order to prevent a rigid body translation.

\[ \text{Figure 5: rigid body translation} \]

As in the previous cases the vertical load is not influential and, neglecting the friction phenomena, the rigid body translation component \( \Delta_s \) is given by:

\[ \Delta_s = \frac{F}{k_a \cdot n_a} \]  

(15)

With \( k_a \), stiffness of the each connection and \( n_a \) number of connections.

Defining \( l_b \) as the distance between angle brackets (which can be expressed as the ratio between wall length and the number of connections) we obtain:

\[ \Delta_s = \frac{F \cdot l_b}{k_a \cdot l} \]  

(16)

2.4 RIGID BODY ROCKING

The hold-down system is used to prevent the rotation of the wall caused by the overturning moment of the horizontal force \((F \cdot h)\). The hold-down, placed on the inclination angle, is loaded by a force equal to \( F \cdot h/l \).

The hold-down elongation can be evaluated by:

\[ \gamma = \frac{v \cdot h}{l} \cdot \frac{1}{k_h} \]  

(17)

Where \( k_h \) is the stiffness of the hold-downs.

Considering the rigid rotation of the wall, as reported in Figure 6, the rotation angle can be determined using:

\[ \gamma = \frac{v}{l} = \frac{F \cdot h}{l^2 \cdot k_h} \]  

(18)

therefore the rocking component \( \Delta_h \) is:

\[ \Delta_h = \gamma \cdot h = \frac{F \cdot h^2}{l^2 \cdot k_h} \cdot \frac{1}{k_h} \]  

(19)

\[ \text{Figure 6: rigid rotation caused by hold-down elongation} \]

Differently from the previous cases, the hold-down deformation component is influenced by the vertical load, which produces a stabilizing moment:

\[ M = q \cdot \frac{l^2}{2} \]  

(20)

Two different situations are possible:

- The wall rotates because the overturning moment is greater than the stabilizing moment;
- The wall does not rotate because the stabilizing moment is greater than the overturning moment;
Follows that the rocking component $\Delta_h$ can be evaluated as:

$$\Delta_h = \begin{cases} \frac{h}{l \cdot k_h} \left( F \cdot h - \frac{N \cdot h}{2} \right); & \text{for } F \cdot h > \frac{N \cdot h}{2} \\ 0; & \text{for } F \cdot h \leq \frac{N \cdot h}{2} \end{cases}$$

(21)

The first equation refers to the active hold-down and the second one refers to not-active hold-down.

### 2.5 TOTAL HORIZONTAL DISPLACEMENT

To sum up, considering all the deformation contributions and rewriting the vertical distributed load as a concentrated force like you can see on the Figure 7, when the hold-down is active, the displacement of a timber framed shear wall can be evaluated using:

$$\Delta = \frac{F \cdot h}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{N \cdot h} + \frac{F \cdot k_a}{l \cdot k_h} - \frac{F \cdot h^2}{G_p \cdot n_p \cdot t_p + F \cdot k_a \cdot t_p + \frac{F \cdot h^2}{l \cdot k_h}}$$

(22)

In the same way, when the hold-down is not active, the displacement can be evaluated using:

$$\Delta = \frac{F \cdot h}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{N \cdot h} + \frac{F \cdot k_c}{n_p \cdot k_c} + \frac{F \cdot k_a}{l \cdot k_h}$$

(23)

The above equations can be rewritten respectively as follows:

$$\Delta = \frac{F}{K_p} + \frac{F}{K_c} + \frac{F}{K_A} + \frac{F}{K_h} - \frac{N \cdot h}{l \cdot k_h}$$

(24)

$$\Delta = \frac{F}{K_p} + \frac{F}{K_c} + \frac{F}{K_A}$$

(25)

Where $K_p$ is the panel stiffness, where $K_c$ is widespread nailing stiffness, where $K_A$ is the shear connectors stiffness, where $K_h$ is the hold-down stiffness. In addition, $K_{TOT}$ is the wall stiffness and it can be estimated with:

$$\frac{1}{K_{TOT}} = \frac{1}{K_p} + \frac{1}{K_c} + \frac{1}{K_A} + \frac{1}{K_h}$$

(26)

if the hold-down is not active, the wall stiffness is:

$$\frac{1}{K_{TOT}} = \frac{1}{K_p} + \frac{1}{K_c} + \frac{1}{K_A}$$

(27)

Considering the wall stiffness, when the hold-down is active, we can write:

$$\Delta = \frac{F}{K_{TOT}} \cdot \frac{N \cdot h}{l \cdot k_h}$$

(28)

defining:

$$\Delta_N = \frac{N \cdot h}{l \cdot k_h}$$

(29)

so we can write:

$$\Delta = \frac{F}{K_{TOT}} - \Delta_N$$

(30)

Therefore we obtain:

$$\begin{cases} \Delta = \frac{F}{K_{tot}} - \Delta_N \\ F = K_{tot} \cdot \Delta + K_{tot} \cdot \Delta_N = K_{tot} \cdot (\Delta + \Delta_N) \end{cases}$$

(31)

- These equations allow us to make two important considerations: the displacement $\Delta$ is limited from the vertical load, indeed the $F/K_{tot}$ part is decreased by vertical load part $\Delta_N$.
- the force $F$ required to move the wall of $\Delta$ is the sum of the elastic force $K_{tot} \cdot \Delta$ and of the vertical load part $K_{tot} \cdot \Delta_N$.

This behavior is reported in Figure 8:

![Figure 7: wall with concentrated load scheme](image)

![Figure 8: Force F vs displacement Δ linear relationship](image)

In the same way, when the hold-down is not active, the wall displacement is:

$$\Delta = \frac{F}{K_{TOT}}$$

(32)

and the vertical load is irrelevant.

### 3 $K_{tot}$ PARAMETRIC STUDY

From (31) it follows that the force $F$ required depends on the wall stiffness $K_{tot}$. Most design approaches assume $K_{tot}$ linearly proportional to the wall length, without considering its dependence on wall geometry, connection system or vertical loads. At this purpose it is
presented a parametric study of \( K_{tot} \), evaluating how it is influenced by the wall length \( l \).

If the Hold-down is active \( K_{tot} \) can be obtained by

\[
\frac{1}{K_{tot}} = \frac{1}{K_P} + \frac{1}{K_C} + \frac{1}{K_A} + \frac{1}{K_H}
\]

(33)

Explicit

\[
\frac{1}{K_{tot}} = \frac{h}{l} \left( \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot s_e}{n_p \cdot k_c} + \frac{i_a}{l \cdot k_a} + \frac{h^2}{l^2 \cdot k_h} \right)
\]

(34)

Therefore the wall can be schematized by 4 springs in series as shown in the Figure 9.

![Figure 9: Wall rheological model](image)

In order to underline the influence of Hold-down contribution two non-dimensional parameters were defined:

\[
h = \frac{h}{b}
\]

(42)

\[
\phi = \frac{s_e \cdot k_r}{b \cdot k_c}
\]

(43)

The study has been developed for \( h = 1, 2, 3 \) and for values of \( \phi \) from 1.44 to 2.87.

The first parameter \( h \) shows the influence of hold down contribution by the height of the wall; the second one is referred to the stiffness of the hold down connection in relation to the sheet nail-slip stiffness. The influence of the hold down component becomes large in fact when the wall is height and the hold-down connection is weak. The other contributions \( K_P, K_C, K_A \) was evaluated considering the following properties:

- 2 OSB/3 15 mm sheathing boards
- 2.8 x 60 mm ring nails
- Angle brackets \( (k_c = 3342 N/mm) \) [4]
- Angle brackets distances \( l \) equal to 625 mm

The Equation 34 can be rewritten as

\[
\frac{1}{K_{tot}} = \frac{1}{h} \cdot \frac{1}{\theta + 1} + \frac{1}{\beta + 1} + \frac{1}{\varphi + 1} + \frac{1}{\delta + 1}
\]

(39)

Therefore the wall stiffness is given by

\[
K_{tot} = \frac{\theta \cdot \beta \cdot \varphi \cdot \delta \cdot l^2}{(\theta \cdot \delta + \beta \cdot \varphi \cdot \delta + \theta \cdot \beta \cdot \delta) \cdot l + \theta \cdot \beta \cdot \varphi}
\]

(40)

Equation 40 shows as the stiffness wall is not linearly proportional to the wall length \( l \). This is true only when the hold-down are not active. In fact when \( \delta \) goes to infinity we obtain:

\[
\lim_{\delta \to \infty} K_{tot} = \frac{\theta \cdot \beta \cdot \varphi \cdot l}{\beta + \theta \cdot \varphi + \varphi \cdot \beta} = \alpha \cdot l
\]

(41)

According to the previous equation, the non linear proportionality is given by the presence of the active hold-down; the stiffness \( K_H \) is in fact proportional to the squared wall length \( l^2 \).

In order to understand the influence of the contribution of hold-down a parametric study of wall stiffness is here presented.

Defining \( K_{tot,b} \) as the stiffness of a wall long as the board length \( b \), the relationship \( K_{tot}/K_{tot,b} \) vs \( l/b \) are shown in Figures 10-13. The dashed line stands for the linear proportional wall stiffness dependence on wall length.

![Figure 10: Behavior dimensionless for \( \Phi = 0.72 \)](image)

![Figure 11: Behavior dimensionless for \( \Phi = 1.92 \)](image)
According to figures 10-12, when the wall height increases and the parameter $\Phi$ decreases the continuous lines come off the dashed line.
As example, assuming $h=2$ and $\Phi=1.44$ and for the above assigned value of $K_p$, $K_c$, a wall long 4 times $b$ is 6 times stiffer than a wall with a length equal to $b$, and not 4 times.
Therefore the wall stiffness cannot be considered a priori linear proportional to the wall length.

4 WALL NUMERICAL ANALYSES

In an effort to validate the results of analytical and theoretical studies a numerical model was developed using program SAP 2000.
The analysis was performed by means of a wall complete model, where all sheet nail-slip is modelled by means of a linear link.
After checking the agreement between the numerical and theoretical results a simplified model is proposed.

4.1 COMPLETE NUMERICAL MODEL

The elastic behavior of a framed timber wall loaded by a horizontal load was investigated using a numerical model using software SAP 2000. This model is called “complete” unlike the “simplified” one, developed in order to perform numerical analyses of buildings.
Pinned frame elements have been utilized to model the timber frame, which should therefore be regarded as a mechanism: the sheathing boards, modeled through shell elements, guarantee the stability of the wall. The frame elements are connected to the shell points through linear elastic springs (Joint link) simulating the behavior of nailed connections. The bottom beams is then connected to the foundation by means of Joint linear link in order to model Hold-Down and angle brackets and endowed with parameters validated through a previous experimental campaign.

Figure 13: complete numerical model

From performed analyses, the proposed equation is able to predict more accurately the importance of the single elastic components with respect to the existing formulations [5].

4.2 SIMPLIFIED NUMERICAL MODEL

After validating the proposed equation to predict the horizontal displacement of a framed timber wall by means of a complete numerical model, a simplified model is proposed in order to perform elastic analysis on timber framed buildings. This model allows an easier and faster analysis of a building without losing accuracy compared to the complete one.
The shear deformation components (sheet nail-slip and sheeting board deformation) are modelled by means of an equivalent linear link which stiffness, $K_{sheq}$ is evaluated using the follow analytical equation:

$$K_{sh} = \frac{K_c \cdot K_p}{K_c + K_p} \quad (44)$$

Two different simplified models are proposed, where the equivalent linear link has two different orientation. In the first one a diagonal spring is used and in the second a vertical link whose lateral stiffness is defined.

Figure 14: simplified numerical models

It is important to underline that in the second model the stiffness of timber framed has to be considered infinity to prevent the bending deformation of the beams.
In the first one model the stiffness of the diagonal spring can be obtained as:
Both models have a vertical spring for Hold-down contribute ($K_v$) and a horizontal one for translation contribute ($K_h$). Vertical loads can be designed as spread or concentrated loads on the top beam.

5 VALIDATION OF RESULTS

After checking the theoretical equation by means of a complete numerical model, a comparison with experimental tests on full scale walls results was developed. Displacements under different intensities of the horizontal load were predicted using the analytical formula and compared with displacement of load vs displacement curve of tests. In particular the results of two types of walls are reported.

\[
K_{th,h} = \frac{K_{th}}{\cos^2(\theta)}
\]

(45)

The parametric study of the lateral wall stiffness shows that there is not a linear proportionality with the wall length cause the hold down deformation component. It is also very important take account of vertical loads since they reduce the horizontal displacements and at the same time increase the lateral force requested to get a specified displacement. Analytical formula was validated with experimental results obtained by full scale wall tests.

ACKNOWLEDGEMENT

The presented research has been carried out within the framework of the RELUIS Project which is financed by the Italian Emergency Management Agency (Protezione Civile).

REFERENCES


Figure 15: load vs displacement

The results show a good correlation between analytical prediction and experimental results. It is important underline that the analytical formula is not created to fit the horizontal force vs displacement curve but to predict the elastic displacement under a specified load.

6 CONCLUSIONS

A theoretical formula to predict the horizontal displacement of a timber framed wall is reported. This can be considered very useful in order to predict displacements under a horizontal force and to calibrate a simplified model of the wall for elastic analysis (i.e modal analysis).
FULL SCALE STATIC LOADING TESTS FOR TWO STORIED PLANE FRAME OF TRADITIONAL TOWN HOUSE IN KYOTO, JAPAN

Yasuhiro Hayashi¹, Noriko Takiyama², Shinichi Hirosue³, Takuya Matsumoto⁴, Atsushi Nakagawa⁵

ABSTRACT: We have conducted static loading tests of plane frame of a two-storied traditional town house in Kyoto, Japan. In our study, we have developed a lateral loading system in order to apply large shear deformation angle of 0.2 rad. Based on our tests, the deformation performance and stress transferring mechanism of town houses are well understood. Concentration of deformation or damage in a specific story will be prevented by the existence of pillars and toriniwa portion.

KEYWORDS: Static loading tests, Traditional town house, Two-storied plane frame

1 INTRODUCTION

There are many traditional wooden town houses forming the historical townscape in Kyoto, Japan. On the other hand, there are a lot of old wooden buildings collapsed in the Hyogo-ken Nambu, Kobe earthquake in 1995. Therefore, it is important to carry out seismic retrofit of town houses in Kyoto but their seismic performance (horizontal loading capacity and deformation capacity) are not clarified enough. Supposing a two-storied town house in Kyoto, in this research, we have conducted horizontal static loading tests of a plane frame which passes in the ridge direction. Then, the response behaviour of town houses during strong ground motions is discussed based on our test results.

2 OUTLINE OF EXPERIMENTS

2.1 SPECIMENS

In order to understand the structural properties of a two-storied town house, five specimens are used for the static loading tests as shown in Fig.1. A plane frame, which passes the central pillar, daikoku-bashira, in the ridge direction, is called as Specimen-D. The height and width of Specimen-D are about 5.2m and 5.8m, respectively. It is one of the evident features of town houses in Kyoto that there are many through columns (pillar). A mud wall is placed in the first floor of rooms. The type of joints, the dimensions and material of members are determined based on the specification used in actual traditional town houses, in Kyoto. Dimensions and material of principal members are listed in Table 1. The Specimen-D can be divided into the room (Specimen-DR) and a narrow street with open ceiling space called toriniwa (Specimen-DT). Furthermore, Specimen-DR1 is also divided into the first floor (Specimen-DR) and second floor (Specimen-DR2). Pillars are placed on the natural stones and are not fixed at all. Therefore, the sliding or uplift of pillars is free.

2.2 LOADING SYSTEM

Cyclic loading tests with gradually increasing displacement amplitude are conducted. The height of specimen is 5.255m. Therefore, the maximum half amplitude of displacement at the roof level is about 1.0m
in order to make the maximum shear deformation angle up to 0.2 rad. To apply such a large deformation, the following loading system has been developed.

The loading frame consists of two plane frames. Each frame consists of two steel pillars and two horizontal steel beams and is connected each other so that their movement should be same. Pillars are supported by pin joints at the bottom, and horizontal beams are also connected to pillars by pin joints. Therefore, loading frame is unstable in the in-plane direction. In order to apply the horizontal displacement to a specimen, specimen is connected to loading frame through load cells at roof level or second floor level. In-plane horizontal displacement of a specimen is controlled by a hydraulic jack, whose stroke is 1m, connected to pillars. We perform the loading by amplifying displacement using the principle of the lever as shown in Fig. 2. The loading process is shown in Fig. 3. The deformation angle $R$ is the horizontal displacement of the top of the specimen divided by the height 5.255 m. The positive and minus deformation angles are loading in the right and left directions, respectively.

Weights corresponding to the fixed load and movable load of a town house are set on the top and second floor.

**Table 1: Dimension and material of members**

<table>
<thead>
<tr>
<th>Member</th>
<th>Dimension (mm)</th>
<th>Wood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>Daikoku-bashira</td>
<td>150×150</td>
<td>Japanese cypress</td>
</tr>
<tr>
<td>Side post</td>
<td>120×120</td>
<td>Japanese cypress</td>
</tr>
<tr>
<td>Stud</td>
<td>105×105</td>
<td>cedar</td>
</tr>
<tr>
<td>Girder</td>
<td>105×240</td>
<td>pine</td>
</tr>
<tr>
<td>Roof beam</td>
<td>120×180</td>
<td>pine</td>
</tr>
<tr>
<td>Purlin</td>
<td>105×105</td>
<td>cedar</td>
</tr>
<tr>
<td>Joist</td>
<td>90×90</td>
<td>cedar</td>
</tr>
<tr>
<td>Roof stud</td>
<td>90×90</td>
<td>cedar</td>
</tr>
</tbody>
</table>

**Table 2: Vertical load and axial force of columns**

<table>
<thead>
<tr>
<th>Vertical load (kN)</th>
<th>2F</th>
<th>1F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Weight</td>
<td>1.6</td>
<td>1.1</td>
</tr>
<tr>
<td>Weight</td>
<td>30.2</td>
<td>16.6</td>
</tr>
<tr>
<td>Specimen Weight</td>
<td>3.4</td>
<td>3.3</td>
</tr>
<tr>
<td>Weight</td>
<td>13.4</td>
<td>13.4</td>
</tr>
<tr>
<td>Specimen</td>
<td>3.4</td>
<td>3.4</td>
</tr>
<tr>
<td>Weight</td>
<td>13.4</td>
<td>13.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Axial force (kN)</th>
<th>2F</th>
<th>1F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>Weight</td>
<td>13.5</td>
<td>13.5</td>
</tr>
<tr>
<td>Specimen</td>
<td>c</td>
<td>d</td>
</tr>
<tr>
<td>Weight</td>
<td>23.2</td>
<td>16.2</td>
</tr>
<tr>
<td>Specimen</td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>Weight</td>
<td>7.3</td>
<td>8.1</td>
</tr>
<tr>
<td>Total (kN)</td>
<td>48.7</td>
<td>34.4</td>
</tr>
</tbody>
</table>

Photos and diagrams are shown for visual representation.
level of specimens. The fixed load (weight) and movable load (the weight of specimen) is listed in Table 2. The axial force for each column is calculated and listed in Table 2. Total weight of Specimen-D is about 50 kN. And the axial force for the daikoku-bashira of Specimen-D, column ‘c’ in Table 2 is about 23 kN. Therefore, half of the total weight is applied to daikoku-bashira.

3 TEST RESULTS AND DISCUSSIONS

3.1 DAMAGE PROGRESS

First, damage progress of a mud wall and frames as deformation increase is investigated. The progress of typical damage to the Specimen-D is shown in Fig. 4 and typical damage condition of beam-column joints and uplift status of column base are shown in Photo 2. A mud wall starts cracking around horizontal members inside the mud wall at 1/20 rad. and the center of mud wall cracks in a vertical direction at 1/15 rad. End failure at the tenon of beams starts to occur at the rotational angle of 1/30 rad. as shown in Photo 2(c).

Splitting failure occurs in the fiber direction near the cotter slot of a beam-column joint at 1/15 rad. as shown in Photo 2(e). Although capital tenon and tenons at beam-column joints fractured, but no pillar does not break at all until it reaches 1/8 rad. of deformation angle. Figure 5 shows the relationship between the 1st story deformation angle and 2nd story deformation angle. The 2nd story deformation angle is 2.5 times as large as 1st story deformation angle at 1/75 rad. But the the deformations of the first and second stories tend to be nearly equal as deformation angle increases. This tendency is due to the deformation equalization effects of pillars such as daikoku-bashira and toriniwa, and failure in the mud wall and beam-column joints. Namely, the toriniwa and pillars play a role to prevent damage concentration to the first or second story.

Next, the relationship between horizontal load applied to the top of Specimen-D and deformation angle is shown in Fig. 6. Resisting force of Specimen-D disappear around the deformation angle of 1/8 rad. due to the PΔ effects.
3.2 RESTORING FORCE CHARACTERISTICS
Figure 7 shows the skeleton curves in the right and left directions. Restoring force in the left direction is 1.5 times as large as that in the right direction. The asymmetry of skeleton curve is mainly due to the asymmetrical location of mud wall, namely asymmetrical column base uplift and mud wall failure as shown in Fig. 4.
Figure 8 shows the comparison between Specimens-D, Specimen-DT and Specimen-DR. There is almost no difference in skeleton curves of Specimen-D and DR, and the maximum strength of Specimen-DT is very small. Namely, the restoring force of toriniwa is very small. However, as described before, the toriniwa and pillars play a role to prevent damage concentration to the first or second story.

3.3 BEHAVIOURS OF COLUMN BASE
Uplift and sliding behaviour of columns for Specimen-D are shown in Figs. 4 and 9. The uplift of column base occurred around a mud wall due to the tensile force induce by the rocking of the mud wall. Therefore, the uplift of side pillar, ‘a’ in Fig. 9(a), occurred under positive (rightward) loading, but the uplift of stand column, ‘b’ in Fig. 9(a), occurred under negative (leftward) loading. Column base uplift increases as shown in Fig. 9(a), as the deformation angle increases up to 1/20 rad. or 1/15 rad. After that, the uplift decreases after the mud wall is heavily damaged. The maximum uplift of side pillar is about 21mm and is much larger than that of stand column which is restricted to move upward by a beam.
On the other hand, frame does not slide as one-body system as shown in Fig. 9(b). This is because the ratio of the horizontal loading capacity to the total weight is about 0.09 and is less than friction coefficient of about 0.4 between wood and stone. However, some columns with small axial force slide independently. Especially, the sliding of side pillar is predominant after the mud wall is heavily damaged. The axial force of side pillar is not so large and the pillar moves so that the width of mud wall should spread.

3.4 STRESS TRANSMITTING MECHANISM
The shear force of columns is calculated from the bending moment identified using strain gauges stuck on columns as shown in Fig. 10(a). The 1st story shear force is equal to the 2nd story shear force, because horizontal load is applied to the top of frame. Therefore, the shear force of the mud wall is calculated from those of columns as shown in Fig. 10(b). Shear force distributions and bending moment diagram for representative deformation angles are shown in Figs. 10(c) and 11, respectively. The maximum shear forces of mud wall in the right and left loading directions are about 7kN and 9kN, respectively, as shown in Fig. 10(b). Therefore, the maximum shear force of mud wall is larger than the maximum restoring force because there is an oppositely-directed shear force to the loading direction in the daikoku-bashira pillar as shown in Figs.
10(a), 10(c), and 11(a). In addition, the oppositely-directed shear force decreases as the damage of mud wall become severer as shown in Figs. 10(a) and 11(b). Therefore, it is considered that the oppositely-directed shear force is due to the existence of the mud wall.

3.5 COMPARISON WITH CURRENT DESIGN METHOD

Finally, the 1st story shear forces obtained from our experimental test results and estimation by the current design method are compared in Fig. 12. The experimental result is calculated from the load-deformation relationship by excluding the $P\Delta$ effects. Estimation by the current design method is based on the summation of skeleton curves for unit structural elements. Therefore, it is pointed out that the asymmetrical restoring force characteristics with respect to the loading directions are not considered in the current design. Furthermore, the loading capacity estimated from the current design method drastically decreases due to the failure in the mud wall. However, according to the experimental results, the plane frame holds the restoring force of 5kN or more even at the deformation angle of 1/10 rad.
4 CONCLUSIONS

We have conducted static loading tests of plane frame of a two-storied traditional town house in Kyoto, Japan. In our study, we have developed a lateral loading system in order to apply large shear deformation angle of 0.2 rad. Based on our tests, the deformation performance and stress transferring mechanism of town houses are well understood as follows.

(a) No pillar breaks until the horizontal restoring force is lost around the deformation angle of 1/8 rad. due to the $P\Delta$ effect.
(b) Concentration of deformation or damage in a specific story will be prevented by the existence of pillars and torinawi portion.
(c) Since the ratio of the horizontal loading capacity to the total weight is less than 0.2, specimen frame does not slide as one-body system. However, some columns with small axial force move independently.
(d) The following phenomena, which are neglected in the current seismic design method for the traditional wooden buildings in Japan, have been observed.
   1) Uplift of columns near a mud wall
   2) Asymmetric restoring characteristics
   3) Oppositely-directed shear force to the loading direction

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SEISMIC RESPONSE OF POST-TENSIONED LVL WALLS COUPLED WITH PLYWOOD SHEETS

Asif Iqbal, Stefano Pampanin, Massimo Fragiacomo, Alessandro Palermo, Andrew Buchanan

ABSTRACT: Laminated veneer lumber (LVL) structural members have recently been proposed for multi-storey timber buildings based on ongoing research at University of Canterbury, New Zealand. The members are designed with unbonded post-tensioning for recentering and ductile connections for energy dissipation.

This paper describes the experimental and numerical investigation of post-tensioned LVL walls coupled with plywood sheets under both quasi-static cyclic and pseudo-dynamic seismic testing protocols. It is observed that energy is dissipated mostly through yielding of the nails, and the LVL walls return close to their initial position while remaining virtually undamaged. Different arrangements of nails have been tested to compare their energy dissipation characteristics. Numerical models with appropriate elements to represent the contact interface at the base as well as simulating post-tensioning and hysteresis have been developed to predict the behaviour of the systems and they match the experimental results with good accuracy.

The results indicate good seismic performance, characterized by negligible damage of the structural members and very small residual deformations. The only component significantly damaged is the nailed connection between the plywood sheet and the LVL walls. The plywood can be easily removed and replaced with new sheets after an earthquake, which is reasonably cheap and easy to install, allowing for major reduction in downtime. Other advantages include low mass, flexibility of design and rapidity of construction. Post-tensioning of structural members involves low-cost and can be easily arranged on site. With all these benefits together, the concept has potential for consideration as an alternative solution for multi-storey timber buildings.

KEYWORDS: Prestressed Timber, Coupled Shear Walls, Seismic Response, Plywood Connections

1 INTRODUCTION

Recent developments in the seismic design of reinforced concrete structures have produced new design approaches for moment-resisting frames and walls. High-performance (still low-cost) seismic resisting solutions, namely jointed ductile connections, have been developed under the U.S. PRESSS (PREcast Structural Seismic Systems) programme coordinated by the University of California, San Diego [1-2] for the seismic design of multi-storey precast concrete buildings. Such solutions are based on “dry” joints between pre-fabricated elements and unbonded post-tensioning techniques. A particularly efficient solution is given by the “hybrid” system [3] which combines unbonded post-tensioned tendons with damping/energy dissipating devices. The unbonded post-tensioning provides a re-centering capacity (in addition to the axial load contribution, when present), while the inelastic deformation is concentrated at the critical section which undergo a “controlled rocking” motion. The dissipation capacity is provided by sacrificial fuses, which can consist of mild steel internal bars or external dissipation devices. A flag-shape hysteretic behaviour (Figure 1) is a typical feature of such hybrid systems.

As a result, an extremely efficient structural system is obtained which can undergo large inelastic displacements similar to their traditional counterparts...
(monolithic connections), while limiting the damage to the structural system and assuring full re-centring capacity after the seismic event (thus no residual or permanent displacement).

After the original developments of hybrid solutions in precast concrete structures, the concept has been recently applied in timber structures [4-7], showing that the PRESSS-technology can be successfully implemented regardless of the properties of the material used.

In particular, when considering the possible application of hybrid system to low rise multi-storey timber construction, Laminated Veneer Lumber has been shown to be a suitable material since it has a high level of homogeneity and also exhibits superior strength characteristics when compared to rough sawn or glulam timber. Preliminary tests, carried out by Palermo et al. [4-7], proved the feasibility of hybrid systems for multi-storey timber construction, considering moment-resisting frame systems or cantilever walls. This paper extends the hybrid concept to coupled walls with particular focus on hysteretic connections consisting of nailed plywood sheets.

Figure 1: Idealized “flag-shaped” hysteresis behaviour in a hybrid connection[8-9]

2 DEVELOPMENT OF HYBRID SOLUTION FOR COUPLED LVL WALLS

As part of a comprehensive research investigation for the development of innovative seismic resisting systems for timber construction, a number of different hybrid solutions for frame and wall systems have been successfully tested for implementation in multi-storey LVL buildings [4-7]. Either internal or external energy dissipation devices have been implemented for both beam-to-column, column-to-foundation or wall–to-foundation connections.

These tests demonstrated the feasibility of developing high performance, low-cost seismic resisting LVL frames or walls for low-medium rise multi-storey construction. When focusing on “prefabricated” wall systems, further improvement can be achieved, by coupling the walls using transverse rocking dissipative connections [10] or alternative energy dissipation devices [1-2].

In order to improve the cyclic behaviour of wall or panels systems, research activities has been carried out using single [1] or coupled [10] solid timber walls constructed with LVL.

3 BEHAVIOUR OF WALLS WITH NAILED PLYWOOD CONNECTIONS

Typical seismic-resisting systems adopted for multi-storey timber construction usually involve the use of plywood panels to gain shear capacity. As the wall moves, strength and ductility are provided by the deformation of nails holding the plywood to the studs and plates. The wall system acts like a vertical cantilever beam where shear is resisted by the panel sheathing and the connection to the chords, while the bending capacity is provided by the lateral chords in tension and compression (Figure 2).

As a result, the cyclic loading of the wall produces typical pinched hysteresis loops, due to the behaviour of the nailed connection, reducing the stiffness and energy dissipation and leading to larger displacements in subsequent cycles. The tendons provide re-centering capacity in post-tensioned walls, ensuring almost small residual displacements. Another advantage of externally attached plywood sheets is related to the possibility to replace or repair them after an earthquake event and the accessibility for maintenance during the life-time of the building.

Figure 2: Force transfer in a traditional plywood shear wall [11]

The method of dissipation herein presented is two 12mm sheets of plywood placed against the faces of the two walls and nailed around the perimeter of the plywood sheet. This sort of dissipater is cheaper than the external fuse-type dissipater; the movement between the walls causes plastic deformation of the nails which provides the energy dissipation contribution to the hybrid system. Overstrength design has been adopted to ensure that the nails are the “weakest element” and that failure of the plywood sheet does not occur. The spacing of the nails was set to ensure that the system was sufficient to provide enough self-centering and dissipation contribution.
4 EXPERIMENTAL TESTING OF LVL WALLS COUPLED WITH PLYWOOD

The setup of the coupled walls is shown in Figure 3; the same specimen has been used for all the testing illustrated in this work. The walls are 2.5m high, 0.78m long and 0.195m thick. They are loaded at a height of 2m above the foundation, i.e. at the point of contra-flexure of the full-scale system. The walls are constructed from three panels of LVL 65mm thick, glued together with epoxy and then nailed to hold them in place while the epoxy sets. The material properties are given in Table 1, based on specific material testing. Four unbonded post-tensioned tendons, two per wall, were stressed to an initial force of 43.5kN (0.32 f_p); the low design value comes out from the prevention of possible yielding of the tendons; in fact the geometrical configuration of the wall section, during the test, induces marked elongation in tendons for negligible increment of gap opening.

Table 1: Material properties of test specimens[12]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength parallel to grain</td>
<td>45 MPa</td>
</tr>
<tr>
<td>Elastic Modulus of LVL</td>
<td>13.2 GPa</td>
</tr>
<tr>
<td>Shear Modulus of LVL</td>
<td>660 MPa</td>
</tr>
<tr>
<td>Yield stress of prestressing strand</td>
<td>1560 MPa</td>
</tr>
<tr>
<td>Ultimate stress in prestressing strand</td>
<td>1870 MPa</td>
</tr>
</tbody>
</table>

Both quasi-static cyclic and pseudo-dynamic testing were carried out. Details of the specimens are given in Table 2. The loading protocol adopted for quasi-static cyclic testing is a modification of ACI T1.1-01, ACI T1.1R-01 [13], proposed for the testing on innovative jointed precast concrete frame systems. The modification maintained the target drift levels, but reduced the number of cycles, i.e. from three to two cycles, for each level of intensity. Pseudo-dynamic testing was performed to simulate the dynamic response of the structural system subjected to an earthquake input ground motion and to show the effect of hysteretic damping and re-centering properties on the overall response (maximum and residual displacements). The same test set-up as the quasi-static testing, was adopted. As the wall was 2/3 scale, assuming constant density criterion, an amplification of 3/2 was applied to the original accelerograms while the duration (time) was scaled by 2/3. Details of the earthquake record adopted are shown in Table 3.

Table 2: Properties of specimens tested

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Init.PT, kN(%yield)</th>
<th>Nail Spacing</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Quasi-static</td>
<td>43.5(30)</td>
<td>100mm</td>
<td></td>
</tr>
<tr>
<td>Quasi-static</td>
<td>43.5(30)</td>
<td>50mm</td>
<td></td>
</tr>
<tr>
<td>Pseudo-dynamic</td>
<td>43.5(30)</td>
<td>50mm</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3: Test setup of coupled wall specimens

Figure 4: View of coupled walls with plywood sheet

5 TEST RESULTS FOR COUPLED LVL WALLS

Results of two quasi-static tests are presented in Figures 5 and 6. As expected, the amount of energy dissipation increased with reduced nail spacing clearly visible from the shape of the hysteresis loops. Equivalent viscous damping of about 5% to 7% of critical was achieved. Negligible residual deformation was observed at the end of both the tests. The pseudo-dynamic test result is presented in Figure 7 which also indicates significant energy dissipation during the test.
A reduction in the stiffness occurred during the test due to the stiffness reduction of the nailed plywood dissipaters, leading to less energy dissipation contribution. In spite of the reduced seismic performance, this method of dissipation is cheaper and simpler to attach compared to the aforementioned fuse-type dissipaters.

6 ANALYTICAL STUDY

A numerical model of the wall-to-foundation connection validated the experimental results. Different modelling approaches and techniques for hybrid precast concrete structures are available [14-15] that are equally applicable in this case. A simple analytical approach based on section analysis and multi-axial spring concepts have been followed here.

Moment-rotation behaviour of the wall-to-foundation connection is calculated by the analytical procedure called Monolithic Beam Analogy for jointed ductile connections originally proposed by Pampanin et al. [16] and subsequently refined by Palermo [17], which relies on a member compatibility condition in terms of displacements between a monolithic and a hybrid solution. The combined contributions from the prestressing tendons and the energy dissipaters in a hybrid connection are modelled by two springs in parallel with appropriate characteristics to produce the flag-shaped hysteresis. Adopting this concept, simple numerical models have been developed and applied in this research.

The Multi spring model is characterised by representation of contact in the critical section (wall-to-foundation) with a multi-spring element [18, 19]. The multi-spring model simulates the contact section interface with a number of axial springs. The multi-spring contact element is set up for 2 to 10 contact points, representing the position of the springs. Two different integration schemes, i.e. Gauss quadrature and Lobatto integration, can be used to optimise the position of the springs and calculate their weight.

The model achieves a good simulation of the local stresses, strains, variation of the neutral axis position at joint opening and as well as allows considering the beam elongation effects. The characteristics of the springs can be properly chosen considering the different contact (unilateral, bilateral) behaviour of the section. The other elements characterising the hybrid connection, i.e. the unbonded post-tensioned cables and the energy dissipaters, are modelled with longitudinal springs. The hysteretic rule for the unbonded post-tensioning cable can be assumed non-linear elastic, if the cables do not reach the yielding point, while for the energy dissipaters a proper hysteretic loop has to be chosen depending on the type of energy dissipater. The walls are represented by elastic finite beam elements. Figure 8 shows schematic model of the coupled wall specimen. Wayne Stewart hysteresis model used to model the nailed plywood connections. Comparison between results from the model and that of experiment under symmetric test protocol is shown in Figure 9.
7 CONCLUSIONS

The efficiency of using nailed plywood sheets as hysteretic dampers to couple post-tensioned LVL walls has been herein presented. Preliminary results of the experimental investigation confirmed the performance of such a hybrid system. The configuration of the plywood sheets allows exploiting the rocking behaviour of wall systems and translates it into an energy dissipation mechanism. The arrangement is simple and easy. Specimens with different nail configurations were tested highlighting the significant design flexibility of the system. Effectiveness of hybrid systems is evaluated by the re-centering ratio, as mentioned in NZS3101 - Appendix B [9], defined by the ratio between restoring moment due to the post-tensioning load on the wall and moment provided by the plywood sheets. For the two test specimens with 100mm and 50mm nail spacing the value of the ratio was about 4 and 2 respectively which indicate that significant energy dissipation as well as complete re-centering was achieved although the amount of damping is lower than that achieved with alternate types of connections.

Virtually no damage is observed in the original structural members after a number of cycles of seismic loading, which ensures low cost for post-earthquake repairs. The repair costs of the system, after a major earthquake, consist of the replacement of the only sacrificial elements, i.e. nails or parts of the plywood sheets, which is a low-cost operation. As confirmed by the results, the proposed arrangement has the potential to be a good option for achieving seismic resistance in multi-storey timber buildings with coupled wall systems.

ACKNOWLEDGEMENT

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STUDY ON SEISMIC DIAGNOSIS FOR WOODEN STRUCTURES
PART 2: RELATIONSHIP BETWEEN EVALUATION OF SEISMIC DIAGNOSIS AND DAMAGE LEVEL

Chikako Tabata¹ and Yoshimitsu Ohashi²

ABSTRACT: This paper presents the relationship between the seismic diagnoses evaluations developed by the Japan Building Disaster Prevention Association (JBDPA) and the actual earthquake damage levels. First, 111 conventional wooden houses damaged by the 1995 Hyogoken-nanbu earthquake are diagnosed. Then, the damage levels are assessed on the basis of feedback to questionnaires received from residents of these houses. The analysis revealed that the revised-detailed diagnoses provide values that are significantly proportional to the damage levels.

KEYWORDS: Seismic diagnosis, Damage level, Conventional wooden house, 1995 Hyogoken-nanbu earthquake

1 INTRODUCTION
In Japan, there are several sources of seismic diagnoses. Among them, the diagnosis published by JBDPA is used most widely. This diagnosis was first issued in 1979, and subsequently revised in 1985 and 1995. The enlarged 1995 edition [1] involved two methods: simplified and detailed diagnoses.
In 2000, the Building Standard Law of Japan was revised. The enlarged 1995 edition was further revised in 2004 [2].

2 SUBJECTS
The subjects of the diagnoses are conventional wooden houses, which are post and beam constructions, that were damaged by the 1995 Hyogoken-nanbu earthquake. Investigations were conducted on 111 two-storey houses situated in Takarazuka and Nishinomiya as well as Kobe and Tsuna in Hyogo Prefecture. These houses were built in 1963-1995.

3 METHOD OF SEISMIC DIAGNOSIS
3.1 ENLARGED 1995 EDITION
The enlarged 1995 edition includes two types of diagnoses, namely ‘simplified’ and ‘detailed’ diagnoses. In this paper, they are referred to as ‘enlarged-simplified (ES)’ and ‘enlarged-detailed (ED)’. ES is used in cases where the position of a shear wall is unknown, whereas ED is used in cases where the position of a shear wall is specified. In addition, ES covers two cases. When shear walls exist, ‘ES with braces (ESB)’ is applied. On the other hand, when no shear walls exist, ‘ES with no braces (ESNB)’ is used. For further details of the enlarged edition, refer to [1].

3.2 REVISED EDITION
The revised edition also includes two types of diagnoses, namely ‘standard’ and ‘detailed’ diagnoses. At present, detailed diagnosis is considered to be the most accurate method. In these diagnoses, the required strength (Qr) and retaining strength (Pd for the revised-standard diagnosis, Qd for the revised-detailed diagnosis) are evaluated. The ratio of the retaining strength (Pd, Qd) to the required strength (Qr) is also considered. In addition, there are two ways to obtain the required strength (Qr), namely ‘simplified’ and ‘detailed’ methods. In this paper, ‘RSS’ and ‘RSD’ refer to the simplified and detailed methods, respectively. Furthermore ‘RDS’ and ‘RDD’ indicate the simplified and detailed methods, respectively, of obtaining the required strength (Qr) for the revised-detailed diagnosis. The most promising method is RSS. For further details of the revised edition, refer to [2].

The seismic diagnosis methods and conditions used for the calculations are omitted in this paper as they have already been reported in [3].
4 EVALUATION DAMAGE LEVEL

4.1 EVALUATION METHOD

Damage investigation involved field surveys and questionnaires that were distributed to the residents of the houses considered. The investigation was performed by the University of Tokyo in the areas affected by the 1995 Hyogoken-nanbu earthquake, which had a seismic intensity of 7 on the Japanese scale used at that time. Field surveys involved exterior investigation using photographs, sketches, memos as well as damage level evaluation. After permission was obtained, the interior of the houses was also investigated.

We established six damage levels on a scale of 0 to 5 according to ‘Classification criteria and technical guidelines restoration of buildings at the earthquake disaster’ [4] published by JBDPA, and assessed the houses on the basis of damage investigation. Table 1 shows the typical examples of responses to the contents of the questionnaires for each damage level. Table 2 shows the basis and results of the damage level evaluation.

Table 1: Examples of responses to contents of questionnaires for each damage level

<table>
<thead>
<tr>
<th>No</th>
<th>Contents of questionnaire</th>
<th>Damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0: No damage</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1: Slight damage</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>2: Little damage</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>3: Moderate damage</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>4: Serious damage</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>5: Collapse</td>
<td>5</td>
</tr>
</tbody>
</table>

As shown in Table 2, 35 houses were built before 1981, and 76 were built from 1981 to 1995. For about 40% of the houses, the damage level exceeded level 3. Only one collapsed houses was classified as damage level 5.

Table 2: Results of damage level evaluation

<table>
<thead>
<tr>
<th>Damage level</th>
<th>Damage level evaluation</th>
<th>Constructed year Before 1981</th>
<th>1981 - 1995</th>
<th>Total number of houses</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 No damage</td>
<td>Fall of furnitures. Wrinkles of wall cloth, etc.</td>
<td>1</td>
<td>18</td>
<td>19</td>
</tr>
<tr>
<td>1 Slight damage</td>
<td>Cracks in mortar external walls. Cracks in bathroom tiles, etc.</td>
<td>5</td>
<td>17</td>
<td>22</td>
</tr>
<tr>
<td>2 Little damage</td>
<td>Partial evagination of mortar external walls. Evagination of bathroom tiles, etc.</td>
<td>11</td>
<td>18</td>
<td>29</td>
</tr>
<tr>
<td>3 Moderate damage</td>
<td>Evagination of interior walls, etc.</td>
<td>5</td>
<td>19</td>
<td>24</td>
</tr>
<tr>
<td>4 Serious damage</td>
<td>The house is considered to be rebuilt since interior and external walls are damaged severely</td>
<td>13</td>
<td>6</td>
<td>19</td>
</tr>
<tr>
<td>5 Collapse</td>
<td>The house has collapsed completely or it can not support vertical loads</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Total number of houses 35 76 111
Figure 1 shows the results, including the number of houses, of the damage levels evaluation according to region. The total number of houses situated in Tsuna and Kobe is 51, whereas 60 houses remain in Takarazuka and Nishinomiya. There is a greater tendency or the damage levels to be high in the region of Tsuna and Kobe, because they were nearer to the epicentre of the earthquake compared with the Takarazuka and Nishinomiya region. In addition, the house that collapsed is situated in the region of Tsuna and Kobe.

5 ANALYSIS

5.1 TRENDS OF THE EVALUATION AVERAGE

Figures 2–4 show the relationship between the evaluation averages of seismic diagnoses and earthquake damage levels of 111 houses. The evaluations by each diagnosis were obtained from both X and Y directions of the first floor. And the lower value of two directions was regarded as the represented evaluation of the house. As shown in Figure 2, for ESNB, the evaluation averages range from a high of 0.71 to a low of 0.46, and the obtained averages were significantly lower than those of the other diagnoses. The averages of the ESB evaluations were 1.20–0.80, whereas those of the ESB evaluations were about 0.5 higher than those of ESNB evaluations. That is, ESB and ESNB are within the same band and have the tendency to give similar evaluations at each damage level. On the other hand, the value of ED ranges from 0.81 to 1.59 and decreases as the damage level becomes high.

As shown in Figure 3, the evaluation average of RSS decreases to about 0.1 as the damage level becomes high. However, RSD is similar to ED, except in the case of damage level 5. The averages of RSS and RSD at damage level 5 are also large. In addition, the difference in values from damage levels 0 to 5 is small. Figure 4 shows that for the RDS and RDD evaluations, the averages at each damage level tend to be about 0.2 smaller than those of ED diagnosis. That is, there is a significant correlation between the evaluation average and damage levels of ED, RDS and RDD.

5.2 TRENDS OF THE EVALUATION STANDARD DEVIATION

As shown in Figure 2, with the exception of damage level 5, the standard deviation range of the ESNB evaluation, i.e. 0.08–0.17, is the narrowest, but the ESNB value tends to be 0.5–1.0. On the other hand, the range of ED is the widest at each damage level. Moreover although the houses have the same damage level, the obtained values vary with the different evaluations.

Figures 6 and 7 show the results of the revised diagnoses, all of which have standard deviation that range from about 0.2 to 0.3. However, it is difficult to specify the damage level from the evaluation because the standard deviation of the evaluation is large.

5.3 TRENDS OF THE AVERAGES AND STANDARD DEVIATION OF THE EVALUATIONS ACCORDING TO REGION

Figures 8–12 show the relationship between the averages and standard deviation of the seismic diagnoses evaluations and earthquake damage levels according to region.
The black line in the figures shows the standard deviation of the evaluations for the Tsuna and Kobe region, whereas the grey line shows that for the Takarazuka and Nishinomiya region. Figures 9–12 show the result obtained using the revised edition. With a few exceptions, the houses in the Tsuna and Kobe region tend to be evaluated more favourably with the revised-detailed diagnoses.

6 CONCLUSIONS

Several seismic diagnoses were applied to 111 existing conventional Japanese wooden houses that were damaged by the Hyogoken-nanbu earthquake. Then, the damage levels of these houses were evaluated. From our analysis, the following conclusions can be drawn:

1) As the estimation of earthquake resistant elements becomes more precise, the correlation between the evaluation and earthquake damage level increases. In other words, the reliability of the evaluation increases.

2) In particular, the revised-detailed diagnoses provide values that are significantly proportional to the damage level. However, it is difficult to specify the damage level solely from the evaluation. This is because the standard deviation of the evaluation is large.

3) For the revised-detailed diagnoses, the houses with damage level 4, which indicate serious damage, were evaluated to have a value of about 1.0. Although the 1995 Hyogoken-nanbu earthquake exceeded the expectations of the Building Standard Law of Japan, the revised diagnoses tended to give estimates that were more conservative than the actual seismic performance of the houses.

4) With a few exceptions, the houses in the Tsuna and Kobe region tend to be evaluated more favourably with the revised-detailed diagnoses.

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The authors thank the Sakamoto Laboratory of the University of Tokyo for providing data regarding the investigation of the damage caused by the 1995 Hyogoken-nanbu earthquake. The authors express their deep appreciation to Professor Sakamoto and the members of his laboratory at the University of Tokyo. The authors are also deeply indebted to Mr. Yanagisawa of INTEGRAL Co. for his kind cooperation.

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A Study on the Vibration Characteristics of Traditional Timber Frames including Bracket Complexes

Iuko Tsuwa¹, Mikio Koshihara²

ABSTRACT: When we build or renovate traditional timber structures such as temples and shrines in Japan, earthquake resistant designed buildings are needed. Therefore the quantitative evaluation of their structural behaviors is indispensable. In this paper, shaking table tests were performed with traditional timber frames including bracket complexes called Kumimono in Japanese. Additionally we conducted the earthquake response analysis. The analysis results were compared with experimental ones. The vibration characteristics of whole frames and each structural element were discussed.

KEYWORDS: Traditional timber structure, Bracket complexes, Shaking table test, Earthquake response analysis

1 INTRODUCTION

When we build or renovate traditional timber structures such as temples and shrines in Japan, earthquake resistant designed buildings are needed. Therefore the quantitative evaluation of their structural behaviors is indispensable. In the buildings such as temples and shrines, column, mud wall, Nuki which is a tie beam extending from one pillar to another, and bracket complexes called Kumimono in Japanese which is a structural component between a column and roof, etc, are considered as structural elements. Each element has been researched and modelled experimentally and analytically. However it is not clear how each element contributes to the whole structural behavior. In this study, the shaking table tests were performed with specimens which modelled the frame of the first story in fifth storied pagoda. Specimens are two types. One is consisted of columns, Nuki, bracket complexes. The other one added mud wall to another type. Additionally, we conducted earthquake response analysis based on the experimental results and existing theory model. The analysis results were compared with experimental ones. The aim of this research is to clarify the vibration characteristics of each element in a frame and the difference of the behaviors about bracket complexes in the two types of specimens.

2 OUTLINE OF EXPERIMENT

2.1 TEST SPECIMEN

The specimens were the 2/3 scale model of a frame in the Asuka style of the five-storied timber pagoda as shown in Figure 1. The span between columns was an outside plane of structure in the first story of the pagoda as shown in Figure 1, in order to clarify the modeling method of the structures consisting of Nuki, columns, Kumimono modelled the parts of 2 pairs on the central two columns because Kumimono in corners is projecting at 45-degree angle in a flat and has complex forms and the aim of this study was to get basic data of each element. A cornerstone was put under each column. They were connected with a dowel. Tree species were yellow cedar.

Three types of specimens were used in the tests as shown in Figure 2. Specimen 1 consisted of columns, Koshinuki, Jinuki, Kumimono. Specimen 2 was made by adding mud walls between the columns of the Specimen 1.

2.2 EXPERIMENTAL METHOD

The shaking table tests were carried out at the shaking table of Chiba Experiment Station in Institute of Industrial Science. Horizontal unidirectional shaking was conducted. Figure 3 shows the experimental method. The vertical load of 39.9kN made of steel frames and lead was put on each specimen. A load cell to measure axial force and shear force was placed under each column. The acceleration, displacement and the strain of specimens were measured using about 75 devices.

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As a large input motion BCJ-L2, the level 2 of a simulated wave provided from the building center of Japan, was used. In the tests of Specimen 1, the maximum input acceleration was increased by 10% from 10% to 100%. In the tests of Specimen 2, one input with BCJ-L2 20% and two inputs with BCJ-L2 100% were carried out.

3 EXPERIMENTAL RESULTS

3.1 THE RELATIONSHIP BETWEEN LOAD AND DISPLACEMENT

3.1.1 Specimen 1

Figure 4 shows the relationship between load and displacement from the inputs of BCJ-L2 20%, 40%, 80%, 100% to Specimen 1. Until the test of 20%, the relationship between load and displacement was linear. After the test of 30%, the relationship became nonlinear. The stiffness went down at the displacement of about 5mm. After the yield point, the hysteresis curve became swollen. The negative gradient appeared in the deformation from the maximum to the origin. The maximum shear force was 10.75kN at the test of BCJ-L2 80%. The maximum displacement was 180.2mm, 1/13rad deformation angle of a column, at the test of BCJ-L2 100%.

3.1.2 Specimen 2

Figure 5 shows the relationship between load and displacement for the inputs of BCJ-L2 20% and BCJ-L2 100% twice to Specimen 2. In the test of BCJ-L2 20%, the stiffness was higher than that in the test of 20% for Specimen 1. It was assumed to be due to the resistance of the mud wall. In the first BCJ-100%, the load went up to about 12kN within 40mm displacement. It can be assumed to be due to the resistance force of mud walls. After the collapse of mud walls, the resistance force of the specimen dropped and a hysteresis curve changed to the curve like Specimen 1. In the test of the second BCJ-100%, the load did not rise like the first one. At around 5kN load, the stiffness changed and a load increased gradually up to around 10kN.
3.1.3 The comparison between Specimen 1 and 2

Figure 6 shows the relationship between load and displacement for BCJ-L2 100% of Specimen 1 and Specimen 2. The characteristics of every hysteresis curve were almost same after the collapse of the mud wall.

3.1.4 The relationship between load and displacement of Kumimono

Figure 5 shows the relationship between load and displacement of Kumimono for all specimens from the tests of BCJ L2 100%. The displacement of Kumimono was calculated by subtracting that of Daiwa and the horizontal displacement occurred by the rotation of Daiwa from that of Toshi-hijiki. The horizontal load was the sum of shear forces from the load cell under columns. The displacement of Kumimono in Specimen 1 was larger than that of Specimen 2. It can be assumed that the wall stiffness has an influence on the behaviour of Kumimono.

3.2 Characteristics of deformation

Figure 7 shows the horizontal deformation of specimens in the range of the maximum amplitude from the test of BCJ L2 100%. Most of the displacement of Specimen 1 was the displacement from the inclination of columns. The line between Daiwa and Toshi-hijiki was vertical almost all time. It means that the displacement of Kumimono was very minute. As for Specimen 2, the characteristic was almost same. However, at the maximum deformation, the line between Daiwa and Top inclined. It can be seen that the deformation of Kumimono increased slightly.

3.3 THE BENDING MOMENT AT THE JOINT OF COLUMN-KOSHINUKI

When columns inclined, the rotation motion occurred at the joints of columns and Koshinuki. By the motion, the compressive strains inclined to the grain of members came up and bending moment happened at Koshinuki. Figure 9 shows the relationship between the bending moment and rotation angle of a joint of columns and Koshinuki for Specimen 1 and Specimen 2. The data of each specimen is the sum of all tests because compressive strains inclined to the grain of members did not get back if deformation by crushing stress occurred once. The positive rotation angle of Specimen 2 was not same as the negative one. It can be assumed to be due to the different gap of a column and Koshinuki in right and left. The stiffness of Specimen 2 became lower than that of Specimen 1. The maximum bending moment of specimen 1 was almost same with specimen 2. It was about 2300 kN*mm at 0.1 rad. It can be seen to be due to the reduction of resistance force happened by iteration shaking.
4 EARTHQUAKE RESPONSE ANALYSIS

4.1 MODELLING

Whole frames including columns, the joint of column and Jinuki, the joint of column and Koshinuki, bracket complexes, and Mud wall were modelled as shown in Figure 10. The characteristics restoring force of each element was calculated by the measurement values from experiments, material experiment results and previous theory. Each characteristic was defined as follows.

About column, they have restoring force occurred by column rocking. The force was defined using Equation (1) [1, 2] as follows:

\[
H = H_0 \left[ 1 - \frac{\delta}{b} + 0.99625e^{-0.795\delta^2} + \frac{1.9963}{(25\frac{\delta}{b} + 1)} \right]
\]

where \(H\) = restoring force, \(H_0\) = \(Pb/h\) = restoring force for rigid body, \(P\) = vertical load, \(b\) = the diameter of column, \(h\) = the height of column, and \(\delta\) = horizontal displacement.

The maximum restoring force can be estimated 80% of \(H_0\). The displacement occurred at that time is 0.088 times as much as the diameter of column.

As for the joint of the column-Koshinuki, we estimated bending moment at the joints from the experimental results.

About the joint of the column-Jinuki, the value of the compressive strains at the joints was not measured in experiment. Therefore we estimated the compressive strains inclined to the grain of members using Merikomi theory in [3].

As for mud walls, material tests for wall pieces made by the same specification were carried out. The restoring force was evaluated from the tests in [4,5].

The envelope curve for each characteristic of restoring force and the curve of the sum of elements are shown in Figure 11.

4.2 EARTHQUAKE RESPONSE ANALYSIS

We conducted the earthquake response analysis using the model of restoring force in Figure11. Figure 12 shows the comparison of the experimental and theoretical results about the structural part under Kumimono. The theory value almost corresponded to the experimental results in both Specimen 1 and 2. It can be seen that it is possible to sum the restoring force of each element in the structure under Kumimono in Specimen 1 and 2.
5 CONCLUSIONS

The shaking table tests were carried out using three kinds of plane of traditional timber structure with two types of wall having different rigid. Each result was compared. The evaluation of the horizontal resistance force of each structural element under Kumimono was discussed. The results could be summarized as follows.

1) Adding mud walls in a frame caused a large horizontal resistance force and high stiffness compared with a frame without walls.

2) The characteristics of the relationships after collapse of mud wall became almost same. The hysteresis curve of Specimen 2 after collapse followed the almost same route as that of Specimen 1. In the both relationships, the load gradually increased until the maximum displacement. The form of the relationships had swell after about 1/20 rad deformation angle of a column. The deformation of columns was large in that of the whole structure in both specimens.

3) The deformation of Kumimono in Specimen 2 became about twice as much as that of Specimen 1. However the deformation was minute comparing with whole structure one.

4) The maximum bending moment and maximum deformation at Koshihnuki in Specimen 2 were almost same as Specimen 1. The reduction of the resistance was seen in the relationship between bending moment and rotation angle of Koshinuki in Specimen 2 compared with Specimen 1. It can be assumed to be due to the iteration shaking.

5) It is clarified that the relationship between load and displacement of a frame like Specimen 1 and 2 can be evaluated by adding the restoring force characteristics of each structural element in the frame.

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EFFECT OF LINTEL ON SEISMIC PERFORMANCE IN POST-BEAM STRUCTURE

Hyung-Kun Kim¹, Chun-Young Park², Sung-Jun Pang³, Kwon-Hwan Hwang⁴, Jun-Jae Lee⁵

ABSTRACT: This study experimentally evaluated the seismic performance of post-beam timber frame structures with energy dissipation mechanism. To evaluate the effect on seismic performance of lintel which has been widely used as wall frame in Korean traditional post-beam structure, total two frames were prepared. One had no lintel and the other one had lintel at the height of 800 ㎜, respectively. Cyclic loading tests were conducted for each frame according to the standard loading protocol. Frame which had lintel showed slightly higher stiffness. And it showed noticeably significant energy dissipation performance after yielding of the joint. And that leads to the conclusion that lintel has structural effect and it should be considered as an important factor when evaluating seismic performance of the structure after yielding of the joint.

KEYWORDS: Post-Beam Structure, Seismic Performance, Hysteresis Curve, Energy Dissipation, Dovetail Joint

1 INTRODUCTION

Timber structures have been showed significant seismic performance in historical earthquake. Extensive studies have been conducted on seismic performance of light framed houses throughout North America and Europe and post-beam structures in East Asian region. Han-ok which means Korean traditional building is categorized into post-beam structures. As well as its beautiful appearance, its structural performance has been an issue. However, it was not easy to calculate structural stability because Han-ok has been built by own hands of craftsman not by machine. In this reason most of Han-ok has been over designed and caused wasting of expensive wood members.

Among all of the members lintel has been overlooked for its structural performance. Lintel is a horizontal member which is used as wall frame and it has been categorized as non-structural member without any experimental background. Therefore this study is aimed to investigate the effect of lintel on structural performance in post-beam structure and furthermore to investigate seismic performance using cyclic loading test.

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Figure 1: Middle and Bottom Lintel in Han-ok
2 Materials and Methods

2.1 Specimens

Glu-lam using Japanese Larch (Larix leptolepis) was prepared for full-scale test. Size of specimens were 180 X 180 X 2400 (㎜) for the post, 180 X 240 X 3420 (㎜) for the beam and 180 X 180 X 3420 (㎜) for the lintel. Dovetail joint was chosen to imitate Korean traditional joint and cut by Pre-Cut machine (Hundegger). Detail view of the joint is presented in Figure 2.

2.2 Cyclic Loading Test

Structure 1 was constructed with two posts and a beam, Structure 2 had same composition as Structure 1 with lintel in the height of 800 ㎜ from the bottom. Detail of the test specimen is presented in Figure 4. Cyclic loading test had been conducted for those two frames. Cyclic load was applied on the left upper corner of the frame according to the loading procedure shown in Figure 3. Loading speed and maximum displacement were 1 ㎜/sec and ±100 ㎜, respectively. Using 6 LVDTs and computer data acquisition system, displacements of each part were obtained. Supporting condition was assumed to be hinge support. Detail of loading procedure for each cycle is shown in Table 1.

2.3 Horizontal Load

Han-ok has beautiful yet significantly heavy roof. This weight of the roof is delivered to rafter and then distributed to beams and upper parts of the posts uniformly. To calculate roof load delivered to unit frame, standard Han-ok was selected. Unit roof load was calculated according to KBC (Korean Building Code 2009) and the value was 3,659.09 kgf. And simulated horizontal load was applied on the frame using 18 hydraulic cylinders.

Table 1 Detail of loading procedure

<table>
<thead>
<tr>
<th>Drift Angle (rad)</th>
<th>Frequency (Hz)</th>
<th>Cycle Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0025</td>
<td>0.043668</td>
<td>3</td>
</tr>
<tr>
<td>0.005</td>
<td>0.021834</td>
<td>3</td>
</tr>
<tr>
<td>0.010</td>
<td>0.010917</td>
<td>3</td>
</tr>
<tr>
<td>0.015</td>
<td>0.007278</td>
<td>3</td>
</tr>
<tr>
<td>0.020</td>
<td>0.005459</td>
<td>3</td>
</tr>
<tr>
<td>0.025</td>
<td>0.004367</td>
<td>3</td>
</tr>
<tr>
<td>0.030</td>
<td>0.003639</td>
<td>3</td>
</tr>
<tr>
<td>0.040</td>
<td>0.002729</td>
<td>3</td>
</tr>
<tr>
<td>Failure</td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 3: Loading Procedure

Figure 4: Test Setup of Frames
2.4 Evaluation Methods
Strength properties were calculated using a method proposed by the Japan Housing and Wood Technology Center. Details of the methods are shown in Figure 5.

Figure 5: Method details for calculating strength properties

Energy dissipation can be evaluated by calculating the inner area of the hysteresis loop. In general, energy dissipation should be calculated at the point that the story-drift ratio reaches 3.5%. And this corresponds to the point between the 7th and 8th cycle in this study. However, experiment of structure 2 which had lintel was not completed to reach this cycle for safety reason, so energy dissipation of the 6th cycle was compared. Third cycle of each hysteresis curve was selected and simplified to calculate the inner area. Then displacement was separated into differential length to apply mensuration of division as shown in Figure 6.

Figure 6: Calculation of Energy Dissipation

3 Result and Discussion
3.1 Hysteresis Curve
Figure 7 shows hysteresis curves of two structures. The two structures show similar shape of hysteresis curves. They yield after 3rd cycle and lose most of their stiffness. However, unlikely to the structure 1, strength of the structure 2 increases slightly after yielding. And also, structure 2 doesn’t show dramatic decrease in strength after yielding. This result means that lintel has structural effect after yielding.

Figure 7: Hysteresis Curves of Structures

3.2 Strength Properties
Structure 2 which have lintel showed larger value of K. Lintel make structure stiffer and that leads to structure yield in less drift angle. In case of \( \mu \) which can be calculated by dividing \( \theta_u \) with \( \theta_v \), showed larger value. It is considered that lintel gives more ductility to the structure after yielding and lintel should be considered as a critical member when conducting seismic design.

Table 2 Strength Properties

<table>
<thead>
<tr>
<th></th>
<th>Fy</th>
<th>( \theta_y )</th>
<th>K</th>
<th>( f_{\text{max}} )</th>
<th>( \theta_{\text{max}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure 1</td>
<td>109.60</td>
<td>0.004</td>
<td>3659.77</td>
<td>109.65</td>
<td>0.02</td>
</tr>
<tr>
<td>Structure 2</td>
<td>122.71</td>
<td>0.003</td>
<td>4119.69</td>
<td>107.28</td>
<td>0.01</td>
</tr>
</tbody>
</table>

(Fy: Yield Force, \( \theta_y \): Yield Drift Angle, K: Initial Stiffness, \( \mu \): Ductility Factor)
3.3 Energy Dissipation

Energy dissipation performance of each frame was evaluated by calculating the inner area of the hysteresis curve. Structure 1 which was composed of two posts and beam showed 434.1(kgf-rad) of energy dissipation throughout 6 cycle of loading. On the other hand, Structure 2 which had lintel showed 565.6(kgf-rad) throughout same cycle. And the difference of value was significant after third cycle when the joint started to yield. Comparison of the energy dissipation performance is shown in Figure 9. As similar to the $\mu$, ductility factor, energy dissipation performance is one of the critical factor when conducting seismic design. Lintel proved that it could absorb or transform dynamic energy into other form of energy when it was subjected to cyclic loading.

Table 3 Energy Dissipation for Each Cycle and accumulation

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Structure 1</th>
<th>Structure 2</th>
<th>Structure 4-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.7</td>
<td>2.7</td>
<td>3.6</td>
</tr>
<tr>
<td>2</td>
<td>12.5</td>
<td>15.3</td>
<td>16.3</td>
</tr>
<tr>
<td>3</td>
<td>44.4</td>
<td>59.6</td>
<td>41.9</td>
</tr>
<tr>
<td>4</td>
<td>91.4</td>
<td>131.0</td>
<td>72.4</td>
</tr>
<tr>
<td>5</td>
<td>161.1</td>
<td>312.1</td>
<td>105.2</td>
</tr>
<tr>
<td>6</td>
<td>253.4</td>
<td>565.6</td>
<td>138.9</td>
</tr>
<tr>
<td>7</td>
<td>177.4</td>
<td>555.1</td>
<td>239.9</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>735.0</td>
</tr>
<tr>
<td>Total</td>
<td>565.6</td>
<td>705.0</td>
<td></td>
</tr>
</tbody>
</table>

Figure 9: Energy Dissipation Performance

4 Conclusions

Following conclusions can be drawn from the present study:

- Lintel shows significant energy dissipating performance after yielding
- Applying lintel to structure can be an effective way to enhance seismic performance of structures
- Bearing occurred between post and lintel throughout the whole behaviour is the reason for increasing of energy dissipation

Further study on the bearing in post-lintel joint is required.

REFERENCES


EVALUATING IN-PLANE SHEAR MODULUS OF WOOD-BASED STRUCTURAL PANELS BY RACKING TEST OF SMALL NAILED WALL

Kenji Aoki¹, Tatsuya Shibusawa²

ABSTRACT: Racking test of small wall specimens sheathed with various wood-based structural panels was conducted and the in-plane shear modulus of the panels was evaluated by several measurement methods. The validity of the obtained shear modulus and the efficiency of the measurement methods were discussed. As a result, in the area of lower lateral forces, the shear modulus calculated by diagonal displacement did not show a steady value. While the shear modulus calculated by absolute coordinate was comparatively well corresponding to the values calculated by diagonal strain. The measurement method by the absolute coordinate can be used as an effective method for calculating the shear modulus of wood-based structural panels.

KEYWORDS: In-plane shear modulus, Wood-based structural panel, Racking test, Nailed wall

1 INTRODUCTION

For the timber house construction in Japan, a higher structural safety probability came to be requested by promotion of long-life quality housing and the spread of the housing quality assurance, etc. Though there is already a lot of research on the shear wall that is the main earthquake-proof factor of the timber house construction, shear resistance of nailed joint and shear modulus of wood-based structural panels are required in order to calculating the allowable shear strength of panel sheathed shear wall.

There are several test methods for measurement of the shear modulus, e.g. two-rail shear test, panel shear test, and so on [1]. However, the obtained values are mutually different, and there is not characteristic value of various wood-based structural panels that is appropriate for the seismic design.

Therefore, in this research, racking test of small nailed wall specimen proposed by the manual of the allowable stress design for the Japanese conventional house construction [2] was conducted and the in-plane shear modulus of sheathed panels was evaluated with four measurement methods. Then, the validity of the obtained shear modulus and the efficiency of the test method were discussed.

2 EXPERIMENTS

2.1 SPECIMENS

Specimen was wood-framed shear wall of 0.91m length and 1.82m height sheathed with a wood-based structural panel on one side of the frame as shown in Fig. 1 and 2. Members of the frame, such as beam, sill and columns, were glued laminated timber (JAS standard for glued laminated timber, E95-F315) made of Lodgepole Pine. And the cross-sectional size of all members was 105x105mm. Sheathing panel was connected to frames with CN50 nails (JIS standard, A 5508) spacing 100mm in the perimeters of the panel. Hold-down connectors of HD-B10 were applied to both ends of columns.

The panel type, species or sources, nominal thickness, and the average density of tested panels are shown in Table 1. Seven kinds of plywood, three kinds of oriented strand boards, and two kinds of fiberboard (medium density fiberboard and particleboard) were used for the experiment.

Three specimens were prepared for each panel type.

2.2 TEST METHOD

The schematic diagram of racking test and measurement method was shown in Fig. 2. The reversed cyclic loads were applied at the end of the beam by a computer-controlled actuator with a displacement rate of 1mm/sec. Cyclic test procedure of shear wall was according to the building minister certification method on Performance
Evaluation Organization in Japan. A deformation angle is provided as the horizontal displacement of beam divided by the height of test specimen. And the three cyclic loading protocols were at each deformation angle of 1/450, 1/300, 1/200, 1/150, 1/100, 1/75, 1/50, and the last monotonic loading was 1/15 rad, as shown in Fig. 3. Horizontal displacements of beam and sill, and vertical displacements of two columns were measured by electric transducers. The measurement methods for calculating the shear modulus of panels are as follows: (1) diagonal displacements measured by electric transducers with tensioned wire ropes between freely rotating targets of four corners, (2) absolute coordinates in the horizontal and perpendicular direction of the four targets measured by eight laser displacement transducers, (3) strain measured by strain gauges near the center of the panel, (4) diagonal strain measured by strain gauges near the center of the panel. Fig. 4 shows the installation of the freely rotating target and Fig. 5 shows the installation of electric transducer for diagonal displacement.

![Small wall specimen ready for racking test.](image1)

**Figure 1:** Small wall specimen ready for racking test.

![Cyclic loading protocol.](image2)

**Figure 2:** Schematic diagram of racking test.

![Cyclic loading protocol.](image3)

**Figure 3:** Cyclic loading protocol.

**Table 1: List of wood-based structural panels**

<table>
<thead>
<tr>
<th>Type</th>
<th>Board</th>
<th>Standard, (Spesis or Sources)</th>
<th>Nominal thickness (mm)</th>
<th>Average density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PW-S</td>
<td>Plywood</td>
<td>JAS standard for plywood, grade 2, 5ply, (Japanese cedar)</td>
<td>12.0</td>
<td>427</td>
</tr>
<tr>
<td>PW-SK</td>
<td></td>
<td>JAS standard for plywood, grade 2, 5ply, (Japanese larch and Japanese cedar)*1</td>
<td>12.0</td>
<td>522</td>
</tr>
<tr>
<td>PW-KS</td>
<td></td>
<td>JAS standard for plywood, grade 2, 5ply, (Japanese larch and Japanese cedar)*1</td>
<td>12.0</td>
<td>549</td>
</tr>
<tr>
<td>PW-K</td>
<td>Plywood</td>
<td>JAS standard for plywood, grade 2, 5ply, (Japanese larch)</td>
<td>12.0</td>
<td>648</td>
</tr>
<tr>
<td>PW-SL</td>
<td></td>
<td>JAS standard for plywood, grade 2, 5ply, (Russian larch and Japanese cedar)*2</td>
<td>12.0</td>
<td>545</td>
</tr>
<tr>
<td>PW-LS</td>
<td></td>
<td>JAS standard for plywood, grade 2, 5ply, (Russian larch and Japanese cedar)*2</td>
<td>12.0</td>
<td>578</td>
</tr>
<tr>
<td>PW-L</td>
<td></td>
<td>JAS standard for plywood, grade 2, 5ply, (Russian larch)</td>
<td>12.0</td>
<td>660</td>
</tr>
<tr>
<td>OSB-N</td>
<td>Oriented Strand Board</td>
<td>JAS standard for structural panel, grade 3, (production in North America)</td>
<td>11.1</td>
<td>690</td>
</tr>
<tr>
<td>OSB-U</td>
<td></td>
<td>JAS standard for structural panel, grade 3, (production in Europe)</td>
<td>11.1</td>
<td>704</td>
</tr>
<tr>
<td>OSB-Q</td>
<td></td>
<td>JAS standard for structural panel, grade 4, (production in Europe)</td>
<td>9.5</td>
<td>682</td>
</tr>
<tr>
<td>MDF</td>
<td>Medium Density Fiberboard</td>
<td>JIS standard, A 5905, type 25M</td>
<td>12.0</td>
<td>767</td>
</tr>
<tr>
<td>PB</td>
<td>Particleboard</td>
<td>JIS standard, A 5908, type 18M</td>
<td>12.0</td>
<td>770</td>
</tr>
</tbody>
</table>

*1 and *2: veneer constructions are mutually different.

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3 RESULTS AND DISCUSSION

3.1 LOAD-DISPLACEMENT CURVES

Fig. 6 shows the load-displacement envelope curves of all specimens. In the types of plywood sheathed wall specimens, a high load-carrying capacity was obtained by using higher density plywood. However, the wall specimens sheathed with lower density plywood indicated relatively high deformation capacity. In the types of OSB sheathed wall specimens, relatively higher stiffness was obtained than that of plywood sheathed wall specimens. However, the load carrying capacity was not so high. In the types of fiberboard, load carrying capacity was almost the same that used the higher density plywood. In addition, the stiffness of the fiberboards was also high as well as OSB.

3.2 CALCULATION OF THE IN-PLANE SHEAR MODULUS

By using the envelope curves shown in Fig. 6, the in-plane shear modulus (=G) for each panels was calculated by three methods as follows,

(A) The shear modulus was calculated by using diagonal displacements measured by the electric transducer with tensioned wire ropes as equation (1) below,

\[ G = \frac{M}{l \cdot h \cdot \gamma} \]  \hspace{1cm} (1)

where \( G \) = in-plane shear modulus (N/mm²),
\( \gamma = 2(\delta_1 - \delta_2) / L \),
\( \delta_1, \delta_2 \): diagonal displacements (mm),
\( L \): distance between initial targets (mm),
\( M \): moment (= PH)
\( P \): load (N),
\( H \): wall height (mm),
\( l \): panel length (mm),
\( h \): panel height (mm),
\( t \): panel thickness (mm).

(B) At first, the diagonal length was calculated from the absolute coordinate of the target of four corners. Next, the relative deformation from the initial length was calculated, and last, the shear modulus was calculated by using equation (1).

(C) The shear modulus was calculated from shear strain obtained by strain gauges as equation (2) below,

\[ G = \frac{P}{l \cdot t \cdot \varepsilon} \] \hspace{1cm} (2)

where \( \varepsilon = |\varepsilon_1 - \varepsilon_2| \),
\( \varepsilon_1, \varepsilon_2 \): shear strain of each gauge,
\( P \): load (N),
\( l \): panel length (mm),
\( t \): panel thickness (mm).

3.3 EFFECT OF MEASUREMENT METHOD

To obtain a steady numerical value, the range of the examination of shear modulus was assumed to be 0.1-
0.9\(P_{\text{max}}\). Fig. 7 shows examples of the relationship between the calculated shear modulus and the lateral load of the wall specimens. In the area of lower lateral load, the shear modulus calculated by diagonal displacement showed an inconstant value. And the shear modulus calculated by absolute coordinate was comparatively well corresponding to the values calculated by diagonal strain in the types of plywood sheathed wall specimens. On the other hand, in the types of wood-based structural panels, like OSB, PB, and MDF, since a steady numerical value was not obtained, it is necessary to improve the procedure for measurement and to verify it again. The shear modulus calculated by strain gauges showed a roughly constant value.

3.4 COMPARISON OF PANEL DENSITY AND IN-PLANE SHEAR MODULUS

Table 2 shows result of average in-plane shear modulus and standard deviation for three specimens of each panel type. The averaged shear modulus of three specimens was obtained in the 10-40% area of the maximum load of each specimen. The shear modulus calculated by rectangular strain was close to the previous research results [3] measured by two-rail shear test according to ASTM standard.

Table 2: Result of average in-plane shear modulus

<table>
<thead>
<tr>
<th>Panel type</th>
<th>Average density (kg/m(^3))</th>
<th>Shear modulus G (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1) Diagonal displacement</td>
<td>(2) Absolute coordinate</td>
</tr>
<tr>
<td>PW-S</td>
<td>427</td>
<td>585</td>
</tr>
<tr>
<td>PW-SK</td>
<td>522</td>
<td>1025</td>
</tr>
<tr>
<td>PW-KS</td>
<td>549</td>
<td>863</td>
</tr>
<tr>
<td>PW-K</td>
<td>648</td>
<td>664</td>
</tr>
<tr>
<td>PW-SL</td>
<td>545</td>
<td>789</td>
</tr>
<tr>
<td>PW-LS</td>
<td>578</td>
<td>580</td>
</tr>
<tr>
<td>PW-L</td>
<td>660</td>
<td>658</td>
</tr>
<tr>
<td>OSB-N</td>
<td>690</td>
<td>1536</td>
</tr>
<tr>
<td>OSB-U</td>
<td>704</td>
<td>1069</td>
</tr>
<tr>
<td>OSB-Q</td>
<td>682</td>
<td>1008</td>
</tr>
<tr>
<td>MDF</td>
<td>767</td>
<td>851</td>
</tr>
<tr>
<td>PB</td>
<td>770</td>
<td>25099</td>
</tr>
</tbody>
</table>

Notes: italic values are standard deviation. Not measured, \(^{*1}\)average value from two specimens, \(^{*2}\)value from a specimen.

Figure 7: Transition of shear modulus to lateral load.
Fig. 8 shows the comparison of density of structural panels and the calculated shear modulus. Along with the panel density increase, the shear modulus also rises gradually. However, the shear modulus was considerably different by the measurement method. The shear modulus calculated by absolute coordinate indicated roughly high values compared with the values calculated by strain gauges.

Fig. 9 shows the comparison of density of plywood and the calculated shear modulus. Three lines shown in figure are the collinear approximations to each three kinds of calculated values, such as absolute coordinate, rectangular strain, and diagonal strain. It is understood that the density of plywood and the in-plane shear modulus have a significant correlation.

4 CONCLUSIONS

Racking test of small wall specimens sheathed with various wood-based structural panels was conducted and the in-plane shear modulus of the panels was evaluated by several measurement methods. As a result, while the shear modulus calculated by diagonal displacement did not show a steady value, the shear modulus calculated by absolute coordinate was comparatively well corresponding to the values calculated by rectangular strain. While, in the types of OSB, PB, and MDF, a steady numerical value was not obtained, so it is necessary to improve the procedure for measurement and to verify it again. The measurement method by the absolute coordinate can be used as an effective method for calculating the in-plane shear modulus of wood-based structural panels.

REFERENCES


RELIABILITY OF GLULAM BEAMS EXPOSED TO FIRE

Mislav Stepinac1, Vlatka Rajčić2, Boris Androić3, Dean Čizmar4

ABSTRACT: This paper presents reliability analysis of a glulam load-bearing beam in a fire. Global fire safety concept of timber structure analysis is presented according to the recommendations from Eurocodes. Special attention was given to natural fire design with two different methods of parametric exposure which are given in Eurocode 1995-1-2. Differences in load-bearing capacity from Eurocode 1995-1-1 and 1995-1-2 are. The limit-state functions for maximum bending stress of glulam beam in fire conditions are formed. The charring rate of wooden beam is calculated by a model that includes explicitly the principal sources of uncertainties and variability. Reliability indexes are obtained from the limit state of the beam exposed to 30 min fire for the beams which spans were varied from 4 to 12 meters. Conclusions are given about structures with reliability class RC3 for which design according only to Eurocode 1995-1-2 is not sufficient.

KEYWORDS: glulam beam, reliability, fire, Eurocode, bending, the limit state function

1 INTRODUCTION

Until recently, the fire resistance of buildings was based on the ISO standard curve. The only option for determining the fire resistance was to carry out tests in laboratories. ISO Standard curve used by the current norm is too simple, unrealistic and lead to uneconomic situations with no guarantee of security proportional to the invested money.

A new approach to solve the problem of fire, called "Global Fire Safety Concept" consider the following steps: [11]

- takes into account characteristics of buildings important to the emergence and spread of fire are: fire scenario, fire load, charring rate, type of barriers, ventilation conditions,
- quantifies the risk of fire starting activities, the size of considered fire compartments and their occupancy and the impact of active fire fighting measures are taken into account; this risk analysis is based on existing statistical data of real fires and probabilistic methods,
- determines the design value of the parameter (the fire load) from the previous steps,
- determines the design heating curve of the design fire load that takes implicitly into account the fire risk and fire fighting measures,
- simulates the global behaviour of structures subject to the design heating curve in combination with a static load in case of fire,
- determines the fire resistance time \( t_{\text{fi,d}} \); it can often be infinite when the structure is able to withstand a static load from beginning to the end of the fire, verifies the safety of the structure by comparing the calculation fire resistance time \( t_{\text{fi,d}} \) with required time \( t_{\text{fi,req}} \) depending on the time of the evacuation and the consequences of the failure, in most cases the time required \( t_{\text{fi,req}} \) is prescribed in regulations made by the national authority.

On the example of mechanical response of structures in fire can best be seen the difference between the old and new approaches to protect structures against fire. In fact, fire is an accidental action on the structure, so it cause the structural responses. The sequence of events on the construction in the occurrence of fire is shown in Figure 1. The resistance of structural elements in fire was determined in a fire chamber by examining the elements exposed to standard fire, which is presented to the ISO
834 fire curve. New engineering approach is trying to solve the problem of fire protection with numerical procedure.

**Figure 1: The sequence of events on the construction in the occurrence of fire**

The first step in analysing the structure in case of fire is to determine the thermal effects. Thermal effects can be determined using one of the nominal curve of fire or carrying out the computational analysis of the structure. As the first method does not have an “engineering” character and the engineer is reduced to a passive role, numerous methods are being developed and the most effective ways of computational analysis are trying to be made. Selection and identification of fire scenarios is a start of the concept of reliability for structures with natural fire. It can be said that the risk assessment and scenario of fire is a critical aspect of the evidence procedure of reliability of structures exposed to fire effect.

**2 THEORETICAL BACKGROUND AND APPROACH TO THE DESIGN OF FIRE**

Due to the determination of design fire load density, \( qf,d \), there are two different approaches.

**2.1 LEVEL 1 - PROBABILISTIC APPROACH [10]**

\[
q_{f,d} = \gamma_{q,f} \cdot q_{f,k}
\]

where \( q_{f,k} \) = characteristic value of fire load density related to floor area \( A_f \), \( \gamma_{q,f} \) = global factor that breaks down the partial factors.

**2.2 LEVEL 2 - SEMIPROBABILISTIC (STANDARDIZED) APPROACH [7]**

\[
q_{f,d} = q_{f,k} \cdot m \cdot \delta q_1 \cdot \delta q_2 \cdot \delta n
\]

where the global factor is replaced with more partial factors; \( q_{f,k} \) = characteristic value of fire load density related to floor area \( A_f \), \( m \) = combustion factor (usually taken for cellulosic materials \( m=0.8 \)), \( \delta q_f \) = factor that takes into account the risk of fire due to the size of the section, \( \delta q_2 \) = factor that takes into account the risk of fire due to the purpose of the room, \( \delta n = \prod_{i=1}^{10} \delta_{ni} \) factor that takes into account a variety of active fire protection measures (if these measures cannot be predicted, it is taken \( \delta_n = 1.5 \)).

Level 2 of determination of fire load calculation is suitable for the practice and provides a solution that is on the side of higher reliability. Access to level 2 of determination of fire load calculation was adopted in EN 1991-1-2 (Annex E).

**3 FIRE FROM THE ASPECT OF THE RELIABILITY ASSESSMENT**

Reliability is defined as the ability of a structure or a structural element to fulfill certain requirements, including the estimated lifespan; it is expressed through probabilistic expressions, and includes safety, serviceability and durability. The fire load can be considered as action. However, this action is accidental, so the target value probability, \( p_f \), must be less than or equal to the product (of multiplication) of the targeted failure value in case of fire, \( p_{f,i} \), and the probability of occurrence of fire, \( p_{f,i} \):

\[
P_{f,i} \cdot P_g \leq p_f
\]

Equation (3) can be written as follows:

\[
P_{f,i} \leq \frac{p_f}{P_g}
\]

In that case, reliability index \( \beta \) is no longer the fixed value 3.8, but depends on different factors of probability of fire and is indicated with \( \beta_{fi} = f(p_{f,i}) \).

Knowing the probability of occurrence of fire that engulfs the section, and depending on the size of the section, current level of engagement and active measures of protection, the reliability index \( \beta \) can be determined through the following equation [10]:

\[
\beta_{fi} = \Phi^{-1} \left( \frac{p_f}{P_g} \right) = \Phi^{-1} \left( \frac{7.25 \times 10^{-4}}{P_g} \right)
\]

The probability of occurrence of fire \( p_{f,i} \) is determined with the following equation [10]:

\[
p_{f,i} = p_1 \times p_2 \times p_3 \times p_4 \times A_{f,i}
\]

in which:

- \( p_1 \) = probability of occurrence of a fully developed fire, including the effects of the interventions of users and fire brigade (for 1 m² of the layout of the construction and for one year),
- \( p_2 \) = reduction factor dependant on the type of the fire department and the passage of time from the onset of the alarm to the arrival of the firefighters,
- \( p_3 \) = reduction factor in case of the automatic fire notification (through smoke or heat detection),
- \( p_4 \) = probability of the sprinkler system failure (if they exist),
- \( A_{f,i} \) = area of the fire sector.

Assuming the fire is the main action the weighting factor for the fire load equals \( w_{f} = 0.7 \). Weighting factor for the main action under room temperature is greatly reduced, and in case of fire is considered secondary action.
Partial factor $\gamma_{qf}$ can be determined from Figure 2., from which the calculated fire load $q_{f,d}$ derives:

$$q_{f,d} = q_{f,d} \times \gamma_{qf}$$

(7)

The real fire curve determined with this method is a realistic and probabilistic fire curve that defines the thermal activity for the purpose of design the construction with satisfactory reliability.

4 FIRE IN TIMBER STRUCTURES

Unlike the fire design of the steel, concrete or composite structures, methods of design of fire in timber structures have been greatly simplified. Generally, it is not necessary to check the reduction of strength in the residual section, because each increase of temperature is considered small and is ignored. Method of design according to EN 1995-1-2 is a two stage process. First, the calculation of charring depth, and then determine the strength of the residual section. There are two methods of design, one under the standard fire, and the other to a real fire, or parametric.

4.1 STANDARD FIRE

EN 1995-1-2 [8] gives two values of charring rates $\beta_0$, for exposure to only one side and $\beta_n$ for multi-face exposure. In this paper, we observe the second case for which applies (8):

$$d_{\text{char,n}} = \beta_n \times t$$

(8)

where $d_{\text{char,n}}$ = depth of charring (notional), $\beta_n$ = design charring rate under standard fire exposure and $t$ = time of fire exposure.

4.2 REAL FIRE

Unlike the standard fire where the charring rate is constant, in real fires, characteristics of the room in which is an element (surface openings, floors, walls, partitions), fire load density, and physical characteristics of the wood element are taken into account for the charring rate.
are also visible differences between the two different calculation methods with real fire. In the initial minutes of the development of fire somewhat higher value of resistance provides effective section method, while the method of reduced strength gives larger values of resistance for long duration fires.

5 CALCULATION OF RELIABILITY OF GLULAM BEAM EXPOSED TO REAL FIRE

5.1 STATIC SYSTEMS AND DESIGN

The static system is simply supported beam. The laminated beam is a part of the roof frame system, and it is affected by dead load and snow. The value of the dead load is 0.45 kN/m² and the load of snow equals 2.74 kN/m² (location of the city of Gospić was taken as an example). Distance between the main girders is 3.3 m. Timber class GL24k according to EC5 [9] was used. The beam was dimensioned with Eurocode 5 [8] according to the ultimate limit state for bending affected by the real fire (reduced strength and stiffness method). The most critical cross section was analyzed. Shear was not decisive, and checking the serviceability limit state in case of fire is not necessary. Beam is laterally supported. Using traditional design in accordance with Eurocode 5 [9], with no fire effect, minimal heights of cross section (complete capacity) are shown in Table 1.

Table 1: Minimal cross section heights obtained from the bending capacity without fire

<table>
<thead>
<tr>
<th>Range [m]</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height [cm]</td>
<td>24</td>
<td>30</td>
<td>36</td>
<td>42</td>
<td>48</td>
<td>54</td>
<td>60</td>
<td>66</td>
<td>72</td>
</tr>
</tbody>
</table>

Total action is calculated according to (6):

\[ M_{y,d,f} = M_{x,d,f} = \frac{G + P}{8} \times L^2 \]  (9)

where \( M_{x,d,f} \) = design action in fire, \( G \) = characteristic value of permanent load per m², \( P \) = characteristic value of live load per m², \( L \) = beam span in m.

Total resistance in case of fire is calculated in accordance with [8]:

\[ M_{Rd,f} = W_{ef} \times b_{ef} \times k_{mod,f} \]  (10)

\[ b_{ef} = b - 2 \left[ d_0 + \beta_{par}(1.5 \times t_0 - \frac{t^2}{4t_0} - \frac{t_0}{4}) \right] \]  (11)

\[ b_{ef} = b - \left[ d_0 + \beta_{par}(1.5 \times t_0 - \frac{t^2}{4t_0} - \frac{t_0}{4}) \right] \]  (12)

\[ f_{20} = k_f \times f_{w,k} \]  (13)

where \( M_{Rd,f} \) = design capacity of components in case of fire, \( W_{ef} \) = reduced section modulus after exposure to fire, \( h_{ef} \) = reduced cross-section height, \( f_{20} \) = 20% fractile strength at normal temperature, \( k_f \) = coefficient (laminated wood 1.15, hard wood 1.25). Calculation was made for spans between 4 and 12 meters under 30 minute fire and corresponding cross-sections of full capacity were obtained.

Figure 6: Comparison of the necessary cross-section height for the given beam with fire design and without fire design (EC5)

5.2 FORMING LIMIT STATE FUNCTION

The reliability analysis is done for a beam with dimensions b/h = 20/36 cm and a span of 6 meters. Reliability analysis was made with computer software VAP. The limit state function is formed for the maximum bending stress. The general equation is (11):

\[ Z = X_1 \times R - X_2 \times E = 0 \]  (14)

where \( X_1 \) = model uncertainty for designed resistance in fire, \( R \) = designed resistance of material, \( X_2 \) = model uncertainty for actions on structure, \( E \) = actions on structure.

The limit state function for the maximum bending stress in fire condition case is (12):

\[ Z = X_1 \times \frac{b_{ef} \times h_{ef}^2}{6} \times f_{w,k} - 1.2 \times \left( \frac{G + P}{8} \times L^2 \right) = 0 \]  (15)

where \( X_1 \) = model uncertainty for designed resistance in fire, \( h_{ef} \) = reduced beam width, \( h_{ef} \) = reduced beam height, \( f_{w,k} \) = bending strength, \( X_2 \) = model uncertainty for actions on structure, \( G \) = permanent load, \( P \) live load, \( L \) = beam span (6m).

Figure 7: Reliability indexes considering different spans for ultimate limit state prior to fire exposure
5.2.1 Basic variable modelling
Upon defining limit states basic equations were modelled. The probability of failure as a function of coefficient of variation of strength is shown in Figure 8. Various variations of coefficient of variation of strength and coefficient of variation for model uncertainty for designed resistance give us indexes of reliability and failure probability. As recommended in literature [10] and [11], variation coefficients of strength of 0.15 and variation coefficient of model uncertainty for designed resistance with value of 0.15 were taken into account.

Recommendations for distributions and remaining variation coefficients were also taken from [11, 12 and 13]:

i) permanent load: normal distribution (G), mean value 1.5 kN/m², coefficient of variation \( V_G = 0.1 \)
ii) snow load (P): Gumbel's distribution, mean value 5.8 kN/m², coefficient of variation \( V_P = 0.3 \)
iii) beam span (b): normal distribution, mean value 15.8 cm, coefficient of variation \( V_b = 0.01 \)
iv) beam height (h): normal distribution, mean value 33.9 cm, coefficient of variation \( V_h = 0.01 \)
v) bending strength \( (f_{mk}) \), lognormal distribution, mean value 31.02 N/mm², coefficient of variation \( V_{f_{mk}} = 0.15 \)
vi) Characteristic bending strength is given within norms as 5% fractile and for wood class GL24k it is \( f_{mk} = 24 \) N/mm².

Table 2: Statistic parameter values of basic variables

<table>
<thead>
<tr>
<th>Basic variable</th>
<th>Distribution</th>
<th>Mean X</th>
<th>Standard deviation ( \sigma )</th>
<th>Coefficient of variation ( V )</th>
</tr>
</thead>
<tbody>
<tr>
<td>G (kN/m²)</td>
<td>N normal</td>
<td>1.5</td>
<td>0.15</td>
<td>0.1</td>
</tr>
<tr>
<td>P (kN/m²)</td>
<td>G Gumbel</td>
<td>5.8</td>
<td>1.73</td>
<td>0.3</td>
</tr>
</tbody>
</table>

5.2.2 Analysis results
Table 3. presents reliability indexes for a beam with span of 6 m. Reliability class of RC2 was chosen which belongs to middle class effects, ie, significant human and material losses. Figure 9. presents bending weighting factors for 30 minute fire. The image shows that most significant factors are variable load, wood strength and model uncertainty factors. Permanent load and the cross-section have very little influence in bending without fire.
Also, 17 different beams with spans between 4 and 12 meters have been analysed and 17 different limit state equations have been obtained. Beam width remains constant while the beam height changes according to Table 5.1. Accordingly, the values of reduced section width and height are also changing. The level of reliability which resulted according to level 2 is minor than the one which is prescribed by Eurocode for effective section method, while 80% percentage of the samples have sufficient reliability when method of reduced strength have been obtained.

6 CONCLUSION

Reliability of glulam beams in case of fire has been analysed and basic hypotheses about timber structures in fire are described in this paper. According to European standards, new concept of structures in fire is shown. A special review is given on the design of the real fire and the two representative methods for design. Comparison between level I (Eurocode) and level II (FORM - First Order Reliability Method) was made. Differences between the design procedures in EN 1995 and in EN 1995-1-2 were described. They shown that the procedure according to EN 1995 is not enough, and fire as an action must be taken into account. This leads to the conclusion that when increasing demands on the structure, level of reliability of structures designed according to Eurocode 5 significantly changes and for structures below the level RC3 is insufficient. The application of the new principle had been shown on the example of a laminated wooden beams exposed to fire. Reliability indexes were obtained from computational analysis of beams exposed to 30 minute fire. The calculated values are based on the range of the beam which varied from 4 - 12 m. Figure 11. shows the reliability index in the Eurocode (reliability class RC2) compared to the reliability indexes obtained by the methods of reduced strength and effective section. The reduced strength method gave us a higher value of reliability. The obtained reliability indexes are satisfactory considering that the structure is exposed to 30-minute fire and the full value of the permanent and live load.

Table 3: Calculated and standardized reliability indexes with the corresponding failure probabilities [10]

<table>
<thead>
<tr>
<th>Limit state to bending in the fire (30 min)</th>
<th>Calculated indexes of reliability/Probability of failure</th>
<th>Standardized reliability index for 50 years (RC2) for a surface area of 2500 m² h₀</th>
<th>Probability of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective section method</td>
<td>2.69 / 0.0036</td>
<td>3.03</td>
<td>0.00116</td>
</tr>
<tr>
<td>Reduced strength and stiffness method</td>
<td>3.04 / 0.00304</td>
<td>3.03</td>
<td>0.00116</td>
</tr>
</tbody>
</table>

REFERENCES

STRENGTH AND SERVICEABILITY – EXTREME EVENTS

FIRE PROTECTION ABILITIES PROVIDED BY GYPSUM PLASTERBOARDS

Alar Just¹, Joachim Schmid², Birgit Östman³

ABSTRACT: Gypsum plasterboards and gypsum fibreboards are often used as cladding on timber frame assemblies. Being the first layer on the fire exposed side, the protective properties of the boards are important for the whole assembly, especially for load bearing structures. Different design rules for the protection effect of gypsum plasterboards are presented and compared in the paper.

KEYWORDS: Gypsum plasterboards, failure time, start time of charring, fire resistance

1 INTRODUCTION

Timber frame assemblies consist usually of a timber frame, insulation in the cavities and cladding. The fire resistance of load bearing timber structures depend on the remaining cross-section due to the charring depth of the timber members. Different charring rates apply depending on whether they are initially protected or initially unprotected from direct fire exposure. Charring starts with slow charring behind a protective cladding. This is the case when the cladding remains in place after start of charring. When the cladding falls off, charring increases at a much higher rate than that of initially unprotected wood. The start time of charring and the failure time of gypsum boards are therefore important properties for the fire safety design of timber frame construction.

There is an order of importance of the contribution of components to the fire resistance of an assembly. The greatest contribution is provided by the cladding on the fire-exposed side that is first directly exposed to the fire, both with respect to insulation and to failure (fall-off) of the cladding. In general, it is difficult to compensate for poor fire protection performance of the first layer by improved fire protection performance of the following layers.

Gypsum plasterboards and gypsum fibreboards are often used as cladding. The gypsum core consists of natural gypsum, industrial gypsum and/or recycled plasterboards. Gypsum fibreboards have cellulose reinforcement. There are also other non-combustible claddings in use, for example magnesium silicate board, cement fibreboard etc. The fire resistance of assemblies made with non-combustible panel products depends on several important interrelated properties: The thermal insulation of the board; the ability of the board to remain in place and not disintegrate or fall off after dehydration; resistance to shrinkage; the ability of the core material to resist ablation from the fire side during extreme fire exposure. Time related protective properties as start time of charring and failure time are needed for fire design models of timber structures. Fall-off due to thermal degradation is a failure which cannot be calculated using finite element programs due to the complex failure mechanism.

In this paper available methods and possibilities for protection times of different claddings are presented and compared.

2 DESIGN METHOD BY EN 1995-1-2

The design for large timber members as well as timber frame structures in fire follows the fire part of a structural Eurocode 5 part 1.2. - European standard EN 1995-1-2 [1]. Charring is a central parameter for determining the degree of fire resistance of a timber member: the original cross-section must be reduced by the charring depth. Charring rates of wooden studs are significantly different before and after the protective claddings fall-off. The failure time of boards is therefore an important parameter for design of timber framed structures. For timber members the charring rate increase when the protective cladding has fallen down. The residual cross-section and load bearing capacity decreases significantly. Figure 1 shows the different phases for calculating bearing capacity of timber structures in fire. Break points are depending on important input values as start time of charring and failure time of the cladding.

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No charring occurs during Phase 1 until a temperature of 300°C is reached behind a protective layer. Phase 2 is referred to as the protection phase, and protection is assumed to remain in place until the end of this phase – which is failure time $t_f$. The charring is relatively slow during this phase. Phase 3 is the post-protection phase, and begins at failure time $t_f$. Charring is fast due to the lack of fully developed char coal layer as thermal barrier. Start time of charring $t_{ch}$ is the time when charring of wood starts behind the cladding. It is the time from the beginning of fire to reaching 300 °C on wooden surfaces behind cladding. The principle is in accordance with [1].

Fall-off is a failure which cannot be calculated using finite element programs due to the complex failure mechanism. Failure could occur because of pull-out of fasteners or due to thermal degradation of the board. EN 1995-1-2 gives equations for calculation of the start time of charring for gypsum plasterboards, but requires that cladding failure times are given by the producer based on full scale fire tests. However, the producers can normally not provide those important properties and there is no standard in use for establishing them.

3 ANALYSIS OF DATABASE

To provide data for the design model in EN 1995-1-2, independent of producers, a database of full-scale fire tests with gypsum boards was collected and analysed at SP Trätek [3]. The database consists of over 350 full scale test results from different fire laboratories all over the world. The analysis of the data shows a large variety of properties of products fulfilling the requirements for gypsum plasterboard Type F. Scatter can be observed within types and between different producers. Conservative equations are worked out taking into account different productions. See examples in Figure 2. Equations for protective times for single and double layer gypsum plasterboards, Type A and Type F at wall and floor structures are published in the European Technical Guideline Fire Safety in Timber Buildings [10]. The principle of those equations is to cover all known range of products of relevant types in Europe.

Figure 1. Protection phases for timber member behind the cladding

Figure 2. Failure times from different fire tests and provided conservative equations for single (1/1) and double layer (2/2) claddings of gypsum plasterboards, Type F at wall construction [3].
The present results in the database [3] are subjective and based on visual observations during testing. It is assumed that the recorded failure time is always longer than the real. Seldom the observer watches through the window of the furnace and waits for the cladding failure. Data about fasteners as dimensions, edge distance etc are not always specified in test reports.

Figure 3 shows an analysis of the database related to temperatures behind gypsum plasterboards, Type F, at fall off time. With some exceptions the fall-off temperature is 500 to 850°C for floors and 600 to 850°C for walls. König et al [4] report that the critical falling-off temperatures are 600°C for ceiling linings and 800°C for wall linings.

4 IMPROVED COMPONENT ADDITIVE METHOD

An improved component additive method for integrity and insulation analysis [9] has been created at ETH Zürich. The method can also be used for calculating the start of charring time as a protection time for cladding layers. The method is based on extensive experimental results and finite element thermal analysis. It is therefore applicable and offers more precise solutions for a greater variety of cladding materials compared to the earlier component additive method. According to the improved method, the contribution to the fire protection of the cladding may be assumed to be satisfied where the average temperature rise over the whole exposed surface of the particleboard is limited to 250K, and the maximum temperature rise at any point on that surface does not exceed 270K [9].

The method described in [9] can also be used for calculating the start time of charring by summing protection times of cladding layers.

\[ t_{ch} = \sum t_{prot,i} \]  

(2)

where \( \Sigma t_{prot,i} \) is the sum of protection times of \( i \) layers protecting timber members.

The protection time of each layer is taking the effect of preceding layers and backing layers into account respectively, if any.

For gypsum plasterboards, Type F correction times are used. Those times increase the protection time of the layer backed by gypsum plasterboard, Type F.

5 K-CLASSES

A European system with K classes for the fire protection ability of building panels has been introduced and is defined in EN 13501-2 [5]. The K classes are based on full scale fire resistance testing at horizontal orientation according to EN 14135 [6], with the main parameter being the temperature rise behind the panel after different time intervals (10, 30 and 60 minutes). Collapse or falling parts are not permitted. The K classes originate from the Scandinavian countries, where they have been used mainly for gypsum plasterboards, since the criteria originally also included reaction-to-fire requirements.

The fire protection class K expresses the ability of a wall or ceiling cladding to protect the material behind the covering against ignition, charring and other damage for a specified period of time.

Figure 4. Insulation time for different boards [11]

There are two types of K classes, K₁ and K₂ depending on the substrate behind the covering. K₁ is defined only for 10 minutes and substrates less than 300 kg/m³. K₂ is defined for all time intervals (10 min, 30 min or 60 min) and includes all substrates. The prescribed protection is in both cases is fulfilled if during a test in accordance with [6] within the classification period (10 min, 30 min or 60 min):

- there is no collapse and no burning of the covering or parts of it
- the mean temperature of the substrate does not exceed the initial temperature by more than 250 K

There are single layer gypsum boards, Type F with thickness of 15 mm have often being classified to K₂₃₀ [10]

6 FIRE TESTING

Fire testing of assemblies gives usually less conservative results for fire resistance of the whole structure compared to calculation methods. Testing is good for optimising set-ups of wall or floor assemblies. On the other hand testing is time and money consuming.

Important European standards for testing of timber frame assemblies with gypsum plasterboards in fire are EN 1364-1[12], 1364-2[13], 1365-1[14], 1365-2[15].
Observations of failure time are not mandatory when performing full-scale fire test according to the European test standards, but such observations have been recommended to be mandatory to the Fire Sector Group of Notified bodies for the CPD. Test report shall include the observations of failure times of claddings at the fire exposed side during fire resistance testing of wall and ceiling assemblies [ref 16].

Instrumentation of test structures with thermocouples inside the structure is not required by EN 1364 and EN 1365 [12-15]. Therefore the instrumentation is not used. Still the instrumentation is strongly recommended because of getting and documenting more information about the behaviour of different parts of the structure in fire.

Often certified structures are published by producers with declared values of fire resistance for designers and end-users. Test results of such complete structural assemblies are limited to tested set-ups. Change of components in the structure makes the result invalid and it may need new testing of the whole assembly. Although, extended applications are often possible, but quite complicated according to European standards.

Failure times and start times of charring can be determined by prEN 13381-7 [7] that will probably be ready by 2013-2014. It is a promising tool for determining protective times of gypsum plasterboards by means of model scale and full-scale testing.

7 RESULTS AND DISCUSSION

There are different methods available for calculating start time of charring behind gypsum plasterboards. Comparison of the methods is shown in Figure 5.

If the specific product or specific material characteristics as input values are not known, then the conservative routine according to the European technical guideline Fire Safety in Timber Buildings [10] to calculate failure times may be used. The design methods in handbook are based on the same safety level as Eurocode 5.

Otherwise, values from producers are more appropriate. ETH method (see Chapter 4 in [10]) can be used for calculation of start time of charring $t_{ch}$.

K-classes can be used as protection times when charring should be avoided, i.e mainly for load bearing structures. K-classes can be counted as start time of charring containing extra safety margin.

Producers of claddings should provide their own values of failure times to utilise better performance of their products. The procedure to change European standards for gypsum and other boards to allow for specification of failure times is very slow. The recommended intermediate way to classify different boards for timber frames according to protection times is to use the European Technical Approval procedure with certain prescribed test set-ups. ETAG 018/4 [8] might be extended and used as procedure to determine the fire protection times of claddings for timber frame assemblies.

Existing test results should be handled carefully. The older fire test results may be used only if proved that production is not changed meanwhile.

A new procedure is needed for determining protective times provided by gypsum plasterboards to the timber members in fire, based on test results. Product standard EN 520 should be revised adding information on structural performance for fire rated boards, Type F.

New data are needed for the revision of the fire part of Eurocode 5, that is planned to start in 2013.

ACKNOWLEDGEMENT

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EXPERIMENTAL BEHAVIOR OF WOOD COLUMNS UNDER EXTREME LOADING: CYCLIC BUCKLING

Faouzi Bouras¹, Myriam Chaplain², Zahreddine Nafa³, Denys Breysse⁴, Huyen Tran⁵

ABSTRACT: Wood and wood products are increasingly used as construction materials. Today, designers have learned to design wood structures based on engineering principles. However, studies on wood behavior in seismic zones are almost nonexistent. Thus, we carry out for wood columns, two set of cyclic compression loads leading to buckling. The aim of these experiments is to understand and to analyze wood column’s behavior in seismic zones. The first cyclic program is led in post buckling: specimens are subjected to cyclic buckling lead in post-instability. The second one is carried out in pre buckling: cyclic buckling before instability.

KEYWORDS: Wood, buckling, post buckling, pre buckling, instability, cyclic tests

1 INTRODUCTION

Wood and wood products are increasingly used as construction materials; however, studies on cyclic buckling wood columns are limited. In static Fairker [1] studied wood columns of different lengths and species. He treated the influence of buckling length / thickness ratio on the resistance. Agarwal [2] achieved an experimental work on solid timber columns of small cross-sections. The variables investigated include breadth to depth ratio, and wide range of slenderness ratio, one each, of conifers. The study indicates that imperfections in material, initial curvature of strut and eccentricity of loading can be considered by using a proper value of \( \eta \) in Perry’s formula. Reduction factors for evaluating the strength of such struts for Indian timbers have been reported.

Studies on cyclic buckling are carried out on other materials (concrete, steel, composites…), for example we cite works achieved by Broderick [3], Gomes [4] and Kyungsoo [5]. For the wood columns this kind of study is no available. Cyclic torsion tests on wood beams are realised [6]. Thus, this study concerns the experimental analysis of behavior of structural components of wood subjected to cyclic loading - seismic types - leading to instability such as buckling. This work is part of a process to better design the wooden structures in seismic zones. Two cyclic compression programs have been carried out, first loaded in post-instability (post-buckling) [7] and the second in pre-instability (pre-buckling). The experimental device is set up as quasi-articulated buckling (Figure 1). The tested material is solid wood (spruce) and laminated veneer lumber (LVL). In post buckling, glulam specimens have also been tested. The protocol of cyclic tests is inspired by those used to simulate seismic actions and in particular those proposed by Ceccotti [8].

Figure 1: Experimental device

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2 EXPERIMENT

Specimens studied are realised in solid wood (spruce), glulam (GL28), laminated wood (Laminated Veneer Lumber Kerto-S). These specimens are a homothetic to columns which can be found in timber structures. They are stored in a room at constant temperature of 20°C and 65% relative humidity. The average size of specimens is: (800 x100x17.5 mm), density and moisture content are listed in Table 1.

Table 1: Specific gravity and moisture content of the specimens

<table>
<thead>
<tr>
<th></th>
<th>Nb</th>
<th>SG</th>
<th>MC (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW</td>
<td>36</td>
<td>0.44±0.03</td>
<td>12.3±1.4</td>
</tr>
<tr>
<td>LVL</td>
<td>17</td>
<td>0.54±0.02</td>
<td>12.2±1.7</td>
</tr>
<tr>
<td>GL</td>
<td>36</td>
<td>0.48±0.01</td>
<td>12.1±0.4</td>
</tr>
</tbody>
</table>

Nb, SG, MC respectively number of specimens, specific gravity and moisture content. Results are average ± standard deviation. SW and GL are respectively solid wood and glulam.

The columns are inserted into the grooves cut on the sides of cylindrical attachments and supported between V-notched attachments (Figure 2).

![Figure 2: support details](image)

During testing, the time, the displacement of the crosshead of the machine, the compressive force applied and the eccentricity of the beam in the middle are measured. The record of the eccentricity was performed using a displacement sensor (LVDT). The test machine used is a 20 Tons hydraulic press.

3 EXPERIMENTAL SCHEDULE

3.1 STATIC TESTS

Before cyclic tests, monotonous-static-tests are performed. At the beginning of instability we note the crosshead displacement as $(\delta_e)$ for post buckling tests and the corresponding load $(F_e)$ for pre-buckling tests. The experiments speed is 1mm/min. Figure 3 (load-crosshead displacement) shows an example of the behaviour of the three species used: glulam (GL), LVL and solid wood (SW). The determination of the displacement $\delta_e$ and the load $F_e$ is presented for solid wood. The average value of $\delta_e$ is around 2 mm for the three species. The obtained values of $(\delta_e)$ are used to establish the post-buckling protocol. The values of loads $(F_e)$ used for establish the pre buckling protocol are expose in table 3 section 3.3.1.

![Figure 3: Force-displacement curves for the three used species for post buckling tests](image)

Table 2 resumes the mean values obtained of instability forces, instability displacements and eccentricity (at the middle of beam).

Table 2: Static tests results for the three kinds of specimens for post-buckling

<table>
<thead>
<tr>
<th></th>
<th>Fe (kN)</th>
<th>Displacement (mm)</th>
<th>Eccentricity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW</td>
<td>15.63±3.8</td>
<td>1.76±0.33</td>
<td>10.7±2</td>
</tr>
<tr>
<td>GL</td>
<td>20.14±4</td>
<td>1.64±0.36</td>
<td>12.8±2.4</td>
</tr>
<tr>
<td>LVL</td>
<td>20.64±5.5</td>
<td>1.87±0.48</td>
<td>15.5±3</td>
</tr>
</tbody>
</table>

& standard deviation

3.2 POST BUCKLING TESTS

In post-buckling we consider that the instability of the column in the structure corresponds to a local instability which does not affect the overall response of the structure. The testing machine is controlled in displacement. The protocol is inspired by those used to simulate seismic actions and in particular those proposed by Ceccotti [8]. The protocol proposed has been amended to facilitate the processing of results and it is a mix between a rapidly growing cycle test and a fatigue test. Analysis of monotonous buckling curves allows determining the load cycle shape: after a pre-load at a speed of 50 mm / min until $\delta_e$ (displacement in the beginning of the instability under static loading), the specimens are submitted to periodic triangular loading with a constant amplitude equal to 0,5$\delta_e$ [7] and at a speed of 290 mm/min. The cyclic loading is done by "packages" of 50 successive cycles with a gradual increase of 0,5$\delta_e$ between packages. In the first package, the maximum displacement is equal to 2,25$\delta_e$ (Figure 4). During testing, the eccentricity is measured at midheight of the beam.
3.3 PRE-BUCKLING TESTS

3.3.1 Bending tests

We carried out static bending tests before and after pre-buckling tests, in order to evaluate the Young modulus and its evolution during cyclic tests. Here, only solid wood and LVL (Kerto-Q) specimens are tested. Young modulus values allow us to have “theoretical” Euler critic load by using the expression:

\[ F_e = \frac{E \cdot I}{L_b^2} \]  \hspace{1cm} (1)

Where \( E \) is Young modulus in longitudinal direction, \( I \) is moment of inertia and \( L_b \) is specimen’s buckling length.

The obtained “theoretical” values of \( F_{eth} \) (bi-articulated system) and comparing them to experimental results of buckling loads \( (F_e) \), we noted a big difference. The experimental results in static tests were overestimated. Because of that we proposed to place a thick teflon sheet (0.6mm) (Figure 2) and a total lubrication of the devices. Figure 5 shows that behavior with old device and new device in static test is not similar for the same beam.

Table 3 summarizes the average Young’s modulus calculated before and after pre-buckling cyclic tests, the corresponding theoretical instability loads \( F_{eth} \), experimentally instability loads \( F_e \) and \( F_{e50} \) (\( F_{e50} \) is 50% of the cumulated distribution load).

Figure 4: Protocol of post-buckling cyclic tests

![Figure 4: Protocol of post-buckling cyclic tests](image)

Figure 5: Behavior in static test for the same beam (LVL) with the old and the new (lubricated) device

![Figure 5: Behavior in static test for the same beam (LVL) with the old and the new (lubricated) device](image)

Table 3: Mean values of Young modulus, theoretical instability forces and experimental instability forces

<table>
<thead>
<tr>
<th></th>
<th>BCL</th>
<th>ACL</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_{eth} )</td>
<td>( E_1 )</td>
<td>( F_{eth2} )</td>
</tr>
<tr>
<td>SW</td>
<td>6.78</td>
<td>8929</td>
</tr>
<tr>
<td>sd</td>
<td>1.23</td>
<td>1626</td>
</tr>
<tr>
<td>LVL</td>
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<td>6377</td>
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<tr>
<td>sd</td>
<td>0.88</td>
<td>1169</td>
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</table>

BCL, ACL: respectively before and after cyclic loading.

3.3.2 Pre-buckling tests

The test machine is controlled in force. Specimens are tested in pre-instability phase (before instability). We consider that buckling of the element of structure is harmful to all structure. Beams are pre-charged with a speed of 50mm/min till 9% of \( F_{e50} \). Then, they are subjected to a periodic triangular loading with a maximum of 90% of \( F_{e50} \) representing constant amplitude equal to 81% of \( F_{e50} \). The speed of tests is 24kN/s with a theoretical frequency of 1 hertz. The loading increases by 9% of \( F_{e50} \) every 100 cycles (Figure 7).

Figure 6: Cumulated frequency distribution of experimental forces \( F_e \) (static tests) and theoretical forces \( F_{eth} \) (calculated from static bending tests): determination of \( F_{e50} \)

![Figure 6: Cumulated frequency distribution of experimental forces \( F_e \) (static tests) and theoretical forces \( F_{eth} \) (calculated from static bending tests): determination of \( F_{e50} \)](image)

Figure 7: Force-displacement curve for solid wood and LVL beams

![Figure 7: Force-displacement curve for solid wood and LVL beams](image)
STRENGTH AND SERVICEABILITY – EXTREME EVENTS

SLE50 values in Figure 6, define load level references as:

\[ S_{E50} = \frac{F_{max}}{F_{E50}} \]  

Where \( F_{max} \) represents the maximum force for each 100 cycles.

4 CYCLIC RESULTS

4.1 POST BUCKLING

4.1.1 Results and discussion

Figure 8 shows an example of load-eccentricity curves obtained during testing of glulam wood. The curves for other species of material are similar.

After instability, when the displacement exceeds the \( \delta e \) value, the strength usually half decreases. Then, force continues to decline, by cons, the stresses in the specimen increases until it reaches the limit of the material due to the increase of eccentricity (Figure 9). The evolution of maximum stress in the central part of the beam is presented in Figure 9 for the same glulam column shown in Figure 8. We can observe the increase of stress before the brittle fracture by tension.

4.1.2 Damage analysis

The damage is studied by using damage theory. It is characterized by a parameter \( D \) which varies between 0 when the material is not damaged and \( D_c \) when failure appears. We take \( D_c=1 \). D is obtained by analyzing the evolution of stiffness (Figure 10). The loss of rigidity is expressed as follow:

\[ D = 1 - \frac{K}{K_0} \]  

Where \( K \) is the force-displacement curve slope, \( K_0 \) the initial slope.

The second way to obtain a damage parameter is to study the evolution of the hysteresis curves. We can observe in Figure 11 that the energy dissipated during a cycle decreased when the material is damaged. Thus the parameter \( D \) is:

\[ D = 1 - \frac{E}{E_0} \]  

Where \( E \) is the surface of hysteresis loop and \( E_0 \) the initial one.
Figure 11: Stiffness and energy evolution during cyclic tests

Figure 11 shows that the calculated energy decreases through cycles. Thus, wood is able to accumulate and dissipate energy during severe action like seismic one. Also, stiffness degradation is observed. The decrement of stiffness and dissipated energy shows that there has been damage and/or a loss of viscosity during the cyclic test. Calculation of damage parameter D according to energy E and stiffness K are presented in Figure 12. Curves are globally similar. The presented examples concern LVL and glulam. Because of the noisy machine test results, we find some negative values (especially for energy values) at the beginning of cyclic tests (it has no physical sense). In addition we found a lot of dispersion. So, for the rest of calculation, parameter D is function of stiffness K.

Figure 12: Variation of parameter D according to energy and stiffness for solid wood

For each specimen and for each cyclic test the parameter D is calculated. At the beginning, the extreme fiber stresses (without damage) expression are:

\[
\sigma_{cr} = \frac{F}{S} \pm \frac{M}{W}
\]  

(5)

Here \( \sigma \) is compression/tensile apparent stress (material considered not damaged), \( F \) the applied effort, \( S \) initial cross section, \( M \) the bending moment and \( W \) initial flexural modulus.

Introducing the damage effects, the extreme fibres stress are also calculated supposing that, during cyclic tests, the thickness is reduced as expressed in Equation (8). The width beam is assumed to be constant.

\[
S_c = S \times (1 - D)
\]  

(6)

\[
W_c = \frac{h \times b_c^3}{6} \quad b_c = b \times (1 - D)
\]  

(7)

Where \( S_r \) is reduced cross section and \( b_c \) is reduced thickness.

Real or effective stress will be:

\[
\sigma_{cr} = \frac{F}{S_r} \pm \frac{M}{W_c}
\]  

(8)

We present in table 4 calculation results of stresses at failure for the three species. The ratio between real stress and apparent stress, in compression, is about 2.5 for LVL and solid wood and equals to 3 for GL. In tensile the ratio is in the order of 2.6 for LVL and solid wood and 3.2 for GL.

Table 4: Mean values of Young modulus, theoretical instability forces and experimental instability forces

<table>
<thead>
<tr>
<th>D_f</th>
<th>( \sigma_c )</th>
<th>( \sigma_t )</th>
<th>( \sigma_{cr} )</th>
<th>( \sigma_{tr} )</th>
<th>R(1)</th>
<th>R(2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVL</td>
<td>0.71</td>
<td>66.3</td>
<td>57.1</td>
<td>171.3</td>
<td>155.5</td>
<td>2.6</td>
</tr>
<tr>
<td>Sd</td>
<td>0.1</td>
<td>13.7</td>
<td>12.6</td>
<td>26</td>
<td>60.3</td>
<td></td>
</tr>
<tr>
<td>SW</td>
<td>0.6</td>
<td>48.2</td>
<td>42.3</td>
<td>120.8</td>
<td>111.8</td>
<td>2.5</td>
</tr>
<tr>
<td>Sd</td>
<td>0.14</td>
<td>12.8</td>
<td>11.7</td>
<td>46.5</td>
<td>44.8</td>
<td></td>
</tr>
<tr>
<td>GL</td>
<td>0.64</td>
<td>57.2</td>
<td>49.11</td>
<td>170.3</td>
<td>156.1</td>
<td>3.0</td>
</tr>
<tr>
<td>Sd</td>
<td>0.06</td>
<td>12.8</td>
<td>12.8</td>
<td>51.8</td>
<td>51.7</td>
<td></td>
</tr>
</tbody>
</table>

Sd: standard deviation
R(1), R(2): are respectively ratio (\( \sigma_{cr}/ \sigma_c \)) and (\( \sigma_{tr}/ \sigma_t \))

4.2 PRE BUCKLING

During cyclic tests, wood has hysteretic behavior as for post buckling tests (see Figure 8 for post buckling and Figure 13 for pre buckling). Thus wood behavior is described as visco-elastic (hysteresis loop).

Figure 13: Monotonous and cyclic test response for solid wood (pre-buckling)

Hysteretic behaviour denotes that wood is able to accumulate and dissipate energy during severe action like seismic one. The load increases constantly during pre buckling tests until the instability, unlike the post-buckling tests where force decreases. The Young
modulus evolution, calculated in table 3 before and after cyclic tests is quasi constant. It seems that wood has not submitted a significant damage.

Introducing the expression of $SL_{e50}$ (see Equation 2), the maximum load level $SL$ is defined as:

$$SL = \frac{F_{\text{max}}}{F_e} \times \frac{F_{e50}}{F_e} = SL_{e50} \times \frac{F_{e50}}{F_e}$$

Where $F_{\text{max}}$ is the maximum load applied, $F_e$ is the specimen static instability force, $F_{e50}$ is the reference force defined in Section 3.3.1

For each specimen, applied load level $SL$ was: $SL_{e50}$, $F_{e50}/F_e$. As $SL_{e50}$ is not depending on the specimens (only on the specie), the number of cycles until instability (instability time) is presented versus the ratio $F_{e50}/F_e$ (Figure 14). LVL and solid wood points coincide for small values of static forces $F_e$ and they are distinguish for high values of $F_e$. For the low values of ratio $F_{e50}/F_e$, LVL has instability time smaller than solid wood.

![Figure 14: Comparison of instability's time curves between solid wood and LVL](Image)

**5 CONCLUSIONS**

Two set of cyclic compression tests have been carried out on solid wood and wood based slender columns. The first one is placed before buckling and the second one after buckling. Behavior in monotonic test until the instability has appeared quite independent of the specimen specie. The cyclic tests in post-buckling, show brittle fracture by traction for solid wood and GL and detachment of layers veneer for LVL. The prediction of number of cycles to failure is studied using a damage theory. A damaged parameter is determinate through the slope change of the compression stress versus eccentricity curves. Spruce, glulam and LVL have a viscoelastic behavior with a significant capacity of energy dissipation (pronounced hysteresis loop). In pre-buckling tests, the Young’s modulus calculated before and after cyclic tests are unchanged. For low values of $F_{e50}/F_e$ the LVL has instability time smaller than solid wood. Cyclic tests are so difficult to lead that results are scattered. The dispersion of results can be also explained by the structure of wood material.

**REFERENCES**


ROBUSTNESS ANALYSIS OF STRUCTURAL TIMBER TRUSS SYSTEMS

Dean Čizmar¹, Vlatka Rajčić², Mislav Stepinac³

ABSTRACT: Progressive collapse is characterized by disproportion between the magnitude of a triggering event and resulting in collapse of large part or the entire structure. Robustness of structures has been recognized as a desirable property because of a several large system failures, such as the Ronan Point Apartment Building in 1968, where the consequences were deemed unacceptable relative to the initiating damage. After the collapse of the World Trade Centre, the robustness has obtained a renewed interest, primarily because of the serious consequences related to failure of the advanced types of structures. In the introduction different robustness requirements and probabilistic definitions are given. Special attention is given for timber structures. This paper will focus on a robustness of different timber structural systems that are commonly used in designers’ practise (these trusses are calculated according to EC5 for 100% utilization of elements). Comparison of these systems and a conclusion is given.

KEYWORDS: timber, truss systems, robustness, probabilistic model

1 INTRODUCTION

Progressive collapse is characterized by disproportion between the magnitude of a triggering event and resulting in collapse of large part or the entire structure. Robustness of structures has been recognized as a desirable property because of a several large structural system failures, such as the Ronan Point Apartment Building in 1968, where the consequences were deemed unacceptable relative to the initiating damage. After the collapse of the World Trade Center, robustness has obtained a renewed interest, primarily because of the serious consequences related to failure of advanced types of structures. In order to minimize the likelihood of such disproportional structural failures many modern building codes require robustness of the structures and provide strategies and methods to obtain robustness.

Robustness requirements are provided in two European documents: Eurocode EN 1990: Basis of Structural Design [1] and EN 1991-1-7 Eurocode 1: Part 1-7 Accidental Actions [2]. The first document provides the basic principles, e.g. it is stated that a structure shall be “designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause”. The EN 1991-1-7 document provides strategies and methods to obtain robustness, actions that should be considered and different design situations: 1) designing against identified accidental actions, and 2) designing unidentified actions (where designing against disproportionate collapse, or for robustness, is important).

In the JCSS Probabilistic Model Code [5] a robustness requirement is formulated as: “A structure shall not be damaged by events like fire, explosions or consequences of human errors, deterioration effects, etc. to an extent disproportionate to the severeness of the triggering event”. In order to attain adequate safety in relation with accidental loads, two basic strategies are proposed: non-structural measures (prevention, protection and mitigation) and structural measures (making the structure strong enough to withstand the loads limiting the amount of structural damage or limiting the amount of structural damage). According to Danish design rules robustness shall be documented for all structures where consequences of failure are serious. A structure is defined as robust when those parts of the structure essential for the safety have little sensitivity with respect to unintentional loads and defects, or that an extensive failure of the structure will not occur if a limited part of the structure fails. In the last few decades many definitions of robustness have been proposed. In this paper only a brief description of probabilistic measures relevant for robustness assessment is given.

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² Vlatka Rajčić, University of Zagreb, Faculty of Civil Engineering. Email: vrajcic@grad.hr
³ Mislav Stepinac, University of Zagreb, Faculty of Civil Engineering. Email: mstepinac@grad.hr
Frangopol and Curley [16] proposed a probabilistic structural redundancy index (RI):

$$RI = \frac{P_f^{(dmg)} - P_f^{(sys)}}{P_f^{(sys)}}$$ (1)

where $P_f^{(dmg)}$ is the probability of failure of a damaged system and $P_f^{(sys)}$ is the system failure probability (no damage). The redundancy index as defined above provides a measure of the residual strength of a damaged system. They also considered the following redundancy factor:

$$\beta_r = \frac{\beta_{int\ act}}{\beta_{int\ act} - \beta_{damaged}}$$ (2)

where $\beta_{int\ act}$ is the reliability index of the intact system and $\beta_{damaged}$ is the reliability index of the damaged system.

Lind [17] proposed a generic measure of system damage tolerance, based on the increase in failure probability resulting from the occurrence of damage. The vulnerability ($V$) of a system is defined as:

$$V = \frac{P(r_d, S)}{P(r_0, S)}$$ (3)

where $r_d$ is the resistance of the damaged system, $r_0$ is the resistance of the undamaged system, and $S$ is the prospective loading on the system $P(\cdot)$ is the probability of failure of the system, as a function of the load and resistance of the system. The vulnerability parameter indicates the loss of system reliability due to damage.

In this paper an index of robustness is defined as a ratio between the reliability index of a damaged structure ($\beta_{dmg}$) and the reliability of the intact structure ($\beta_{int}$):

$$I_{rob} = \frac{\beta_{dmg}}{\beta_{int}}$$ (4)

Also a robustness factor $F_{rob}$ can be defined as a ratio of probability of failure of damaged system ($p_f^{dmg}$) and probability of failure of intact system ($p_f$):

$$F_{rob} = \frac{p_f^{dmg}}{p_f}$$

Recently a new definition [12] of both the progressive collapse and the robustness is given. The probability of disproportionate collapse is calculated as a product of probabilities: the probability of an abnormal event that threatens the structure, the probability of initial damage as a result of event and the conditional probability of a disproportionate spreading of structural failure due to the initial damage. Based on this, there are the three main strategies to limit the probability of a disproportional collapse, first is to prevent the occurrence of abnormal events, the second is to prevent the occurrence of an initial damage in consequence of the occurrence of abnormal events. A third strategy is to prevent disproportionate spreading of failure of the initial damage. This part relates to the internal properties of the structure though its robustness. As such the robustness is a property that depends on the structure itself and the amount of initial damage.

## 2 ROBUSTNESS OF TIMBER STRUCTURES

In the last few decades there has been intensely research concerning reliability of timber structures but robustness of timber structures has not been shown much attention. For the purpose of the project “Timber Frame 2000” [11] a six-storey experimental timber frame building was erected, in order to investigate the performance and economic prospects of medium-rise timber frame buildings in the UK. As a part of a testing programme the investigation of disproportionate collapse (robustness) was conducted. Result obtained show that this kind of timber frame system is very robust. Since timber is a complex building material, assessment of robustness is difficult to conduct. As there is obvious correlation between redundancy and robustness, redundant structures will, in principle, be a more robust than statically determinate. However, in respect to timber structures, there are not many highly redundant systems, and the obvious way to assess a robustness of such structures is to demonstrate that the part(s) of the structure essential for the reliability have little sensitivity with respect to unintentional loads and defects. In this article is presented a robustness investigation of different timber truss systems.

## 3 PROBABILISTIC MODEL

Probabilistic calculations were done by First-Order Reliability Methods (FORM) where a reliability index is estimated based on limit state functions for each of the considered failure modes. The probabilistic analysis is performed with a stochastic model for the strength parameters for whole structural elements, and not to the strength for the single laminates and the glue. Second order effects are neglected for beams subjected to compression and combined compression and bending, respectively. Buckling problems and lateral buckling is taken into account as in Eurocode 5 with deterministic coefficients. For the structural analysis a linear Finite Element analysis has been performed where the glulam truss has been modelled by beam and truss elements. Furthermore, only permanent and snow loads are considered in probabilistic analysis. Identification of the significant failure modes of this structure is difficult to perform since there are many possible failure elements. Based on the deterministic structural analysis four different failure modes are considered: 1) combination of bending and compression (M+N) in the upper chord, 2) combination of bending and tension (M+N) in the lower
chord, 3) compression (N) and 4) tension in diagonal elements (N). The ultimate limit state failures are assumed to be brittle (i.e. when an element fails there is no bearing capacity left). The following failure elements are considered for these failure modes:

1. Failure in bottom chord (N+M)
2. Failure due to tension in diagonal element (N)
3. Failure due to compression in diagonal element (N)
4. Failure in top chord (N+M)

For the calculations permanent load $G$ due to self weight and a variable snow load are taken into account. The permanent load of the roof structure is assumed Normal distributed with an expected value $\mu_G = 0.5 \text{ kN/m}^2$ and a coefficient of variation $\text{COV} = 0.1$.

For the region in Croatia where the structure is located the annual maximum snow load at the ground is Gumbel distributed with a characteristic value $S_{gk} = 1.5 \text{ kN/m}^2$ and a coefficient of variation $\text{COV}=0.58$.

The strength variables $f_c$, $f_m$ and $f_t$ (compression strength parallel to grain, bending strength and tensile strength, respectively) are calculated based on the reference properties given in table 1 [7]. Table 2 shows all probabilistic variables taken into account (designation, distribution, mean value and coefficient of variation). Correlations between the stochastic variables are taken as in [5] and [7].

### Table 1: Reference properties and respective distributions

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending strength</td>
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<tr>
<td>Bending MEO</td>
<td>LN</td>
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<td>Density</td>
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### Table 2: Reference properties and coefficients of variation

<table>
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<td>Bending MOE</td>
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<td>Density</td>
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### Table 3: Stochastic variables and respective distributions

<table>
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<td>Model uncertain</td>
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<td>Bending strength</td>
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<td>Tensile strength</td>
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<td>Permanent load</td>
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<td>Snow load</td>
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### Table 4: Stochastic variables and coefficients of variation

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<td>Snow load</td>
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4 ROBUSTNESS OF TIMBER TRUSS STRUCTURAL SYSTEMS

Six different truss systems which are used in practise are chosen and designed according to EC5 for 100% utilization of members (chords and truss members). A serviceability limit state is also considered.

After the design of truss members, for each truss (see figures 1 to 6) the system reliability of the intact structure is calculated (an estimate of the failure probability is obtained as the arithmetic mean of the upper and lower probability bounds). For each of the previously defined failure modes a failure is assumed and a robustness index is calculated based on the definition. Figure 7 represents robustness indices for given truss systems. It can be seen that for most systems (RN1, RN3, RN4, RN5 and RN5) for at least one failure scenario robustness is 0, so it can be concluded that these systems, in respect to robustness,
can be considered as non robust. Truss system RN2 has much higher robustness indices (minimal robustness index $I_{rob}=0.24$ and corresponding failure probability $p_f=3.29 \times 10^{-2}$) so this system could be considered as more robust than others. This can be explained because in this system failure of a chord or a truss element won’t result in a collapse of the whole structure (parallel system).

**Figure 1**: Truss structure RN1

**Figure 2**: Truss structure RN2

**Figure 3**: Truss structure RN3

**Figure 4**: Truss structure RN4

**Figure 5**: Truss structure RN5

**Figure 6**: Truss structure RN6

**Figure 7**: Robustness indices

5 CONCLUSIONS

The paper considers robustness of structures in general and probabilistic approaches for robustness quantification. Special attention is given for timber structures.

Results of different timber structural systems (6 different) with respect to robustness index are given. For most of the systems robustness can be considered as very low (if minimal robustness index is chosen). It is shown that system with parallel elements (RN2) has a much higher robustness. This is because these systems have the possibility to redistribute the forces in structure. It can be concluded that if robustness is desired (high consequence class structures) these systems should be used.

It must be noted that other systems which are not considered robust are not “bad” and should not be avoided in engineering practice, but if used, special attention should be made upon minimizing the probability of failure of elements or joints.

It must be also noted that these results are made on brittle models and possible influence of both material and fastener ductility is not taken into account.

REFERENCES


AN EXPERIMENTAL STUDY ON MEASUREMENT OF SOUND INSULATION OF LIGHT WEIGHT WALL WITH LOESS BOARD USED IN KOREAN TRADITIONAL HOUSE(HAN-OK)

In-hwan Yeo¹, Kyung-suk Cho², Bum-yean Cho³, Byung-yeol Min⁴, Myung-o Yoon⁵

ABSTRACT: The purpose of this study is to evaluate residential performance of Korean traditional house(Han-ok) by comparing sound insulation performance of lightweight wall finished with the existing gypsum board and loess board developed with loess, Korean traditional house finishing material. The measurement of sound insulation performance was performed by test method of international standards, ISO 140-3 and ISO 717-1. The results of measurement showed that sound insulation performance of loess is better than that of gypsum board, and that all specimens applying only loess showed Grade 2 while those specimens applying only gypsum board of sound insulation grades presented by Korean building regulations indicated ‘out of grade’.

KEYWORDS: Building acoustics, Loess board, Lightweight wall, Korean traditional house(Han-ok)

1 INTRODUCTION

The Korean traditional house is called Hank-ok. It is generally constructed with post and beam timber structure. Loess has traditionally been used as one of main finishing materials for Han-ok. Loess as traditional material is easy to get around us but is substituted by cement and concrete due to fine dust and crack occurrence after hardening at modern times. However, the usage of it as a building finish material is increasing again with the interest of modern people with a view of qualitative improvement and health. If Han-ok will be popular, the lightweight wood frame with Loess Boards could be used to wall in Han-ok house such as apartment or public construction partition wall.

The purpose of this study is to evaluate the performance of the sound insulation for the partition wall finished with the loess of eco-friendly materials. The walls were composed of wood framed lightweight structure and dry finishing materials for the dry construction of Han-ok. In general, it is required that sound insulation performance of a certain level or higher to improve residence performance, sound insulation Grade 1-3 should be fulfilled in Korean building regulations. This study would examine the sound insulation performance resulted from the combination of gypsum board and loess board applied to the lightweight dry wall.

2 CONCEPT OF HAN-OK

2.1 HAN-OK

Han-ok is composed with the frame of woods and other natural materials such as loess, stone, straw and etc. Those materials are not change chemically, so have natural properties. Therefore, natural materials differ from modern materials, it is not induce Sick House Syndrome and not produce harmful construction wastes. Han-ok is a green building.

However, Han-ok is built of traditional construction methods because the cost of natural materials and man power could be high. For these reasons, the buildings have constructed limited in culture assets or specified buildings. Now, for taking over the traditional methods
and environment friendly construction, Han-ok is getting popular. For inheriting an advantage of Han-ok, it should be supplied with low costs. That is why it needs dry construction system than wet construction system.

**Figure 1: Han-ok in Korea**

In the past, Han-ok was composed of loess brick wall or loess finishing with straw or bamboo bone wall among wood frame.

The loess wall has advantages for the ventilation and the humidity control but it has disadvantages in the insulation and fine dusts and cracks on the wall. Therefore, there is a problem to deteriorate the residential quality compare to current building materials. Now, Han-ok is improving in the residential quality through modify past methods. For that reason, it is trying to apply dry construction walls than wet one, and eco-friendly insulations.

### 2.2 LOESS BOARD

Loess covers about ten percent (%) of the earth’s surface. It covers on the surface of the Korea plentifully.

Loess boards in this study has been made through Figure 2 process that it is composed of loess, straw, starch and etc., and attached paper on the back of the board. Table 1 shows the property of loess boards.

<table>
<thead>
<tr>
<th>List</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density(kg/m³)</td>
<td>741</td>
</tr>
<tr>
<td>Compressive</td>
<td>84.8</td>
</tr>
<tr>
<td>strength(N/mm²)</td>
<td></td>
</tr>
<tr>
<td>Flexure strength(N/mm²)</td>
<td>119.0</td>
</tr>
<tr>
<td>absorption(g/100 m²)</td>
<td>19</td>
</tr>
<tr>
<td>Thermal conduct ratio (W/mK)</td>
<td>0.114</td>
</tr>
</tbody>
</table>

**Figure 2 Making process of Loess boards**

2.3 **SOUND INSULATION IN KOREA**

In Korea, the performance criteria of buildings shall comply with Building Act and Housing Act. This study is limited to sound insulation among the performance criteria of buildings.

Party/Partition walls between household units or between among detached houses, between bedrooms, classrooms, guestrooms shall have a fire resistance rating and a sound insulation in accordance with Enforcement Decree of the Building Act, Article 53. Also, the sound insulation performance shall have of a specific thickness of reinforced concrete structures, steel framed reinforced concrete structures, masonries, concrete block s, and cement blocks in accordance with the Korean Building Code (KBC). Other structures shall be determined by the prescribed test or the insulation material approved by the authority.

According to the results of research on the sound insulation in Korea, K.W. Kim et al mentioned that effective factors of the sound insulation of lightweight dry walls are following: density of materials, kinds and density of insulation, kinds and methods of installation of stud, thickness of wall, method of isolation, distance of screws, resilient channel and etc.

This study intends to evaluate the sound insulation according to kinds of boards and existence of resilient channel for the lightweight wood framed walls.
3 EXPERIMENT STUDY

3.1 ASSESSMENT METHOD FOR AIRBORNE SOUND IN KOREA

In Korea, the sound insulation performance of partition walls and external walls prescribed shall be classified in the same grades as in Table 2. Sound insulation grades were calculated and determined with the converted value of acoustical attenuation constant (dB) by frequency band measured following ISO 140-3 into single-number quantity for airborne sound and spectrum adaptation term.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Information(dB)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$58 \leq \text{Rw} + \text{C}$</td>
</tr>
<tr>
<td>2</td>
<td>$53 \leq \text{Rw} + \text{C} &lt; 58$</td>
</tr>
<tr>
<td>3</td>
<td>$48 \leq \text{Rw} + \text{C} &lt; 53$</td>
</tr>
</tbody>
</table>

$\text{Rw}$ : single-number quantity for airborne sound  
$\text{C}$ : spectrum adaptation term

It was installed between the sound resource room and the sound receiving room, and tested Sound Reduction Index by ISO 140-3. The test had been performed from February to June in 2011. Laboratory’s atmosphere temperature was $15.0 \pm 2.0$ °C, relative humidity was $45.0 \pm 2.0\%$. The specimen frame size was 3.6m x 2.7m, a gap between specimen and frame was filled with silicone seals. The Sound Reduction Index was measured by the Average Sound Pressure Level between two rooms have installed five microphones in each rooms. Then, the Sound Reduction Index was calculated by differential Average Sound Pressure Level in sound resource room and sound receiving room.

<table>
<thead>
<tr>
<th>Equipment list</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment</td>
</tr>
<tr>
<td>Signal analyzer</td>
</tr>
<tr>
<td>Microphone</td>
</tr>
<tr>
<td>Microphone</td>
</tr>
<tr>
<td>Omnidirectional sound source</td>
</tr>
<tr>
<td>Amplifier</td>
</tr>
</tbody>
</table>

Also, difference value of measured the Average Sound Pressure Level in sound resource room and sound receiving room was revised to sound absorbing power in sound receiving room and then calculated the Sound Reduction Index.

$$R = L_1 - L_2 + 10 \log \frac{S}{A}$$

Where  
$L_1$: Average sound pressure level in the sound resource room (dB)  
$L_2$: Average sound pressure level in the sound receiving room (dB)

$S$ : Size of specimen($m^2$)  
$A$: Sound absorbing power in the sound receiving room($m^2$)

The value of the Sound Reduction Index comply with ISO 140-3 shall be compared with the value of ISO 717-1 of 100–3,150 Hz in frequency band.  
This study was comparison with sound insulation performance of wall through the $\text{Rw} + \text{C}$ value which was assessment index. The single-number quantity for airborne sound insulation rating ($\text{Rw}$) is value, in decibels, of the reference curve at 500 Hz after shifting it in accordance with the method specified ISO 717. And the spectrum adaptation is value, in decibels, to be added to the single-number rating ($\text{Rw}$) to take account of the characteristics of particular sound spectra. To evaluate the results of a measurement made in accordance with ISO 140-3 in one-third-octave bands, given to 0.1dB, shift the relevant reference curve in steps of 1dB towards the measured curve until the sum of unfavourable deviations is as large as possible but not more than 32dB.

3.2 SPECIMENS

Specimens as in Table 4 were made to measure sound insulation performance of lightweight wall including a middle lintel. The factors of sound insulation performance were existence of resilient channel and kinds of boards among factors of K.W. Kim’s study.  
Glue-laminated Timber of Douglas fir from North America was used for the middle lintel of 150x150mm (specimen 2 : 160x150mm) cross section and 38x89mm stud. Glass wool was inserted to improve sound insulation performance.

Specimen 1 was finished with 2 layers of gypsum boards. And, specimen 2 was finished 2 layers of gypsum board with resilient channel, and the interior and exterior of specimen 3 were finished with a layer of gypsum board and a layer of Loess board, respectively. And specimen 4 was done with 2 layers of Loess board.  
The view of specimens installed in the laboratory is shown in Figure 1. Tests were performed following ISO 140-3 Acoustics - Measurement of sound insulation in building and of building elements – parts 3 : Laboratory

Figure 3: Plan of acoustic sound test room
measurement of airborne sound insulation of building elements.

**Table 4: Specimens**

<table>
<thead>
<tr>
<th>Type</th>
<th>Figure</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>GB 2ply + GW + 38x89 Wood stud + GB 2ply</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>GB 2ply + GW + 38x89 Wood stud + Channel + GB 2ply</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>LB 1ply + GB 1ply + GW + 38x89 Wood stud + GB 1ply + LB 1ply</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>LB 2ply + GW + 38x89 Wood stud + LB + LB 2ply</td>
</tr>
</tbody>
</table>

GB: THK12.5 Gypsum Board  
GW: THK 89, Density 9kg/cm³ Glass wool  
LB: THK 12 Loess Board  
Channel: Resilient channel

**Figure 4:** details and section of specimen (with gypsum boards and resilient channels)

**Figure 5:** Installation of specimen in Laboratory (with Loess board)

### 3.3 TEST RESULTS

The results of test type 2 with resilient channel was better than type 1, wall with Loess boards also was improve the sound insulation almost frequency bands. The sound insulation of gypsum boards in high frequency band decreased, but loess boards were increased in value.

Also, when it makes a comparison type 2 with resilient channel and type 3 of combination gypsum and loess boards, type 3 is better. So, loess boards are more efficient than resilient channel. Especially, in high frequency band was superior to sound insulation of Loess boards.

**Table 5: Results of sound insulation**

<table>
<thead>
<tr>
<th>Hz</th>
<th>TYPE1</th>
<th>TYPE2</th>
<th>TYPE3</th>
<th>TYPE4</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>29.9</td>
<td>31.1</td>
<td>40.3</td>
<td>44.0</td>
</tr>
<tr>
<td>125</td>
<td>36.4</td>
<td>36.5</td>
<td>40.6</td>
<td>37.6</td>
</tr>
<tr>
<td>160</td>
<td>35.0</td>
<td>35.4</td>
<td>41.1</td>
<td>39.7</td>
</tr>
<tr>
<td>200</td>
<td>37.4</td>
<td>41.8</td>
<td>45.5</td>
<td>45.0</td>
</tr>
<tr>
<td>250</td>
<td>40.3</td>
<td>40.4</td>
<td>43.7</td>
<td>50.6</td>
</tr>
<tr>
<td>315</td>
<td>39.1</td>
<td>43.9</td>
<td>46.0</td>
<td>53.4</td>
</tr>
<tr>
<td>400</td>
<td>40.9</td>
<td>45.4</td>
<td>50.1</td>
<td>55.8</td>
</tr>
<tr>
<td>500</td>
<td>47.0</td>
<td>50.3</td>
<td>51.4</td>
<td>59.4</td>
</tr>
<tr>
<td>630</td>
<td>45.7</td>
<td>51.5</td>
<td>53.3</td>
<td>60.6</td>
</tr>
<tr>
<td>800</td>
<td>48.5</td>
<td>53.5</td>
<td>54.6</td>
<td>61.1</td>
</tr>
<tr>
<td>1000</td>
<td>50.4</td>
<td>56.5</td>
<td>57.5</td>
<td>62.8</td>
</tr>
<tr>
<td>1250</td>
<td>52.1</td>
<td>58.4</td>
<td>60.8</td>
<td>64.8</td>
</tr>
<tr>
<td>1600</td>
<td>53.3</td>
<td>60.8</td>
<td>65.2</td>
<td>67.7</td>
</tr>
<tr>
<td>2000</td>
<td>49.7</td>
<td>60.1</td>
<td>65.8</td>
<td>67.5</td>
</tr>
<tr>
<td>2500</td>
<td>44.4</td>
<td>53.7</td>
<td>65.0</td>
<td>67.5</td>
</tr>
<tr>
<td>3150</td>
<td>46.5</td>
<td>51.5</td>
<td>63.7</td>
<td>66.4</td>
</tr>
<tr>
<td>4000</td>
<td>51.7</td>
<td>56.4</td>
<td>65.2</td>
<td>66.7</td>
</tr>
<tr>
<td>5000</td>
<td>56.1</td>
<td>60.7</td>
<td>66.0</td>
<td>68.1</td>
</tr>
</tbody>
</table>
**Table 6 : Results of grade**

<table>
<thead>
<tr>
<th>Type</th>
<th>Rw+C</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>46</td>
<td>Out of grade</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>54</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>57</td>
<td>2</td>
</tr>
</tbody>
</table>

**4 CONCLUSION**

The measurement results of the sound insulation performance showed that it improved in the order of Specimen 1<2<3<4. Specimen 1 was out of sound insulation Grade, and Specimen 2 with resilient bar was determined to be Grade 3, and Specimen 3 and 4 was done as Grade 2. Also, the comparison results of Specimen 3 and 4 found that the sound insulation Grade of Specimen 4 is better, and Loess board without resilient bar has better improvement of the sound insulation compared to in the case of Gypsum board. Also, The sound insulation of Loess boards was excellent than gypsum boards in high frequency band.

**REFERENCES**


POSSIBIRITY OF MID-RISE TIMBER STRUCTURE APPROCH BY 1/6 SCALE MODEL EXPERIENCE

Hisamitsu Kajikawa¹, Yoko Miyamoto², Hiroyuki Noguchi³

ABSTRACT: Should eccentricity not be designed in timber structures? Design theory of steel and RC structures are reducing eccentricity and designing to base shear. However, if timber structures have eccentricity, it might become high capacity of energy absorption and toughness. This paper shows outline, result and consideration of 1/6 scale model shaking table experiment. Parameter is modulus of eccentricity, horizontal diaphragm stiffness and collapse story in 2, 3, 5 and 7-story test body. Focusing attention on process of breaking especially, ‘possibility of mid-rise timber structure’ becomes a consideration from this experiment.

KEYWORDS: Mid-rise timber structure, Eccentricity, Breaking process, Seismic peak response, Shaking table experiment

1 INTRODUCTION

Such as 9-story in London, 7-story in Berlin and 8-story in Sweden⁴, mid-rise timber structures have been built in each country. Also, Japanese and Italian research institutes have done the shaking table experiment of full-scale 7-story XLam building⁵ and Japanese and American research institutes have done full-scale 7-story light-frame wooden building⁶. For future many mid-rise timber structures might be built.

But the rule of design of mid-rise timber structures imitates steel and RC structures. And the foreign rule consults Japanese.

In Japanese rule, it is necessary to reduce modulus of eccentricity as low as possible. When RC columns are broken by shear force, vertical bearing capacity disappears. Deformation has to be small because of it. So reducing eccentricity and designing to base shear is rational.

But if deformation is some large from eccentricity, vertical bearing capacity dose not disappear in timber structure. Walls are deformed not only in the direction of excitation but also in the orthogonal because of eccentricity. Walls are deformed in order such as weak walls in collapse story, followed weak wall in the other story, strong wall and orthogonal wall. Force (broken) relay is continuing. The whole test body is deformed. It might become high capacity of energy absorption and toughness. If design about eccentricity can be under control, it is more effective than non-eccentricity.

This paper shows outline, result and consideration of 1/6 scale model shaking table experiment. Parameter is modulus of eccentricity, horizontal diaphragm stiffness and collapse story in 2, 3, 5 and 7-story test body. Input wave is shortened 1/√6 times original time axis. Measuring instrument is laser displacement meter and accelerometer. Consideration is about deformation diagram, seismic peak response displacement and its amplification and process of breaking. Focusing attention on process of breaking especially, ‘possibility of mid-rise timber structure’ becomes a consideration from this experiment.

1 Hisamitsu Kajikawa, Misawa Homes Institute of Research and Development Co., Ltd., Dr.Eng. Email: hisamitsu_kajikawa@home.misawa.co.jp Tel:03-3247-5643
2 Yoko Miyamoto. Graduate Student, Graduate School of Science Technology, Meiji Univ. Email: miyamotoyoko1988@gmail.com
3 Hiroyuki Noguchi Prof., Dept. of Architecture School of Science Technology, Meiji Univ., Dr.Eng.
2.1 METHOD OF EXPERIMENT

Figure 2 shows method and outline of experiment.

2.2 TEST BODY

The skeleton of test body is formed in the columns and beams made of aluminum. Each member such as walls, floors, measuring instrument and weight screw in the beams these are drilled the taps of M4 and M8 screw. Columns and beams are connected by pin joint on contact with ball joint and steel of form of cylinder. So, axial force works frame. Only shear force works wall diaphragms and horizontal diaphragms. Test body is set on shaking table on contact with H steel. Table 1 shows all of test body name, weight and number of walls.

2.2.1 Wall Diaphragm

Figure 3 shows detail of wall diaphragm and Figure 4 shows detail of timber wall. Hysteresis characteristics of bearing wall consist of many elements such as shear deformation, bending deflection, compressive strain and deformation of joints. But it is very difficult to reproduce in these 1/6 scale model and to have width (to adjust) in number of walls. Therefore, we try to imitate hysteresis characteristics of bearing wall (such as skeleton curve, yield point and slip behaviour) by only shear deformation of nails. Wall diaphragm is set walls which can be assumed rigid made of steel (rigid wall, Fig.3) between beam and timber wall. And width of timber wall is given as long as possible compared to height of it. It can be horizontal deformation of nails longer than vertical deformation. Also, driving nails in plane board both up and down side, it can be horizontal deformation of walls twice as long as nails. All of them can be possible to adjust stiffness and bearing force of walls by number of nails. Those can be possible to make hysteresis characteristics which are similar to actual bearing walls.

Figure 2: 7-story 1/6 Scale Model Test Body

A piece of walls is driven 4 nails each up and down side. The types of walls are 7 types. Figure 4 shows detail and layout of nails of two pieces of walls. Figure 6 shows load-displacement relation of two pieces of walls which is taken by load-displacement relation of shear test of nails.
2.2.2 Horizontal Diaphragm

Figure 5 shows details of horizontal diaphragm. The rigid floor is horizontal angle board made of aluminium fixed in 4 corners. The soft floor is wooden frame which plane board placed. Figure 6 shows load-displacement relation of soft floor.

2.3 TEST BODY NAME AND PARAMETER

2.3.1 Input Seismic Wave

Table 1 shows the seismic peak response relative story displacement and deformation diagram when relative story displacement in collapse story is peak. And it shows about all test bodies (all 17 bodies) every input wave. Also, each value in the center of gravity of X and Y is the average of 1 street and 2 street. Relative story displacement collapse story assumed can be deformed the most of all story. Process of breaking shows consideration of details of it.

Also, the following measuring or excitation we done in order to measure first dominant frequency before and after excitation of seismic wave. First is microtremor with shaking table fixed. Second is 5% BCJ-L2 regulated wave (input wave for design, Max: 18[gal]) to each Y and X direction. Third is sweep-sine (30[gal]) to Y direction.

2.5 METHOD OF MEASURING

Figure 3 shows the layout of measuring instrument. Accelerometers measure absolute acceleration on the center of shaking table and on each street and story. Laser displacement meters measure absolute displacement in base of test body and relative story displacement in each street and story. Method of measuring is that laser beam passed through upper and under horizontal members of timber wall.

3 RESULTOF EXPERIMENT AND CONSIDERATION

Table 1 shows the seismic peak response relative story displacement and deformation diagram when relative story displacement in collapse story is peak. And it shows about all test bodies (all 17 bodies) every input wave. Also, each value in the center of gravity of X and Y is the average of 1 street and 2 street. Relative story displacement collapse story assumed can be deformed the most of all story. Process of breaking shows consideration of details of it.

The following shows consideration about deformation diagram. Type1 is only horizontal deformation. Type2 is rotational deformation in addition to it. Type3 is shear deformation in addition to them. Also, every test body shows just about first mode in both elastic and plastic region.

The following shows consideration about method of 1/6 scale model shaking table experiment. This experiment was done under the variety of assumptions such as law of similarity, seismic wave shortened its time axis and imitated hysteretic characteristics. This experiment can capture the trend of the seismic peak response and vibration characteristics. This method can be possible to capture perspective of three-dimensional behaviour and to verify a lot of parameter.
### Table 1: Detail of Test Body and Result of Experiment

<table>
<thead>
<tr>
<th>Floor</th>
<th>Two-story</th>
<th>Three-story</th>
<th>Seven-story</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>weight (each story 230kg - total 460kg)</td>
<td>weight (each story 108kg - total 360kg)</td>
<td>weight (each story 108kg - total 540kg)</td>
</tr>
<tr>
<td></td>
<td>Ma 2-A-R000-G1</td>
<td>Ma 3-A-R000-G1</td>
<td>Ma 7-B-R000-G1</td>
</tr>
<tr>
<td></td>
<td>Ma 2-T-R000-G1</td>
<td>Ma 3-T-R000-G1</td>
<td>Ma 7-B-R022-G1</td>
</tr>
<tr>
<td></td>
<td>M 2-C-R000-G1</td>
<td>M 3-C-R000-G1</td>
<td>M 5-C-R000-G1</td>
</tr>
<tr>
<td></td>
<td>M 2-B-R000-G1</td>
<td>M 3-B-R000-G1</td>
<td>M 5-B-R000-G1</td>
</tr>
<tr>
<td></td>
<td>M 2-B-R038-G1</td>
<td>M 3-B-R038-G1</td>
<td>M 5-B-R038-G1</td>
</tr>
<tr>
<td></td>
<td>M 2-B-R038-G2</td>
<td>M 3-B-R038-G2</td>
<td>M 5-B-R038-G2</td>
</tr>
</tbody>
</table>

**Results:**

- **Two-story:**
  - Weight: 230kg (each story) - Total: 460kg
  - Collapsed on 5th story

- **Three-story:**
  - Weight: 108kg (each story) - Total: 360kg
  - Collapsed on 3rd story

- **Seven-story:**
  - Weight: 108kg (each story) - Total: 540kg
  - Measuring until 5th story

**Graphs and Diagrams:**

- Graphs showing input wave levels (10%, 50%, 60%, 40%, 20%) and relative story displacement (0 to 100mm) for each floor.
- Diagrams illustrating wave levels and center of gravity (strong and weak walls).

**Key Points:**

- **Wave Levels:**
  - Wave level 10%
  - Wave level 50%
  - Wave level 60%
  - Wave level 40%
  - Wave level 20%

- **Center of Gravity:**
  - Strong wall
  - Weak wall
Table 2: Seismic Peak Response Displacement and its Amplification

<table>
<thead>
<tr>
<th>Wave Level [%]</th>
<th>X1 (Strong Wall)</th>
<th>Xg (Center of Gravity)</th>
<th>X2 (Weak Wall)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3: Process of Breaking

<table>
<thead>
<tr>
<th>Type</th>
<th>Two-story test body</th>
<th>Three-story test body</th>
<th>Five-story test body</th>
<th>Seven-story test body</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/30 rad.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/80 rad.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/500 rad.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4 AMPLIFICATION FACTOR OF SEISMIC PEAK RESPONSE DISPLACEMENT

Table 2 shows seismic peak response relative story displacement and amplification factor of type 1, type 2 and type 3. The following shows consideration about seismic peak response displacement. That in the center of gravity is similar to each 3 types until about 1/500[rad.]. That in the weak walls of type 3 is largest, followed type 2 and type 1. In contrast, that in the strong walls of type 1 is the largest, followed type 2 and type 3. And dominated frequency is similar to each 3 types. But that of type 2 and type 3 in the center of gravity is usually larger than type 1 from 1/500[rad.] to 1/80[rad.] because of plastic deformation in the weak walls. That is much larger over 1/80[rad.].

The following shows consideration about amplification factor of seismic peak response displacement. In the case of input acceleration is 80%, that of type 2 in the center of gravity is 0.80-2.43 times as much as type 1, in the weak walls 1.05 -2.89 times and in the strong walls 0.58-2.00 times. In the same case, type 3 in the center of gravity is 0.89-2.66 times as much as type 1, in the weak walls 1.27 -3.80 times and in the strong walls 0.55-1.80 times. Amplification of type 3 is more than type 2. Also, in the case of 100% amplification factor of type 2 of 2-story test body in the center of gravity is 5.63 times, in the weak walls 7.69 times and in the strong walls 3.91 times. In the same case, type 3 in the center of gravity is 6.58 times, in the weak walls 10.34 times and in the strong walls 3.79 times. Amplification in the case of 100% is more than 80%. Incidentally, deformation of type 2 and type 3 is large plastic and type 1 is small (about 2[mm]) in 2-story test body. So amplification factor becomes very much.

5 PROCESS OF BREAKING

Table 3 shows relative story displacement. Deformation of collapsed on bottom-story, mid-story and top-story of type 1 is large sectionally in the direction of excitation but few in orthogonal direction. Also, deformation of collapsed on all story of type 1 becomes larger at the bottom-story in the beginning. In the end it becomes the largest at the top-story. Every story is deformed largely. But it is few in the orthogonal direction. Compared to type 1, deformation of type 2 and type 3 is large not only in the direction of excitation but also in the orthogonal direction. And deformation of type 3 is larger than type 2. In type 3 of 5-story test body each wall is broken gradually for example weak wall, weak wall in the other story, strong wall and the orthogonal wall in that order. And force (broken) relay is continuing. The whole test body is deformed. The walls in the direction of excitation of type 1 resisting force, deformation of the orthogonal direction wall is few. So it is easy to collapse story. The walls in the both direction and the orthogonal direction of excitation of type 2 and type 3 resist force. And if weak walls are collapsed, strong walls aren’t collapsed. So it isn’t easy to collapse story.

6 ADD-UP

The following shows add-up about results of 1/6 scale model shaking table experiment.

I. The seismic peak response deformation diagram is that non-eccentricity and rigid floor (type 1) is only horizontal deformation. Eccentricity and rigid floor (type 2) is rotational deformation in addition to it. Eccentricity and soft floor (type 3) is shear deformation in addition to them.

II. The seismic peak response displacement and its amplification factor is that type 2 is up to 5.63 times in the center of gravity and up to 7.68 times in the weak walls as much as type 1. Type 3 is up to 6.58 times in the center of gravity and up to 10.34 times in the weak walls as much as type 1.

III. The Process of breaking is that type 1 is deformed in only collapse story of the direction walls of excitation. In contrast, type 2 and type 3 is deformed in both the direction and the orthogonal direction of excitation. In addition, type 3 is deformed in horizontal diaphragm. Deformation is usually some large from eccentricity. But orthogonal walls resist force. If design of eccentricity can be under control, whole test body might be deformed. And, it might become high capacity of energy absorption and toughness.

ACKNOWLEDGEMENT

Takatori ns wave (Fig.10) is seismic wave in JR Takatori station. It was contribute by Railway Technical Research Institute.

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CONSTRUCTION METHODS AND EARTHQUAKE ENCOUNTER RECORDS OF JAPANESE TRADITIONAL TIMBER THREE STOREY PAGODAS

Chee Siang Tan¹, Kaori Fujita²

ABSTRACT: A study to assess the seismic resistance of traditional timber three storey pagodas in Japan, which is commonly said to be inferior to that of the five storey pagodas. First, the earthquake encounter of each pagoda and possible damage caused are determined. Then, the construction methods of the pagodas and the way they correspond to the aforementioned records are analysed. Lastly, the results are compared to that of the five storey pagodas. It is true to say that three storey pagodas too exhibit exceptional earthquake resistance, despite there being several collapse records.

KEYWORDS: Three Storey Pagodas, Construction Methods, Earthquake Encounter Records, Seismic Resistance

1 INTRODUCTION

Five storey pagodas in Japan are said to possess exceptional earthquake resistant properties [1]. Despite having experienced multiple earthquakes of Japan Meteorological Agency (JMA) intensity scale 6 and above, no collapse were recorded apart from some minor damages. There is still no satisfactory scientific explanation for the above phenomenon, even though studies and experiments have been conducted since Meiji era [1]. On the other hand, few studies were related to the three storey pagodas, possibly owing to the existence of collapse records due to earthquake amongst the relatively abundant numbers of three storey pagodas in Japan. The authors believe that it is important to also extend the research to three storey pagodas which share the same fundamental structural properties, starting by assessing their earthquake resistance and studying their construction methods. It is hoped that the comparison studies between the five and three storey pagodas would provide some hints on the supposedly superior earthquake resistance of the former to that of the latter.

1.1 AIM

The aim is to collect and study the data of the construction method and earthquake encounter record of the traditional timber three storey pagodas in Japan, in order to assess their earthquake resistance compared to their five storey counterparts.

1.2 RESEARCH METHOD

Three storey pagodas designated as national treasures and important cultural properties by the government, 57 of them in total, all dated back from or before Edo Period are selected. Earthquakes with JMA intensity scale 5 and above that occurred around each pagoda, and had caused severe damage and numerous casualties were extracted from reference 3. These were summarised into a chronological table together with seismic damage recorded and repairs executed, based on official reports and documents (references 6 and 8). Construction method and details of each pagoda were then analysed together with the aforementioned records and compared to that of the five storey pagodas. Similar study [1] on five storey pagodas had been conducted by M. Oyama and K. Fujita in the year 2002, and will serve as the main reference and comparison material for this study.

2 EARTHQUAKE RECORDS

2.1 EARTHQUAKE ENCOUNTER RECORD

The chronological table (figure 8, 9) shows the earthquakes encountered by each pagoda since their respective year of completion, together with damage and repair records. At least 80% of the three storey pagodas had encountered earthquake with JMA intensity 6 and above, with 5 encounters as the maximum. In average, the earthquake encounter is 2.54 times per pagoda, with 145 earthquake encounters in total. All three storey pagodas have encountered earthquake with JMA intensity 5 and above, with Hoki-ji, the oldest amongst

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all three storey pagodas, encountering as many as 32 times. In average, the earthquake encounter is 11.68 times per pagoda, with 666 earthquake encounters in total. The encounter frequency is once every 226 years for intensity scale 6 and above, and once every 60 years for intensity scale 5 and above. It should be noted that the list is not exhaustive and precise for earthquakes occurred prior to modern recording system, thus it is possible that the above figures should register higher frequency. Earthquakes without ascertained intensity, especially those dated back before 1500s, but had caused vast damage were considered as intensity 5 and above.

2.2 SEISMIC DAMAGE

Table 1: Damage recorded in three storey pagodas

<table>
<thead>
<tr>
<th>Name</th>
<th>Damage Count</th>
<th>Damage Type</th>
<th>Construction Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oppo-ji</td>
<td>1</td>
<td>Finial</td>
<td>Stack-up</td>
</tr>
<tr>
<td>Hiyoshi Shrine</td>
<td>1</td>
<td>Finial</td>
<td>Stack-up</td>
</tr>
<tr>
<td>Sanmyo-ji</td>
<td>1</td>
<td>Finial</td>
<td>Stack-up</td>
</tr>
<tr>
<td>Jimoku-ji</td>
<td>1</td>
<td>Collapse</td>
<td>Stack-up</td>
</tr>
<tr>
<td>Hoki-ji</td>
<td>1</td>
<td>Finial</td>
<td>Stack-up</td>
</tr>
<tr>
<td>Yakushi-ji</td>
<td>4</td>
<td>Inclination</td>
<td>Stack-up</td>
</tr>
<tr>
<td>East Pagoda</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Henjo-in</td>
<td>1</td>
<td>Inclination</td>
<td>Long Column</td>
</tr>
</tbody>
</table>

Table 1 shows the seismic damage cited from reference 6 and 8. Out of the 57 pagodas studied, only 7 pagodas were recorded to have been damaged by earthquake, with 10 cases in total. The damage pattern mostly involves the breakage of metallic finials (sōrin in Japanese), and in specific cases, the collapse (Jimoku-ji) and inclination (Yakushi-ji East Pagoda) of the pagoda itself. The metallic finial connected to the central pillar is often damaged at the joint as a result of the collision between the roof and the central pillar during earthquakes. The east pagoda of Yakushi-ji has a unique feature i.e. the decorative pent roofs (mokoshi in Japanese) attached on each storey. As a result, the height itself. The metallic finial connected to the central pillar is

2.3 REPAIRS

Amongst the 36 pagodas with available official repair documents and reports, there were 253 repairs recorded in total, with average 7 repairs per pagoda and frequency of once every 96 years. However, as can be seen from the chronological table, the repairs rarely correspond to the earthquake encounters. Decay and re-roofing are commonly cited as reason for the repairs. Most pagodas were dismantled and repaired after the Act on Protection of Cultural Properties took effect in 1950.

3 CONSTRUCTION METHODS

3.1 CENTRAL PILLAR (SHIN-BASHIRA)

All pagodas in Japan are built with a unique long column that passes through the centre of the pagoda, supporting the metallic finial that extends high above the roof. The central pillar, also known as ‘shin-bashira’ in Japanese, is structurally separated from the rest of the pagoda, thus it does not carry any load of the pagoda save for the metallic finial. The 4 earliest three storey pagodas, all dated back before the 9th century have their central pillars built on the ground level, on top of a base stone. Pagodas after that however have their central pillars built above the ceiling of the first storey. Central pillars have long been hypothesised as the main quake resistant component of pagodas, serving as some sort of latch bolt and damper [4]. Through experiments, it has been proven that central pillar oscillates with a phase different to that of the rest of the pagoda, and as a result providing some sort of damping effect during earthquakes [5]. Judging by the fact that pagodas of both types suffered similar seismic damage, there is no clear difference between the two types of central pillar in terms of seismic resistance.

Table 2: Seismic damage and central pillars

<table>
<thead>
<tr>
<th>Name</th>
<th>Damage Type</th>
<th>Central Pillar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oppo-ji</td>
<td>Finial</td>
<td>First Storey</td>
</tr>
<tr>
<td>Hiyoshi Shrine</td>
<td>Finial</td>
<td>First Storey</td>
</tr>
<tr>
<td>Sanmyo-ji</td>
<td>Finial</td>
<td>First Storey</td>
</tr>
<tr>
<td>Jimoku-ji</td>
<td>Collapse</td>
<td>First Storey</td>
</tr>
<tr>
<td>Hoki-ji</td>
<td>Finial</td>
<td>Ground</td>
</tr>
<tr>
<td>Yakushi-ji East Pagoda</td>
<td>Finial, inclination</td>
<td>Ground</td>
</tr>
</tbody>
</table>

3.2 CONSTRUCTION METHODS

The main components of a typical pagoda framework include the support columns (12 outer columns and 4 inner columns surrounding the central pillar), bracket complex and rafters. Each storey has its own individual columns that are not connected to their corresponding pillars below and above. Construction method of pagodas could be categorised into 3: stack-up method, long column method, and yagura method. Each method...
could be easily distinguished by observing the position of the support columns on the upper storeys. Stack-up method is the oldest and most common construction method. Pagodas built using this method have their columns placed on top of the rafters, in other words, on top of the roof of the storey below. Long column method pagodas have their columns built directly on top of the beam-like bracket complexes, called tsunagi-hijiki in Japanese. As the result, the columns are longer compared to that of the stack-up method, thus the name long column method. In yagura method pagodas, bracket complexes are replaced by beams supporting the columns. There are also combinations of 2 or more methods, usually with the outer columns built using the stack-up method, while the inner columns built using the long column or yagura method. 35 out of the 57 pagodas are built using stack-up method, 9 are of the long column method, 9 being the combination of stack-up and either long column or yagura method, and 1 of yagura method. The rest remains unknown. Long column method was invented later than stack-up method, with the first known case being from Muromachi Period. Long column and yagura method have the advantage of shortening the construction period by discarding the need to consider the balance between the weight of the roof and the upper storeys, as well as the need to complete the roof before continuing to the construction of the upper storeys [2]. Interestingly, all three storey pagodas that suffered seismic damage are of stack-up method.

3.4 INNER COLUMNS ON UPPER STOREYS

There are pagodas that have all their inner columns omitted on all storeys, while some have their shiten-bashira omitted only on the lowest storey but retained on the upper storeys and vice versa. 19 out of the 57 pagodas have shiten-bashira on their 2 upper storeys omitted. Although the omission of inner columns gives an impression of a less stable structure, all 6 pagodas with seismic damage record carry shiten-bashira on their upper storeys. Further study is needed to determine the role of inner columns in preventing seismic damage.

3.5 SIZE AND ROOFING MATERIAL

The total height, length of metallic finial, total area, length of eaves, and slenderness ratio of support columns of each pagoda were analysed. The graphs below show that the 4 oldest pagodas are comparatively larger in scale. The tallest of all, the east pagoda of Yakushi-ji, has slender support columns notwithstanding its gigantic scale of 34m. All pagodas with seismic damage record except Sanmyō-ji generally have taller than average height, longer finial and eaves. On the other hand, pagodas that adopted new construction methods such as the long column method and the omission of inner columns in general are of smaller scale and have shorter eaves. 29 out of the 57 pagodas have tiled roof. Including five storey pagodas, tiled roof pagodas are more susceptible to seismic damage. Those with long column and yagura construction method, as well as those without inner columns are more inclined to having lighter roof such as copper plate and thatched roof.

![Figure 2: Typical plan of a pagoda](image)

![Figure 3: From left: stack-up, long column and yagura](image)

![Figure 4: Total height of three storey pagodas](image)
4 MAIN DIFFERENCE BETWEEN THREE AND FIVE STOREY PAGODA

Central pillar that reaches ground level is much more common in five storey pagodas regardless of construction year. Also, inner support column is never omitted for five storey pagodas. Therefore it is said that the construction and design of five storey pagodas are more prudent and conservative, while three storey pagodas are inclined to being experimented with new construction methods \[2\]. Similar to the three storey pagodas, there are no seismic damage records for long column and yagura method five storey pagodas.

5 CONCLUSIONS

The 57 pagodas studied showed very few cases of seismic damage despite experiencing multiple times of huge earthquake, in fact, fewer in terms of proportion compared to that of the five storey pagodas (5 pagodas, 7 cases out of 22 pagodas \[3\]). This proves their exceptional earthquake resistance. This is especially true in cases of long column and yagura construction method without any seismic damage recorded. On the other hand, large scale pagodas with heavy roof i.e. tiled roof are apparently more susceptible to seismic damage. Three storey pagodas have been subjected to various new construction methods and designs. In such cases, they are usually reduced in scale and weight, possibly to ensure stability. It is not completely true to say that five storey pagodas possess superior earthquake resistance just because there exist several cases of three storey pagodas collapsing due to earthquake. It can only be deduced that there exists a certain level of variants when it comes to the earthquake resistance of three storey pagodas. Further studies should be done to study the pagodas that collapsed during earthquake.

ACKNOWLEDGEMENT

The assistance provided by the members of Fujita Lab. and the university library is greatly appreciated.

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[10] Homepage of Each Temple (Shrine).
Figure 8: Earthquake encounter record chronological table part 1
Figure 9: Earthquake encounter record chronological table part 2
INFLUENCE OF SHAPE FACTORS OF WOOD SCREW ON WITHDRAWAL PERFORMANCE
–Development of wood screw used for soft and light wood–

Hei-Soo Baek¹, Hideki Morita¹, Atsushi Shiiba¹, Yutaka Iimura¹, Fujio Imai²

ABSTRACT: Wood screw has been used for not only non-structural use but also structural one for its good workability and easy dismantlement. As the current wood screw is designed for using in wood with a comparatively high specific gravity, using for soft and light wood such as sugi, *Cryptomeria japonica* D. Don, makes the withdrawal resistance far low. The objective of this study was to develop a new wood screw that is appropriate for soft and light wood. An experimental investigation was conducted to clarify the influence of shape factors of wood screw on withdrawal resistance. Results and discussion are provided.

KEYWORDS: soft and light wood, wood screw, shape factor, withdrawal resistance

1 INTRODUCTION

As for the wood screw, the expansion of use is expected from its good workability and easy dismantlement in the fasteners for not only non-structural use but also structural one. However, the withdrawal resistance decreases according to the difference in specific gravity when used for sugi, *Cryptomeria japonica* D. Don, characterized by soft and light wood, because the current wood screw for structural use is designed for using in wood with a comparatively high specific gravity. From this background, an experimental investigation concerning the influence of shape factors of wood screw on withdrawal resistance was conducted to develop a new wood screw that is appropriate for soft and light wood.

2 EXPERIMENTAL

In this paper, shape factors of wood screw (figure 1) were roughly classified into two groups. We examined the withdrawal resistance of wood screw while controlling for (1) shape factors other than complete thread, such as unthreaded shank length and incomplete thread end shape, and (2) shape factors of complete thread, such as thread pitch, thread major diameter and thread minor diameter.

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Figure 1: Shape factors of wood screw

2.1 INVESTIGATION CONCERNING SHAPE FACTORS OTHER THAN COMPLETE THREAD

2.1.1 MATERIALS AND METHODS

The wood species used in the experiment is sugi grown in Miyazaki Prefecture, Japan. This is air dried wood (average moisture content is 12.6%) and average specific gravity is 0.417. The wood screw was driven into the square timber of 1000mm length × 125mm width × 125mm thickness in direction perpendicular to grain using an electric impact driver. The specifications of three types of wood screws used in the experiment are as shown in table 1. Figure 2 shows the schematic diagrams of six types of withdrawal specimens. The number of specimens was ten per type.

The withdrawal specimen was placed on the table of the universal testing machine and the head of wood screw was slid into the slot of screw fixture. The static loading was
performed at cross-head speed of 1mm per minute until the withdrawal resistance decreased to less than 80% of the peak load. The displacement of the cross-head was measured and was assumed to represent the withdrawal displacement of wood screw.

![Figure 2: Schematic diagrams of six types of withdrawal specimens for the investigation concerning shape factors other than complete thread](image)

### Figure 2: Schematic diagrams of six types of withdrawal specimens for the investigation concerning shape factors other than complete thread

2.1.2 RESULTS AND DISCUSSION

The wood screw withdrawal test results, which controlled for shape factors other than complete thread such as unthreaded shank length and incomplete thread end shape, are shown in figure 3 and figure 4 and table 2.

The average of the peak withdrawal resistances of specimen A60(15) was over 1.70kN (26%) in comparison to that of specimen A60(0). Similarly, the averages of the peak withdrawal resistances of specimen B75(15) and specimen B75(30) were over from 1.87kN (22%) to 2.01kN (23%) in comparison to that of specimen B75(0). From both results, the effect of unthreaded shank portion on the withdrawal resistance contribution was recognized. Besides, in above results, the significant difference was admitted in statistics, for which the level of significance was 0.01. On the other hand, there was little difference in the average of the peak withdrawal resistances of specimen B75(15) and specimen B75(30). Therefore, it is supposed that the withdrawal resistance contribution of more than a certain value cannot be expected even if the penetration depth of unthreaded shank portion increases.

There was no statistically significant difference at a significance level of 0.05 between the average of the peak withdrawal resistances of specimen A60(0) and specimen C60(0). As this result, it is considered that the influence of incomplete thread end shape (drill point and cut point) on the withdrawal resistance is small.

### Table 1: Specifications of three types of wood screws for the investigation concerning shape factors other than complete thread

<table>
<thead>
<tr>
<th>Type of wood screw</th>
<th>Name of wood screw (Ready made or Special order)</th>
<th>A (Ready mode)</th>
<th>B (Ready mode)</th>
<th>C (Ready mode)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>D (mm)</td>
<td>6.25mm</td>
<td>6.25mm</td>
<td>6.25mm</td>
</tr>
<tr>
<td>L (mm)</td>
<td></td>
<td>90mm</td>
<td>120mm</td>
<td>75mm</td>
</tr>
<tr>
<td>Material</td>
<td>M</td>
<td>SWRCH22A</td>
<td>SWRCH22A</td>
<td>SWRCH22A</td>
</tr>
<tr>
<td>Manufacture method</td>
<td>M</td>
<td>Rolling method</td>
<td>Rolling method</td>
<td>Rolling method</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>ST</td>
<td>Dural coat</td>
<td>Dural coat</td>
<td>Dural coat</td>
</tr>
<tr>
<td>Head diameter Dₜ</td>
<td></td>
<td>12.30mm</td>
<td>12.30mm</td>
<td>9.80mm</td>
</tr>
<tr>
<td>Height Lₜ</td>
<td></td>
<td>6.00mm</td>
<td>6.00mm</td>
<td>4.00mm</td>
</tr>
<tr>
<td>Head shape Sₜ</td>
<td></td>
<td>Hexagon head</td>
<td>Hexagon head</td>
<td>Pan head</td>
</tr>
<tr>
<td>Screw drive type</td>
<td></td>
<td>Diff socket</td>
<td>Square socket</td>
<td>Square socket</td>
</tr>
<tr>
<td>Unthreaded shank diameter Dₜ</td>
<td></td>
<td>5.90mm</td>
<td>5.90mm</td>
<td>5.90mm</td>
</tr>
<tr>
<td>(Ratio of Dₜ to Dₜ )</td>
<td></td>
<td>(0.94)</td>
<td>(0.94)</td>
<td>(0.94)</td>
</tr>
<tr>
<td>Unthreaded shank length Lₜ</td>
<td></td>
<td>30mm</td>
<td>45mm</td>
<td>15mm</td>
</tr>
<tr>
<td>(Ratio of Lₜ to L)</td>
<td></td>
<td>(0.33)</td>
<td>(0.38)</td>
<td>(0.20)</td>
</tr>
<tr>
<td>Thread length Lₜ</td>
<td></td>
<td>60mm</td>
<td>75mm</td>
<td>60mm</td>
</tr>
<tr>
<td>Thread major diameter Dₜ</td>
<td></td>
<td>6.25mm</td>
<td>6.25mm</td>
<td>6.25mm</td>
</tr>
<tr>
<td>Thread minor diameter Dₜ</td>
<td></td>
<td>4.45mm</td>
<td>4.45mm</td>
<td>4.45mm</td>
</tr>
<tr>
<td>(Ratio of Dₜ to Dₜ )</td>
<td></td>
<td>(0.71)</td>
<td>(0.71)</td>
<td>(0.71)</td>
</tr>
<tr>
<td>Thread height Hₜ</td>
<td></td>
<td>0.90mm</td>
<td>0.90mm</td>
<td>0.90mm</td>
</tr>
<tr>
<td>Thread pitch Pₜ</td>
<td></td>
<td>2.82mm</td>
<td>2.82mm</td>
<td>2.82mm</td>
</tr>
<tr>
<td>Effective thread contact area per unit length ⓜ</td>
<td>ⓜ</td>
<td>5.36mm</td>
<td>5.36mm</td>
<td>5.36mm</td>
</tr>
<tr>
<td>Thread angle Aₜ</td>
<td></td>
<td>45º</td>
<td>45º</td>
<td>45º</td>
</tr>
<tr>
<td>Complete thread length Lₜ</td>
<td></td>
<td>5mm</td>
<td>7mm</td>
<td>5mm</td>
</tr>
<tr>
<td>Incomplete thread end length Lₜ</td>
<td></td>
<td>4mm</td>
<td>4mm</td>
<td>7mm</td>
</tr>
<tr>
<td>Incomplete thread end shape Sₜ</td>
<td></td>
<td>Drill point</td>
<td>Drill point</td>
<td>Cut point</td>
</tr>
</tbody>
</table>

① L = Lₜ + Lₜ
② Hₜ = ( Dₜ - Dₜ ) = 2
③ Eₜ = ( Dₜ - Dₜ )² - ( Dₜ - Dₜ )² × π = σ
Figure 3: Average load-displacement curves of the wood screw withdrawal test results obtained in the investigation concerning shape factors other than complete thread
(a) Comparison of A60(0) and A60(15)
(b) Comparison of B75(0) and B75(15) and B75(30)
(c) Comparison of A60(0) and C60(0)

Figure 4: Average and standard deviation of the peak withdrawal resistances for six types of specimens used in the investigation concerning shape factors other than complete thread

Table 2: Results of the significance tests on the influence of unthreaded shank length and incomplete thread end shape on the withdrawal resistance

<table>
<thead>
<tr>
<th>Shape Factors</th>
<th>AVE</th>
<th>COV</th>
<th>Ratio of AVE</th>
<th>Significance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>A60(0)</td>
<td>6.41 kN</td>
<td>7%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A60(15)</td>
<td>8.11 kN</td>
<td>7%</td>
<td>126%</td>
<td>(α=0.01)</td>
</tr>
<tr>
<td>B75(0)</td>
<td>10.68 kN</td>
<td>8%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B75(15)</td>
<td>8.68 kN</td>
<td>11%</td>
<td>123%</td>
<td>(α=0.01)</td>
</tr>
<tr>
<td>B75(30)</td>
<td>10.55 kN</td>
<td>6%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C60(0)</td>
<td>6.82 kN</td>
<td>6%</td>
<td>122%</td>
<td>(α=0.01)</td>
</tr>
</tbody>
</table>

AVE: Average of the peak withdrawal resistances
COV: Coefficient of variation of the peak withdrawal resistances
α: Significance level

2.2 INVESTIGATION CONCERNING SHAPE FACTORS OF COMPLETE THREAD

2.2.1 MATERIALS AND METHODS

The wood species used in the experiment is sugi grown in Miyazaki Prefecture, Japan. This is air dried wood (average moisture content is 10.0%) and average specific gravity is 0.398. Only complete thread was driven into the plank of 125mm length × 100mm width × 20mm thickness in direction perpendicular to grain using an
electric impact driver. The specifications of four types of wood screws used in the experiment are as shown in table 3. Figure 5 shows the schematic diagrams of four types of withdrawal specimens. The number of specimens was thirty-five per type.

The withdrawal specimen was placed on the table of the universal testing machine and the head of wood screw was slid into the slot of screw fixture. The static loading was performed at cross-head speed of 2mm per minute until the withdrawal resistance decreased to less than 80% of the peak load. The displacement of the cross-head was measured and was assumed to represent the withdrawal displacement of wood screw.

**Figure 5: Schematic diagrams of four types of withdrawal specimens for the investigation concerning shape factors of complete thread**

### Table 3: Specifications of four types of wood screws for the investigation concerning shape factors of complete thread

<table>
<thead>
<tr>
<th>Type of wood screw</th>
<th>Name of wood screw (Ready made or Special order)</th>
<th>W (Special order)</th>
<th>X (Special order)</th>
<th>Y (Special order)</th>
<th>Z (Special order)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Nominal diameter D</td>
<td>7.00mm</td>
<td>7.00mm</td>
<td>5.70mm</td>
<td>7.00mm</td>
</tr>
<tr>
<td></td>
<td>Nominal length L</td>
<td>120mm</td>
<td>120mm</td>
<td>120mm</td>
<td>120mm</td>
</tr>
<tr>
<td></td>
<td>Material M</td>
<td>SWRCH22A</td>
<td>SWRCH22A</td>
<td>SWRCH22A</td>
<td>SWRCH22A</td>
</tr>
<tr>
<td></td>
<td>Manufacture method M</td>
<td>Rolling method</td>
<td>Rolling method</td>
<td>Rolling method</td>
<td>Rolling method</td>
</tr>
<tr>
<td></td>
<td>Surface treatment ST</td>
<td>Dural coat</td>
<td>Dural coat</td>
<td>Dural coat</td>
<td>Dural coat</td>
</tr>
<tr>
<td></td>
<td>Head diameter Dk</td>
<td>12.30mm</td>
<td>12.30mm</td>
<td>12.30mm</td>
<td>12.30mm</td>
</tr>
<tr>
<td></td>
<td>Head height Lh</td>
<td>6.00mm</td>
<td>6.00mm</td>
<td>6.00mm</td>
<td>6.00mm</td>
</tr>
<tr>
<td></td>
<td>Head shape Sf</td>
<td>Hexagon head</td>
<td>Hexagon head</td>
<td>Hexagon head</td>
<td>Hexagon head</td>
</tr>
<tr>
<td></td>
<td>Screw drive type DTs</td>
<td>Square socket</td>
<td>Square socket</td>
<td>Square socket</td>
<td>Square socket</td>
</tr>
<tr>
<td>Unthreaded shank portion</td>
<td>Unthreaded shank diameter Ds</td>
<td>7.00mm</td>
<td>7.00mm</td>
<td>7.00mm</td>
<td>7.00mm</td>
</tr>
<tr>
<td></td>
<td>(Ratio of Ds to D,crest)</td>
<td>(1.00)</td>
<td>(1.00)</td>
<td>(1.23)</td>
<td>(1.00)</td>
</tr>
<tr>
<td></td>
<td>Unthreaded shank length Ls</td>
<td>30mm</td>
<td>30mm</td>
<td>30mm</td>
<td>30mm</td>
</tr>
<tr>
<td></td>
<td>(Ratio of Ls to L)</td>
<td>(0.25)</td>
<td>(0.25)</td>
<td>(0.25)</td>
<td>(0.25)</td>
</tr>
<tr>
<td></td>
<td>Thread length Lt</td>
<td>90mm</td>
<td>90mm</td>
<td>90mm</td>
<td>90mm</td>
</tr>
</tbody>
</table>

### 2.2.2 RESULTS AND DISCUSSION

The wood screw withdrawal test results, which controlled for shape factors of complete thread such as thread pitch, thread major diameter and thread minor diameter, are shown in figure 6 and figure 7 and table 4.

The average of the peak withdrawal resistances of specimen X20(0) which extended thread pitch from 2.30mm to 3.00mm decreased by 0.08kN (3%) as compared with that of specimen W20(0). Since a significant difference was not observed in this case, it is considered that the influence of thread pitch on the withdrawal resistance is negligible.

The average of the peak withdrawal resistances of specimen Y20(0) which reduced thread major diameter from 7.00mm to 5.70mm decreased by 0.18kN (7%) as compared with that of specimen W20(0). And there was a statistically significant difference at a significance level of 0.05. From this result, it is conjectured that the withdrawal resistances are improved by increasing the effective thread contact area results from enlarging thread major diameter when thread minor diameters are the same.

The average of the peak withdrawal resistances of specimen Z20(0) which enlarged thread minor diameter from 4.40mm to 5.70mm increased by 0.18kN (7%) as compared with that of specimen W20(0). And the significant difference was established at a significance level of 0.05 between both results. Therefore, it is clarified that the withdrawal resistances are improved by increasing the ratio of thread minor diameter to thread major diameter results from enlarging thread minor diameter when thread major diameters are the same.

The table also includes formulas for calculating various parameters:

- \( L = L_u + L_t \)
- \( H_t = (D_{\text{crest}} - D_{\text{root}}) / 2 \)
- \( E_A = (D_{\text{crest}} - 2D_{\text{root}})^2 + (D_{\text{crest}} - 2D_{\text{root}})^2 \) x \( p \)
3 CONCLUSIONS

From the wood screw withdrawal tests conducted while controlling for shape factors, the following two results are obtained to reveal the features of soft and light wood.

(1) Until now, it has been common that the withdrawal resistance of wood screw was calculated using the penetration depth only threaded shank portion without unthreaded shank portion \[3\], because the resistance against withdrawal in wood with a comparatively high specific gravity depends almost entirely on threaded shank portion. But, it was clarified that the existence of unthreaded shank portion contributed to the withdrawal resistance as the results of wood screw withdrawal test in sugi that controlled for conditions of the penetration depth of unthreaded shank portion. Besides, the improvement rate
was also as high as 22-26%, and the difference was significant at 0.01 level of significance. Therefore, it is important to consider unthreaded shank portion with threaded shank portion when we develop a new wood screw that is appropriate for soft and light wood.

(2) And, it has been reported that the withdrawal resistance of threaded shank portion was approximately proportion to the effective thread contact area per unit length \(^4\) because the resistance against withdrawal in wood with a comparatively high specific gravity depends almost entirely on the mechanical resistance of thread. As opposed to this, the result was obtained from the wood screw withdrawal test in sugi that the withdrawal resistance of specimen, which decreased the effective thread contact area per unit length by increasing the ratio of thread minor diameter to thread major diameter when thread major diameters are the same, was improved 7%. Moreover, the difference was significant at a significance level of 0.05. Consequently, it will probably be beneficial to explore the optimal value of the ratio of thread minor diameter to thread major diameter when we develop a new wood screw that is appropriate for soft and light wood.

Finally, it is regarded that the common cause leading to above-mentioned results is the densification effect reflecting the features of soft and light wood which are squishy and not splintery. Further study is warranted to maximize the rate of withdrawal resistance improvement by developing taper shaped screw, exclusively for soft and light wood, which can successfully invoke the densification effect.

REFERENCES


DEVELOPMENT OF CROSS EMBEDDED JOINT USING LAGSCREWBOLT

Makoto Nakatani¹, Takuro Mori², Kohei Komatsu²

ABSTRACT: In previous researches, moment-resisting joints by using lagscrewbolts, which was a large screw fastener, were proposed for timber portal frame structures. The joints showed good performances and aesthetic outside views. However, some beam-column joints showed brittle failures, shear failure of a column member surrounded by lagscrewbolts and pull-out failure of a lagscrewbolt. In order to prevent the failures, a cross embedded joint by using lagscrewbolt was developed and confirmed the performances in this research. The beam-column joints consisted of a steel bracket and lagscrewbolts some of which were embedded into a column at skew angles. The joints showed a good ductile performance and a good reinforcing effect against the shear failure in a column and the brittle pull-out failure.

KEYWORDS: Lagscrewbolt, Cross embedded joint, Skew angle, beam-column joint

1. INTRODUCTION

The number of middle and large scale timber building in Japan recently is increasing due to reports showing that timber buildings are beneficial to the environment. Also, the number of houses constructed using timber portal frames is increasing due to demand from customers to have wider rooms. A moment-resisting joint by using Lagscrewbolt (LSB), which was a large screw fastener, was developed and confirm the performance in previous researches [1-4]. In the joints, LSBs were embedded into the glulam with a parallel to the grain direction or perpendicular to the gain direction. The joints showed good performances and aesthetic outside views. However, some beam-column joints showed brittle failures, which were shear failure of the column at the panel zone surrounded by LSBs and pull-out failure of LSB, due to a glulam size and/or an arrangement of LSBs in the joint.

In this research, a cross embedded joint system using LSBs was developed. The LSBs were embedded into a column at a skew angle to prevent the joint from a brittle shear failure at a panel zone in a column and/or a pull-out failure of LSB.

2. CROSS EMBEDDED JOINT

LSB was used for a series of the tests. LSB has a screw thread on the outside surface and a female thread inside the shank head. In this test, the shank diameter of LSB was 20 mm, the top tread diameter was 25 mm and the pitch length was 10 mm as shown in figure 1. The LSBs were made of steel S45C in accordance with the Japan Industrial Standard (JIS) G4051. A lead whole diameter for LSB in a wooden member was 22 mm. The LSBs were embedded into members by using an impact torque wrench.

Figure 3 shows details of an original beam-column joint with LSBs (Type A) and two proposed joints (Type B and C). They basically consisted of a steel bracket, high tension bolts, special slope nuts and LSBs as shown in Fig.2. The LSBs were connected to the brackets by the high tension bolts and the nuts. The LSBs of type B and C were embedded into a column at a skew angle and crossed in the column. In type B connection, a couple of LSBs for a column were embedded at a skew angle, and the others were embedded at perpendicular to the column. All LSBs of a type C connection for a column were embedded at a skew angle. All LSBs for beams of all type were embedded into them parallel to the grain direction.
3. EXPERIMENT

3.1 MATERIAL AND METHOD

Figure 4 and 5 show geometry and test set-ups of one beam and column joint (L shape) and two beams and column joint (X shape). Each of the two shape specimens was composed as the three joint types shown in figure 2, respectively. Beam and column were made of European red pine (*Pinus sylvestris*) glulam, and the grade was E120-F330 in accordance with the Japan Agricultural Standard (JAS). The dimension of beams and columns was 120 x 270 mm. In each test conditions one specimen was prepared. Total number of specimens was 6 in the test.

Cyclic loads were applied to the specimens by a hydraulic jack. The rotational deformation between beam and column were measured by LVDTs as shown in Fig. 4 and 5.

Figure 2: Steel connection for the joints

Figure 3: Geometry of cross embedded joint

Figure 4: Test set-up of L shape experiment

Figure 5: Test set-up of X shape experiment
3.2 FAILURE PHENOMENA

Figure 6 and 7 show failure phenomena of L shape specimens. The failure phenomenon of type A specimen was a brittle pull-out failure of LSB as shown in Fig.7 (i). However, failure phenomena of type B and C specimens were ductile failure of a slide pull-out of the skew LSBs in the column and an embedment deformation between the steel bracket and the column as shown in Fig.7 (ii). Figure 8 and 9 show failure phenomena of X shape specimens. The failure phenomenon of type A specimen was brittle shear failure at the panel zone surrounded by LSBs in the column as shown in Fig.9 (i). However, failure phenomena of type B and C specimens were ductile failure of the slide pull-out of the LSBs and the embedment deformation as shown in Fig.9 (ii). The crossed LSBs prevented the column from the brittle shear failure. Type B and C joints were significance improved ductility compared with A type joint by the cross embedded LSBs in the column.

![Figure 6: Failure phenomenon of L shape specimen](image)

![Figure 7: Details of failure phenomena of L shape joint](image)

![Figure 8: Failure phenomenon of X shape specimen](image)

![Figure 9: Details of failure phenomenon of X shape specimen](image)

3.3 EXPERIMENTAL RESULT

Table 1 shows all test results. Figure 10 and 11 show results of L shape and X shape specimens, respectively. In L shape specimens, the rotational angles at maximum moment ($\theta_{\text{Mmax}}$) and the energy absorption of type B and C joints were improved compared with those of type A joint due to the ductile embedment deformation. However, maximum moment and Rotational rigidity of type B and C joints were lower than that of type A joint, because a slip modulus of the skew LSB is lower than that of the LSB in the original joint. The LSBs were screwed into the column to the pull-out direction.

In X shape specimen, $\theta_{\text{Mmax}}$ and the energy absorption of type B and C were also improved compared with that of type A joint. The maximum moment of type B and C joints were also improved compared with that of type A.
joint. However, rotational rigidity of type B and C was lower than type A joint. The cross embedded joints (type B and C joints) showed a good ductility compared with original joint (type A joint).

Table 1: Experimental result

<table>
<thead>
<tr>
<th>Joint</th>
<th>$M_{\text{max}}$ (kNm)</th>
<th>$\theta_{\text{max}}$ (rad)</th>
<th>Rotational rigidity (kNm/rad)</th>
<th>Energy absorption</th>
</tr>
</thead>
<tbody>
<tr>
<td>LA type</td>
<td>32.3</td>
<td>0.029</td>
<td>2876</td>
<td>0.88</td>
</tr>
<tr>
<td>LB type</td>
<td>21.7</td>
<td>0.040</td>
<td>1334</td>
<td>1.00</td>
</tr>
<tr>
<td>LC type</td>
<td>28.8</td>
<td>0.100</td>
<td>472</td>
<td>2.58</td>
</tr>
<tr>
<td>XB type</td>
<td>29.4</td>
<td>0.144</td>
<td>1034</td>
<td>4.78</td>
</tr>
<tr>
<td>XC type</td>
<td>39.0</td>
<td>0.181</td>
<td>516</td>
<td>5.90</td>
</tr>
</tbody>
</table>

Note: $\theta_{\text{max}}$ is Rotational angles at maximum moment

4. CONCLUSIONS

The failure phenomena of the proposed cross embedded joint in L shape joint was changed from a brittle pull-out failure of LSB in the original joint to the ductile slide pull-out failure of the LSBs embedded at a skew angle into the column. The rotational angles at maximum moment and the energy absorption of proposed joints were improved compared with those of original joint. The failure phenomena of the proposed joint in X shape joint was changed from a brittle shear failure at the panel zone surrounded by LSBs in the column to a ductile embedment deformation between the steel bracket and the column. The crossed LSBs prevented the column from the brittle shear failure at the panel zone. Therefore, the ductility was significantly improved compared with original joint. The maximum moment of the proposed joints was also improved. However, the rotational rigidity of the proposed joints was lower than that of the original joint. The cross embedded joints showed a good ductility compared with the original joint.

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REFERENCES


A HYBRID SYSTEM FOR ENHANCING RACKING PERFORMANCE OF TIMBER PANELS

Sana Munir¹, Abdy Kermani², Ian Harrison³

ABSTRACT: Timber frame can offer many aesthetic and structural benefits over other construction materials, for example effective insulation for energy efficiency, sustainable design and ease and speed of construction. For all the known benefits of using timber frame there are certain limitations that are often linked to its strength and stiffness performance in connection systems. A key design limitation of timber frame structures is that the lateral resistance to wind loads becomes increasingly difficult to accommodate as opening sizes increase. These limitations are restricting the potential for market growth as clients are increasingly asking for detached dwellings with relatively large openings in their shorter/narrow walls and for multi-storey buildings often with large openings.

This research aimed to resolve the issues arising from the lack of racking resistance in timber frame buildings and is focused on developing a structural reinforcement system for the timber frame market. This will facilitate the construction of detached dwellings and multi-storey buildings with large spans and openings. The system will allow greater architectural flexibility whilst being simple to install and cost effective.

KEYWORDS: Timber frame, Shear wall, Racking resistance, Large openings

1 INTRODUCTION

To overcome the difficulty of insufficient racking resistance in large openings in building elevations; designers utilise a hierarchy of solutions to resist the wind loads. The most commonly used solutions are 1) reducing fastener spacing, 2) using double sheathing, 3) increasing the thickness of sheathing, and 4) utilising large stiff panels or steel portals. However, on-site there are a number of problems with using current solutions for example warping of sheathing by reducing fastener spacing, in particular, in humid construction conditions, and differential settlement between the timber frames and steel portals, which also limits the architectural scope for popular features such as picture windows.

This research aimed to address the problems arising from racking in timber framed buildings and is focused on developing a hybrid structural reinforcement system for the timber frame market, in order to facilitate the construction of detached dwellings and multi-storey buildings with large spans and openings. The main objective was to develop a system which permits greater architectural flexibility whilst being simple to install and cost effective.

The development of the concept started with a parametric evaluation of the stiffness of the existing systems used to provide racking resistance in timber platform frame construction. The study has significant importance in identifying and selecting the right combinations of stiffness and strength levels achievable at the connections and in particular at the foundation level, based on the current UK design/construction practices. This then led to a more detailed analytical and experimental examination of the connections' component configurations and their effects on the performance of timber frames. The concept has been gradually developed into a range of effective moment resisting hybrid timber portals to limit the moment and uplift demands on the foundations for a range of applications. The development of the products, named “Strong-Portals”, required detailed understanding of the factors which contributed to improving racking performance of structural timber systems. Structural modelling and numerical evaluation of the timber fastener systems, augmented with laboratory testing and detailed knowledge of UK construction methods, have helped make the project a success.

2 BACKGROUND

The term ‘shear wall’ is usually used for vertical elements such as walls when appropriately designed to transmit force in its own plane. Horizontal floor diaphragms are designed to transfer loads to shear walls which carry the loads to the foundations. In the structural design of timber the lateral resistance of the walls to carry wind loads transferred from diaphragm is known as
racking resistance. The difference between a shear wall and an ordinary stud wall is that a shear wall in timber construction is a load bearing wall that is designed to take racking loads in addition to vertical loads [1]. Openings in shear/racking resisting walls can have a considerable effect on their performance. In the design of timber framed buildings it is often necessary to provide appropriate rigid framing structure to be able to transfer shear loads around the opening. In cases where there are large openings such as garage or patio doors or large windows etc. the racking performance becomes a design issue as it limits the resistance from the available area of the shear wall.

There has been considerable research on racking resistance of timber frame panels in the past and a large amount of data related to the performance of timber has been published internationally. Various solutions have been proposed to improve the lateral resistance of shear walls. Karalic et al. [2] in 1997 developed a prefabricated reinforcing structure comprising a rectangular frame with triangular structures along the entire length which provided more resistance to horizontal forces as compared with standard stud walls. Midply developed by Karacabeyli et al. [3] is a structural shear wall system for severe earthquakes. It utilizes a novel arrangement of sheathing and framing members to give higher lateral resistance but pose problems regarding the insulation and installation of services. To enhance the shear capacity of walls, Pryor S. [4] developed channel metal strip reinforcement for the edges of sheathing. This considerably improved the tear out resistance of nails and prevented the nails from pulling out. However, with increased capacity; the requirement for uplift from hold-down considerably increased.

Griffiths [5] investigated the performance of shear walls in the mid to late 80’s based on UK methods of construction which provided the empirical basis for British Standard BS 5268-6.1[6]. The investigation carried out by Minjuan et al. in 2010 [7] on the racking performance of walls using Chinese wood based panels sheathed with gypsum or magnesium oxide board, demonstrated good stiffness properties. Yasumura [8] carried out research on racking resistance of wood-framed shear walls with various opening configurations using experimental work and finite element analysis. The investigation on racking performance of wood shear walls consisting of finger-joined studs carried out by Meng et al. [9] suggested a very small reduction in racking capacity as compared to unjoined studs. Salenikovich and Payeur [10] performed a study investigating the correlation between lateral resistance and the magnitude of the vertical distributed load applied to a light-frame wall. Shim and Hwand et al. [11] developed a hybrid model using traditional Korean timber construction and light frame constructions to help with the increase in timber demand in Korea. The study was carried out to define the lateral load resistance of hybrid structures under cyclic lateral load.

Leitch and Hairstans [12] carried out a series of racking tests to assess the accuracy of simplified plastic model for the prediction of racking strength. The study also highlighted the need for a “stiffness check” in order to ensure the structures do not suffer from excessive deflections.

3 METHODS

A parametric evaluation of the effects of variation in stiffness properties in the joints and foundations of a timber portal frame of typical 2.4 m x 2.4 m dimensions with 10 kN racking load was carried out, as shown in Figure 1.

![Figure 1: Analysis model](image1.png)

The study considered the eave joints to possess rotational springs only (i.e. rotationally semi-rigid). However, the foundations were permitted to have both translational and rotational springs. Then the effects of connection stiffness, from fully rigid to fully pinned conditions, on the structural behaviour of the portal frame was evaluated. Sample results are shown in Figure 2.

![Figure 2: Comparison of horizontal deflection at eave joints of portal for varying stiffness levels at foundation](image2.png)

The results of the study showed that there is considerable reduction in lateral deformation of the frame with the increase in the rotational stiffness of the eave joints from 0% (fully pinned) to about 50% rigidity levels. However, further increasing the stiffness/rigidity of the eave joints resulted in little to no improvement in performance of the portal.

The development of the concept for the portal system involved a series of tests in order to achieve the best strength and stiffness combination for the portal. Two sets of tests were carried out on:
i. a series of full-scale eave-joints utilising differing header and column materials, nail spacing and metal work compositions in order to determine the best combination of materials and joint configurations.

ii. a series of full-scale portals to prove the developed concepts and to determine structural performance characteristics.

3.1 CONNECTION DETAILS

A total of 32 full-scale joints with a range of member and component configurations were designed and manufactured. Replicate prototype joints with column and beam members of solid C24 timber, LVL Kerto-S as well as boxed hollow sections made with solid studs and OSB/3 sheathing or Finnish birch plywood were constructed. A range of hybrid jointing systems was also developed and manufactured using specially designed metalwork components by Simpson Strong-Tie, UK. Replicate joints were also produced using both nailed and screw fixings. The aim was to develop an optimum joint configuration by examining the effects of different component configurations on the structural behaviour of the connection systems, in particular, on their stiffness characteristics, structural integrity as well as on their ease and speed of manufacture/assembly.

3.1.1 JOINT TEST SETUP

The test setup for the joints used is given in Figure 3. The joint corresponds to closing joint in the portal indicated as (3) in Figure 1. The leg lengths and the loading arrangement were designed to simulate, where possible, the loading combinations as those in the full-size portal frame.

As mentioned earlier, a comparative study was carried out using different combinations of sheathing material and fasteners. Metal straps for connecting the column to the header members were profiled and optimised to enable the joint to carry the maximum moment by using different fastener configurations for outer and inner connecting straps and in turn to provide maximum stiffness.

3.1.2 MODES OF FAILURE

Two failure modes, for all joint types tested, were observed. These were the splitting of header and shearing of OSB which occurred at extreme load levels, as shown in Figure 4.

3.2 PORTAL TESTING

Following the successful outcome of the joint tests, a series of full-scale portal frames was designed and constructed utilising the key features from the tested joints. This aimed to bring together the optimised features developed during the connection development work. A total of eight portal frames were manufactured and were tested in accordance with the requirements of BS EN 594: 2011 [13]. The tests were designed to further optimise the structural performance of the frames, “Strong-Portals”, and to fine-tune their factory production and on-site assembly details. The beam-to-column connections in Strong-Portals are designed to resist the induced moments generated with the application of racking loads, which in turn reduces the moment and uplift requirements from the foundation. Therefore it gives the designers the flexibility to use concrete pad foundations for portals that can easily be adapted to beam and block floors, with some modification, as widely used in the United Kingdom. In Figure 5 a typical “Strong-Portal” during test preparation is shown.

Strong-Portal is a hybrid structural system which enables efficient utilisation of strength of timber and engineered wood products in a high performance racking resisting frame. Some details are illustrated in Figure 6.

4 SUMMARY

The output of this project is the introduction of a new concept to the timber frame industry as a result of a highly successful collaboration between industrial and academic partners. The concept has provided a practical solution that eliminates the problems associated with
current options and will also interface simply with the traditional timber frame panels.

Figure 5: A Strong-Portal during test preparation.

Figure 6: (a) Typical Installation of Strong-Portal (b) Hold-down detail (c) Moment Connection joint.

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REFERENCES
TIMBER STRUCTURES IN THE NEW FACILITIES OF ÁGUAS EMENDADAS ECOLOGICAL STATION IN BRASÍLIA – BRAZIL

Roberto Lecomte de Mello¹, Catharina Macedo², Ana Carolina Salustiano³

ABSTRACT: The Águas Emendadas Ecological Station is a protected area located in the Federal District, and beyond the great biological diversity presents a dispersion of water quite unique, because its waters flows in different directions forming two big brazilian rivers. Its facilities are composed of some buildings, some of them with timber structures as the Visitors Center. It has been proposed the use of pressure treated rounded eucalyptus as the structural element of the new buildings to be constructed, as the Administration building, the Control Center, Accommodation for researchers and fire department, Garages and the main gate. The buildings were designed according to bioclimatic principles as natural lighting and ventilation, thermal inertia, green covering and protection of the facades. Post-beam structures with bolted connections were proposed as the main structural system and secondary timber structures will be made of tropical hardwood. The architectural aesthetics resulting from the presence of wood suggests that no other building material transmits so much energy integration between the built and natural environment. This paper presents the design of new buildings, highlighting the integration between architectural and structural designs in wooden buildings.

KEYWORDS: Wood, Eucalyptus logs, Timber structures

1 INTRODUCTION

The Águas Emendadas Ecological Station is one of the protected areas in Federal District of Brazil, located at the Cerrado (Brazilian Savannah) biome. It has an extraordinary geographical location and it contains different types of vegetation formation of the Cerrado ecosystems. Other important characteristic of this area is the phenomenon of the water dispersal, which happens at its wetland (vereda in Portuguese). The water, which originates in the vereda, flows in different directions, forming two big Brazilian rivers: Maranhão, tributary to the Tocantins river; and São Bartolomeu, tributary to the Paraná river. Águas Emendadas means ’connected water’, in english.

The Ecological Station has some facilities like small administration buildings and a Visitor’s Center, which was built with timber structures of rounded eucalyptus. In order to keep the architectural aesthetics present in the station, it was proposed the use of eucalyptus logs as main structural material for the new facilities of the area. Existing buildings will be reformed and new buildings were proposed like the Administration building, Control Center, Accommodation for researchers and fire department, Garages and the main gate.

2 METHODOLOGY

The design of the facilities of the Ecological Station was conceived based in the following premises:

- To be designed according to bioclimatic guidelines, in order to be fully adapted to local climate, with some requirements as natural ventilation and illumination, thermal inertia, green covering and protection against sunlight in the facades, aiming to build a “sustainable building”;  
- To be in consonance with the existing facilities (made of timber structures) and to harmonize with the surrounding environment;  
- To propose mixed building solutions between traditional concrete structures and timber structures, aiming to promote “greenest materials”;  
- To show the required integration between structure and architecture in wooden buildings;  
- And finally to highlight the imposing architectural aesthetics that comes from the presence of timber structures in natural areas.

3 DESCRIPTION OF THE PROJECTS

The buildings were designed to be implanted in a specific area of the Ecological Station, where all the administration services will be concentrated in order to cause less impact in the preserved area.

Covered gateways will link all the buildings and native trees will be planted in the area, aiming to reconstitute the original vegetation that occurred in the region. Figure 1 shows a general view of the new facilities proposed to the Águas Emendadas Ecological Station.
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Figure 1: General view of the new facilities of the Águas Emendadas Ecological Station

Post-beam structures were designed for the new buildings, by using eucalyptus logs as the main material. Timber structures will receive bolted joints with galvanized metals and all the pieces will be stained. The main eucalyptus specie to be used in the structures will be *Eucalyptus citriodora*.

Secondary timber structures in the flooring and ceiling will receive sawed wood Tuterubá (*Pouteria oblanceolata*), an Amazonian hardwood that is more and more available in our market due to its resistance and structural behaviour. Figures 2 and 3 illustrate the plans and an external view of the Accommodation for researchers and fire department.

The Administration building will be made from a reconstruction of an existing building in the Station. A mixed solution between armed concrete and timber structures was proposed. Figures 4 and 5 show two facades and an external view of the Administration.

Secondary timber structures in the flooring and ceiling will receive sawed wood Tuterubá (*Pouteria oblanceolata*), an Amazonian hardwood that is more and more available in our market due to its resistance and structural behaviour. Figures 2 and 3 illustrate the plans and an external view of the Accommodation for researchers and fire department.

The Administration building will be made from a reconstruction of an existing building in the Station. A mixed solution between armed concrete and timber structures was proposed. Figures 4 and 5 show two facades and an external view of the Administration.

It will be implanted an Information and Control Centre that will be the main building of the Station. In that building will be located a library, auditorium, exposition hall, refectory and a control centre that will monitor the Ecological Station area. The plan and facades of the building is illustrated in Figures 6 and 7.
Asymmetric timber structures were proposed to support “covered squares”, to be implanted in the Administration Building and in few points of the covered catwalks that links all the buildings, as illustrated in Figure 8. Those structures will reproduce trees designs and will be integrated to the surrounding vegetation.

A huge deposit will be constructed with mixed constructive solutions using timber and metallic structures. Figure 9 shows an external view of this new building.

It was designed according to the architectural aesthetics presented in the other buildings. The name of the ecological station will be highlighted in the facade and the timber structure will be viewed by the visitors; that will be their first impression of the area. Figures 10 and 11 show the plan and an external view of the main gate.

The main gate of the Ecological Station was designed to be an attraction to visitors and to highlight the access from the highway.

It's imperative intensify the use of Amazonian hardwood in the building sector, because the rational use of our forest-based products means adding value and protecting the forested areas in our country.

ACKNOWLEDGEMENT

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GLULAM STRUCTURES IN THE NEW LPF’S WOOD DRYING AND WOOD TREATMENT LABORATORIES – BRAZIL

Roberto Lecomte de Mello¹, Renato Sette²

ABSTRACT: The Forest Products Laboratory (LPF) is a research centre linked to the Brazilian Forest Service (SFB) and it has been acting for about 38 years in the search of technological solutions for the sustainable growth of the forest activity in Brazil, trough the research and the development of new methods related to the use of forest-based products.

Inside this philosophy, LPF develops projects related to the rational use of wood in the building sites, aiming to improve the design and building technologies, where new structural and constructive solutions can be established. Several buildings like pre-fabricated houses for remote areas (islands, hills and others) and governmental buildings were designed and executed, aiming to promote timber structures in our country.

One of these wood buildings is the new LPF’s Wood Drying and Wood Treatment Laboratory, where it was conceived a mixed constructive solution between glulam structures, armed concrete and masonries. The glulam structure was executed in porticos 15.00m-long and 6.50m-height, with metallic covering with thermal and acoustic insulation. Glulam pieces were made of eucalyptus (Eucalyptus sp) and came from the southern region of Brazil.

This article illustrates the execution of the laboratories, showing timber as a structural material and highlighting the several possibilities of building in wood. The presence of timber structures in the building gives it a unique appearance and contributes to promote the rational use of our forest-based products.

KEYWORDS: Wood, Wood buildings, Glulam structures

1 INTRODUCTION

The structural use of wood in Brazil is still very limited, with a massive presence of buildings made of reinforced concrete and steel. Although the country is the largest producer and consumer of tropical timber in the world, this is intended primarily for secondary uses such as braces, roof structure, window frames and furniture. Factors such as the Portuguese colonization, based on the intensive use of sand and stone as well as government policies that favoured the cement and steel industries, contributed to our building tradition and the lack of knowledge by the population in relation to wooden buildings.

In order to promote structural and constructive use of wood in construction, LPF develops architectural and structural wooden building designs for several governmental agencies and for its own facilities. In this context, it was designed and built the new building for the Wood Drying and Wood Treatment Laboratories, where mixed constructive solutions between glulam and concrete structures were proposed, while also utilizing masonries and wood walls. This building is a continuation of the first built pavilion, where LPF’s biomass energy area is installed. This paper presents the design and implementation of this building, highlighting the integration between different structural solutions and exposing the qualities and potential of glulam building technologies in Brazil.

2 METHODOLOGY

The design of the Wood Drying and Wood Treatment Laboratories was conceived based in the following premises:
• To act as wood research laboratories with technical areas and technicians rooms;
• To propose mixed building solutions between traditional concrete structures and glulam structures;
• To show the required integration between structure and architecture in wooden buildings;
• To expose physical and mechanical behaviour of fast-growth species and tropical hardwood species;
• And finally to highlight the imposing architectural aesthetics that comes from the presence of glulam structures.

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² Renato Sette, Brazilian Institute of the Environment and Renewable Resources, SCEN Trecho 2 Edificio Ibama, 70818 - 900 Brasilia, Brazil Email: renato.sette@yahoo.com.br
3 DESCRIPTION OF THE PROJECT

The building destined to the new laboratories was conceived as a large pavilion with two levels linked by a stairway. Its implantation followed the structural modulation of the existing pavilion, destined to Biomass Energy Laboratory.

At the ground level are located rooms for laboratories, areas for large machinery and equipment, and also toilets. The rooms are separated by masonries which are independent of the timber structures. Figure 1 shows the plan of this level.

![Figure 1: Plan of the ground floor](image)

In the first floor (mezzanine) are located the technician’s rooms and this level is all made of wood: Wood siding and flooring structure were executed with Amazonian wood Tuturubá (*Pouteria oblanceolata*). The mezzanine plan is shown in Figure 2.

![Figure 2: Plan of the mezzanine floor](image)

4 EXECUTION OF THE BUILDING

The building construction starts with the execution of the concrete foundations. The timber structure is fastened in concrete blocks that are raised about 60cm of the ground, which gives a lighter appearance to the building and contributes to the durability of the glulam porticos. Figures 5 and 6 show the assembling of the structure.

![Figure 5: Assembling of the first glulam portico](image)

Wood walls located in the mezzanine were made of painted plywood in the internal side and hardwood siding in the external side. Figure 4 shows an external view of the building, highlighting the wood walls.

![Figure 4: External views of the building](image)

The glulam structure was executed in porticos 15.00m-long and 6.50m-height, with metallic covering with thermal and acoustic insulation. The porticos had being set in concrete blocks approximately 60cm from the ground and these blocks were shaped in order to avoid humidity, according to Figure 3. Glulam pieces were made of eucalyptus (*Eucalyptus sp*) and came from the southern region of Brazil.

![Figure 3: Cross section of the building](image)

![Figure 6: Assembling of the glulam porticos](image)
After the assembling of the structure, the covering structure receive wooden pieces that act as “wind braces”, and after this the metallic covering with thermal and acoustic insulation is installed, according to Figures 7 and 8. Inside the building, masonries and the wooden mezzanine are executed, as it can be viewed in Figure 9.

Inside the building, the wooden mezzanine is executed with the flooring made of Ipê (*Tabebuia serratifolia*) and the pieces have a moisture content not exceeding 13%. The wooden walls are assembled with studs that are jointed between the covering structures and the finished flooring. Figures 10 and 11 illustrate the execution of the mezzanine.

After the assembling of the walls, wooden pieces are stained and in the internal side of the walls painted plywood is installed after the execution of electrical and logical network. Glulam porticos are stained and masonries are painted. The finished building is showed in Figures 12, 13, 14, 15 and 16.
5 CONCLUSIONS
The design and the construction of the LPF’s Wood Drying and Wood Treatment Laboratories shows wood performance as a structural material and highlights the several possibilities of building with glulam structures. The presence of wood in the building gives it a unique appearance and contributes to promote the rational use of our forest-based products.

REFERENCES
A STUDY ON THE EDUCATIONAL EFFECTIVENESS OF THE TIMBER STRUCTURAL DESIGN WORKSHOP

Katsuhiko Kohara¹, Masaya Kishita², Masaru Tabata³, Yasushi Komatsuzaki⁴ and Masato Ohchi⁵

ABSTRACT: A proposal based on the products concept is made, and the static loading test of those articles is visited for study in the products development training and the workshop. It can be understood how the stress flows by loading test's doing the articles developed with themselves. A timber building is known, and it is be interest, and we must increase the people who make timber structure a work to increase demand for the wooden building. A workshop found that it was effective as one of those techniques.

KEYWORDS: Timber structural design workshop, Educational tool, Specifications

1 INTRODUCTION

A timber structural design workshop for the children, the common and the authorized architects and builders were carried out, and that educational effectiveness was verified. Even many authorized architects and builders have the wrong knowledge as following. The timber architectures are poor at the typhoon. And the wooden building whose pillar is thick is resistant to the earthquake. We must increase demand for the timber building which has seismic performances firmly. There are a few timber structural design workshops to the children and the common. It is necessary to know the stress flow of the building and the strength of the lumber, so that a participant may have interest and knowledge may be acquired.

2 TIMBER STRUCTURAL TOOLS

We are carrying out technological training to the architect and the administrative person in charge to have regional timber material use with business. An architect and the person in charge of the administration don't know about lumber and wooden structure, because there is no curriculum in the building education of the Japanese university as for the wooden structure. We made a design support structure tools to have them know the characteristics of the regional area, the characteristics of the regional timber material and the earthquake-proof performance of the wooden building. We are carrying out technological training as some textbooks which are design support tools.

2.1 BEAM SPAN TABLES OF CEDAR IN GIFU

A beam in the lumber being used for the residence has a little utilization of the local area material. Therefore, technology must be studied to use with a beam material as one method to raise area material use. We make the beam span tables of Japanese cedar in Gifu. It is shown in figure 2, 3. We carried out the technological training of twelve times in Gifu Prefecture, Aichi Prefecture and Osaka Prefecture by the beam span tables. It found that 80% in the participant wanted to use regional timber material for the beam with future business in the questionnaire investigation after the technological training. The lecture program, the cooperative program and the experimental program are shown in figure 1. We make the beam span tables of Japanese cypress in Gifu. It is shown in figure 4. The young's modulus appearance frequency distribution of Japanese cedar is shown in figure 5. Young's modulus is low as much as the lumber of the big section.

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Figure 1: Lecture Program, Cooperative program & Experimental program

Figure 2: Beam Span Tables of Japanese Cedar

Figure 3: Beam Span Tables of Japanese Cedar

Figure 4: Beam Span Tables of Japanese Cypress

Figure 5: Young’s Modulus Appearance Frequency Distribution

<table>
<thead>
<tr>
<th>Young’s Modulus Appearance Frequency Distribution</th>
<th>2007.4 - 2010.9</th>
<th>Japanese Cedar</th>
<th>Length: 4,000mm</th>
<th>N=16,458</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0%</td>
<td>20%</td>
<td>40%</td>
<td>60%</td>
</tr>
<tr>
<td>120 x 120</td>
<td>4.0%</td>
<td>32.8%</td>
<td>40.6%</td>
<td>20.4%</td>
</tr>
<tr>
<td>120 x 150</td>
<td>4.7%</td>
<td>37.2%</td>
<td>40.8%</td>
<td>15.6%</td>
</tr>
<tr>
<td>120 x 180</td>
<td>6.3%</td>
<td>40.5%</td>
<td>39.2%</td>
<td>13.2%</td>
</tr>
<tr>
<td>120 x 210</td>
<td>6.5%</td>
<td>43.7%</td>
<td>38.8%</td>
<td>10.4%</td>
</tr>
<tr>
<td>120 x 240</td>
<td>9.5%</td>
<td>47.2%</td>
<td>34.7%</td>
<td>8.2%</td>
</tr>
<tr>
<td>120 x 270</td>
<td>11.3%</td>
<td>49.7%</td>
<td>30.2%</td>
<td>8.1%</td>
</tr>
<tr>
<td>120 x 300</td>
<td>14.0%</td>
<td>54.3%</td>
<td>26.5%</td>
<td>5.1%</td>
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<tr>
<td>120 x 330</td>
<td>19.3%</td>
<td>54.2%</td>
<td>24.1%</td>
<td>2.8%</td>
</tr>
<tr>
<td>120 x 360</td>
<td>21.4%</td>
<td>55.7%</td>
<td>18.4%</td>
<td>4.4%</td>
</tr>
</tbody>
</table>

E50
E70
E90
E110
E130

n=3730
n=2275
n=2168
n=2501
n=2545
n= 741
n=1439
n= 166
n= 901

Figure 5: Young’s Modulus Appearance Frequency Distribution
2.2 BEAM SPAN TABLES OF CEDAR BOX-BEAMS

We make the beam span tables of Japanese cedar box-beams in Gifu. It is shown in figure 6. We carried out the technological training in Gifu Prefecture and Shimane Prefecture by the beam span tables.

![Figure 6: Beam Span Tables of Japanese Cedar Box-Beams](image)

![Figure 7: Japanese Cedar Box-Beams](image)

2.3 WOODEN BUILDING STRUCTURE SPECIAL SPECIFICATIONS IN GIFU

Even if lumber is available in the design, you must be able to do the lumber utilization intended in the design at the time of the execution. ‘The tree of Gifu’ wooden building structure special specifications and the commentary were made, and a technological course was carried out as a mediation of the design and the execution. A technological course got the 2,000 participations of the architect.

![Figure 8: 'The Tree of Gifu' Wooden Building Structure Special Specifications and the Commentary](image)

3 WORKSHOPS FOR CHILDREN

The design of the timber building of Nursery school was carried out through the workshop by the local resident's parents and children. Because it is the nursery school which used local area material, it is necessary to introduce the good point of the tree to the kindergarteners. It went through the workshop at the woods, lumber processing, the stage of construction of the construction site so that kindergarteners might have an attachment to the tree. Kindergarteners joined a workshop eagerly.

![Figure 9: Young Modulus Measurement](image)

![Figure 10: Workshop for Children in the Forest](image)

![Figure 11: Workshop for Children in the Timber Mill](image)

![Figure 12: Kindergarten with Japanese Cedar in Gifu](image)

4 CONCLUSIONS

It is understood that educational effect by the workshop spreads even to the management of the facilities in these two facilities. It became a building with the story which the regional timbers were used for, and became the building where an attachment was popular with both facilities by the design through the workshop by the local residents.
ACKNOWLEDGEMENT
We appreciate the client provided an opportunity of the workshop. We appreciate Archi-Cube Architect & Associates

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AIR INFILTRATION IN TIMBER FRAME HOUSE WITH WOOD STUD INFILL WALL

Sejong Kim¹, Hyun Bae Kim², Joo Saeng Park³, Moon Jae Park⁴, Jun Jae Lee⁵

ABSTRACT: This research was carried out to evaluate the airtightness of timber frame house. 2 houses were selected and blow door test was used for measuring the airtightness. The airtightness in ACH50 were 10.98⁶ and 6.07⁶ respectively. Thermal imager was used to find out air infiltration route. From obtained thermal images, heat loss was occurred mainly in joints between wall and frame. So, the joints were covered with PE tapes. And, airtightness was evaluated. Then, effect of the joint on airtightness was about 0.5⁶ in ACH50.

KEYWORDS: Air infiltration, , Timber frame house, Wood stud infill wall

1 INTRODUCTION

Lately, The market of Han-ok, Korean traditional house, has been increased with the interest of eco-friendly living spaces. Han-ok has the advantage of harmony with nature. Wood posts and beams have a role as not only structural member but also interior in Han-ok. But, high cost from wood members with large diameter, defects from drying process of wood members after construction, extended construction period from cutting process in field and etc. have hindered the growth of Han-ok market. Then, adapted modern Han-ok was developed to solve such problems. In that, wood members were cut into end-use dimensions in plant and engineered woods were used for structural member.

Energy efficiency in residential buildings became an important issue with the flow of government policies concerned with reducing energy use in buildings. in 2009, the government of Korea had set the goal of cutting greenhouse gas emissions by 30% by 2020, relative to the ‘business as usual’ (BAU) scenario. Considerable potential for reducing the total energy use is inherent in residential building. Especially, heating energy demand occupies significant part. So, the decrease of heating energy demand in residential buildings is important to raise the efficiency of energy use and to reduce greenhouse gas emissions.

In Han-ok, insufficient air tightness and high thermal transmittance of building envelopes resulted into high cost in heating. Required thermal transmittance of building envelope for the guarantee of energy efficiency can be obtained by wall configuration with high thermal resistance. But, Air tightness is determined by many factors that can not be quantified. Then, the air tightness of building is usually evaluated after construction.

In this study, we tried to evaluate the air tightness of Han-ok, timber frame with wood stud infill wall, and analyze the causes of air infiltration.

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Figure 1: test houses
house ‘B’ in Pocheon, Korea. The both houses were two-stories and south-facing. Floor heating system was applied for heating. The volumes of test house ‘A’ and ‘B’ were 644m³ and 585m³ respectively.

2.2 EVALUATION OF AIR TIGHTNESS

In order to estimate the air infiltration of test building Pressurization /Depressurization Method was used. This is published in ASTM E779-10(Standard test method for determining air leakage rate by fan pressurization). Air tightness can be expressed by CFM50 and ACH50. CFM50 is defined as the air flow (in cubic feet per minute) needed to create a 50-pascal pressure change in the building envelope. CFM50 is one of the most basic measurements of air tightness. The expression ACH50 refers to the number of times in one hour that the inside air volume is replaced with outside air at a house pressure difference of 50 Pa. It is calculated by multiplying the previously obtained CFM50 value by 60 (minutes) and dividing by the volume of the house.

\[
ACH50 = \frac{CFM50 \times 60}{\text{volume of building}}
\] (1)

Minneapolis Blower Door was used to evaluate air tightness of the test houses (figure 2). The test method followed EN13829. The test mode was ‘Depressurize’. It was installed at the front door of the test house. All indoor side doors were opened to estimate the air tightness of entire test house.

2.3 AIR TIGHTNESS OF TEST HOUSES

Measured air tightness of the test houses are shown in table 1.

<table>
<thead>
<tr>
<th>Case</th>
<th>CFM50 (±0.2%)</th>
<th>ACH50 (h⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4162 cfm</td>
<td>10.98 ACH</td>
</tr>
<tr>
<td>B</td>
<td>2091 cfm</td>
<td>6.07 ACH</td>
</tr>
</tbody>
</table>

Average value of ACH50 reported in Europe, Canada, USA and etc. was 6.8 h⁻¹. In case of domestic detached houses, 16.4 h⁻¹(average of 22 cases) and 12.91 h⁻¹(1 case) were reported. Compared with previous studies, air tightness of case A showed similar performance with domestic detached house and case B with foreign one. It was considered that high value of case A was derived from the wider area of openings than case B.

2.4 THERMAL IMAGES BY INFRARED METHOD

Thermal imager (IR Flex cam, Fluke) was used to get thermal images of the test houses (figure 4). Thermal images were obtained before the sunrise in early March, heating season. From the images, we tried to find out the cause of air infiltration.

2.5 THERMAL IMAGES OF TEST HOUSE

As shown in figure 5 and 6, heat transfer occurred more in wood posts and beams than walls. Because of high density of wood, in the locations of posts, beams, wood stud, especially in knots, surface temperatures were high. In the case of connection part of timber frame and wood stud infill walls, the gap between frame and infill wall caused air leakage and showed high surface temperature.
Figure 5: thermal images of case A (bedroom)

The siding of bedroom housing in case A was stucco with closed air gap. So, the outer surface of building envelope showed low temperature. Location of wood stud could be seen because of higher thermal conductivity of wood than thermal insulation.

Figure 6: thermal images of case A (living room)

The siding of living room housing in case A was wood panel without air gap. So, the outer surface of building envelope showed higher temperature than stucco siding. And wood posts and beams were exposed to outdoor air. The joint of frame and wall could be seen as well as the location of nails and wood studs.

Figure 7: thermal images of case B

As mentioned above, wood posts and beams exposed to outside have disadvantage for reducing heat transfer. So, covering with supplements like siding on wood posts and beams would be solution. In figure 7, heat loss in the wall of 2nd floor was less than 1st floor.

Figure 8: thermal images of indoor wall surface

Figure 8 shows the thermal images of indoor wall surface. As shown figure 6, heat loss occurred mainly in connection parts of wall and frame.

3 EFFECT OF WALL AND FRAME CONECTION ON AIRTIGHTNESS

From the obtained thermal images, the joints of wall and frame were considered as main path of heat loss in building envelope. So, the gap between wall and frame were covered with PE tape in test house B (figure 7). Then, the air tightness of the building was measured.

Figure 9: covered gap between wall and frame
Measured air tightness of test house B is shown in table 2. Decrease of ACH50 was about 0.5 h⁻¹.

<table>
<thead>
<tr>
<th>Table 2: Air change rate (ACH50)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Control</td>
</tr>
<tr>
<td>Covered gaps</td>
</tr>
</tbody>
</table>

With considering economic worth of difference in airtightness, building energy efficiency was evaluated with CE3 (Construction Energy Efficiency Evaluation) program. Annual heating energy saving from airtightness 0.5h⁻¹ was 3.8 kWh per 1 m² floor area. This is equivalent with the energy of 0.38 L of oil for heating. In case of test house ‘B’, 72 L of heating oil can be saved annually.

4 CONCLUSIONS

From thermal images the joints between wall and timber frame could be the path of heat loss. This defect can be the cause of air leakage in building envelope. In this case, it induced 0.5⁻¹ of ACH50 in building envelop.
EARTHQUAKE RESPONSE ANALYSIS OF 2-STORY TIMBER FRAMES WITH PASSIVE CONTROL SCHEME BY DETAILED FRAME MODEL

Kazuhiro Matsuda¹, Kazuhiko Kasai², Hiroyasu Sakata³

ABSTRACT: This paper proposes an accurate, member-by-member analytical model for timber structures having energy dissipation walls and/or plywood shear walls. Various member joints are modeled by using nonlinear spring elements whose properties derived from numerous test results, and model’s schemes are described in detail. The analyses were found to reproduce both local and global responses obtained from cyclic loading tests and shaking table tests of a variety of one-story mult-span timber frames. Moreover, difference between single wall and linked walls and difference of seismic response by variation of damper are discussed.

KEYWORDS: 2-Story Wooden House, Passive Control, Frame Model, Joint, Seismic Response Analysis

1 INTRODUCTION
In order to mitigate the earthquake response and damage of wooden houses, an energy dissipation wall was developed (Kasai and Sakata et al. 2005) and a lot of experimental studies using the wall were carried out, such as dynamic cyclic loading tests of the energy dissipation walls, shaking table tests of 1-story and 2-story wooden frames (Matsuda et al. 2007), and so on. However, analytical studies need to be carried out to develop more effective energy dissipation wall and to propose the design method. In particular, a framed analysis is effective to study their local behavior. Therefore the objective of this study is to propose an accurate, member-by-member analytical model for timber structures having energy dissipation walls and/or plywood shear walls. Accurate, framed analytical models for the wooden energy dissipation wall with damper and plywood shear wall were proposed (Matsuda et al. 2011). In this study, the frame model will be expanded to be able to apply to 2-story timber structure. Also the accuracy of the analytical framed model will be confirmed by the comparison between the analytical results and shake table test of 2-story timber structure. In addition, the energy absorption performance of damper by the wall configuration will be discussed.

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2 ANALYTICAL FRAME MODEL OF 2-STORY TIMBER STRUCTURE
2.1 FRAME MODELS OF ENERGY DISSIPATION WALL AND PLYWOOD SHEAR WALL
Accurate, framed analytical models for the wooden energy dissipation wall with damper and plywood shear wall were proposed (Matsuda et al. 2011). The framed analytical model was constructed by using many kinds of nonlinear springs whose properties are derived from the test results of the joint. The joint spring between column and horizontal member consists of three types of spring, axial, rotational and shear spring. The frame models were able to duplicate

![Figure 1: Detail of Frame Model](attachment:image.png)
not only global behavior but also local behavior with a high degree of accuracy in many tests of 1-story structure. A seismic response analysis software, PC-ANSR, was used. The method to create the detailed frame model is able to apply to 2-story timber frame basically. Differences from 1-story are, elongating the holddown bolt length of 2nd floor wall and obtaining reaction force of the holddown bolt from 1st floor column (see Figure 2). Because each members are modeled individually, the modification points are minimalized.

2.2 HYSTERESIS RULE OF BOLT BETWEEN 1ST AND 2ND FLOOR

The restoring force characteristic of the bolt includes two effects, axial deformation of bolt and deformation of metal. In the case of energy dissipation wall, only the axial deformation of bolt was changed since the deformation of metal was exceedingly small. When the bolt behaves in elastic, the axial stiffness of the bolt is estimated by axial stiffness which is calculated by tightening length $L_B$, cross section area $A_B$ and Young’s modulus $E_B$ (Matsuda et al. 2011). Additionally, in this study, axial force of bolt was smaller than 55kN. The behavior of the holddown metal which is applied to the plywood shear wall (Table 1, HD), is affected to a large degree by deformation of metal. In the case of 2nd floor HD metal, same HD metal is used for reaction force. Therefore the model of the bolt is estimated that flexibility for 1st floor is doubled. In the result, the models of the bolt of 2nd floor are estimated as Figure 3.

3 COMPARISON BETWEEN TEST AND ANALYSIS

3.1 OUTLINE OF 2-STORY FRAME MODEL

In order to confirm the accuracy of the 2-story frame model, the comparison of shake table test (Sakata et al. 2008) and seismic response analysis will be discussed. Figure 4 indicates the test specimen. Only center plane of the specimen is substituted to frame model since four-cornered columns hardly have horizontal force. The letters of the model name indicate the kind of structural element, W means wood panel, V means viscoelastic damper, F means friction damper, - means no wall and / means border of between 1st and 2nd floor. Figure 5 illustrates the examples of frame model. There are 1.713ton masses on 2nd floor beam and 1.917ton masses on 1st floor beam. The mass ratio of the 2nd floor to the 1st floor is 0.9, assuming the heavy roof and house where the area of the 1st floor is equal to that of the 2nd floor. Stiffness proportional damping 0.5% is used. Acceleration record which was measured on the specimen’s sills during shake table test is used for the analysis and the records have been adjusted to JMA-Kobe 0.6g. But for -VW-/VW-VW and -FW-/FW-FW JMA-Kobe 0.83g is used since their strength and stiffness are higher than the other specimens. Time step $\Delta t$ is 0.001 sec and total analysis time is 10 sec (10,000step).

3.2 COMPARISON BETWEEN TEST AND ANALYSIS OF GLOBAL BEHAVIOR

Figure 6 shows the comparison of test result and analytical result in both global and damper behavior. As for -1.6W-/W-W, although the maximum displacement corresponds each other, there is large error in the positive side. The reason is that the effect of large deformation of the negative side on positive side is not

---

**Table 2: Joint Types**

<table>
<thead>
<tr>
<th>ST (common specification)</th>
<th>SC2</th>
<th>HD</th>
</tr>
</thead>
<tbody>
<tr>
<td>FC5x2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FC5x1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FC3x2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

![Figure 3: Modelization of Axial Spring of Bolt](image-url)

![Figure 4: Shake Table Test of 2-Story Wood Frame](image-url)
considered in this analysis. In the case of specimens which have energy dissipation wall in each floor, the test results and analysis results correspond each other with high accuracy.

3.3 COMPARISON BETWEEN TEST AND ANALYSIS OF LOCAL BEHAVIOR

Figure 7 shows the comparison of test result and analytical result of local behavior. The rotational angle of local behavior shown in upper stage of Figure 7 is duplicated with high accuracy since the rotation angle tends to follow story drift angle. In the case of center capital of 1st floor, rotational resistance is strong since the joint has two metals (FC3x2 and holddown bolt) in both sides, therefore the rotational angle is smaller than the other joints. The analysis result is able to duplicate this phenomenon.

In the case of axial displacement shown in center stage of Figure 7, the analytical displacement of capital part is smaller than the test result. Since the analysis is not able to duplicate axial displacement which the rotation provokes. However the other feature is similar between the analysis and the test.

In the case of also axial force of bolt shown in lower stage of Figure 7, the analytical result duplicates the test result nearly. In particular, the analysis result is able to duplicate axial force of bolt caused by rotation. As observed above, the frame model is able to duplicate the shake table test of 2-story timber frame in terms of not only global behavior but also local behavior with a high degree of accuracy.

4 EFFECT OF LAYOUT OF WALLS

4.1 EXAMINATION OBJECT

Although the check configuration of structural wall was adopted to the specimen for the shake table test, vertically continuous configuration of structural wall is often adopted to the real houses. In addition, in the case of seismic retrofit or not continuous column vertically, the reaction force of the holddown bolt of 2nd floor is obtained from a just below beam. Therefore the energy absorption performance of these cases will be estimated by using the detailed frame model. In sequence, these model are called Specimen Model, Continuous Wall Model and Beam Reaction Model.
4.2 COMPARISON OF GLOBAL AND LOCAL BEHAVIOR

As for the specimen which has viscoelastic damper, Figure 8 shows the relationship between story shear force $Q$ and story drift $\Delta u$ in each model. The stiffness of Continuous Wall Model decreases about 20% from Specimen Model in each floor and maximum deformation of 2nd floor increases about 50% from
Specimen Model. The stiffness of Beam Reaction Model decreases about 8% from Specimen Model in each floor and maximum deformation of 2nd floor increases about 25% from Specimen Model. Axial force distribution of column and relative displacement of joint at peak strength of 1st floor are shown in Figure 9. When the 1st floor reached a maximum strength, 2nd floor almost reached a maximum strength since the contribution of 1st deformation mode was extremely high.

In the case of Figure 9 (b) Continuous Wall Model, the axial force of 1st floor column increases because the axial force of 2nd floor column is transferred to just below column. Therefore the method to select metal type has been established to consider the configuration of structural wall by Japanese building standard low. Meanwhile, there is no rule to consider the configuration of energy dissipation wall. A method to select specification of the joint considering the configuration of energy dissipation wall should be discussed.

In the case of Figure 9 (c) Beam Reaction Model, at the capital of leftside center column of 1st floor, axial tension force is increasing, and as a result, relative displacement is increasing at the point. Because 2nd floor column and 1st floor beam are connected solidly, the beam is bending and the deformation is concentrated at the 1st floor capital which has weak connection relatively.

4.3 PROPOTION OF LOCAL DEFORMATION

The deformed state and definition of symbol are illustrated in Figure 10. Rocking distortion angle of each floor $\theta_{R1}$ and $\theta_{R2}$ are calculated by Eq. (1) and (2).

$$\theta_{R1}, \theta_{R2}$$

Figure 10: Deformed State and Definition of Symbol
Table 2: Proportion of Local Deformation to Story Drift

<table>
<thead>
<tr>
<th></th>
<th>Specimen Dissipation Wall Using Viscoelastic Damper</th>
<th>Specimen Dissipation Wall Using Friction Damper</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Continuous Wall Model</td>
<td>Beam Reaction Model</td>
</tr>
<tr>
<td></td>
<td>1F-L 1F-R 2F-C</td>
<td>1F-L 1F-R 2F-C</td>
</tr>
<tr>
<td>Bending Deform. of Ground sill</td>
<td>0.095 0.026 0.010</td>
<td>0.090 0.032 0.020</td>
</tr>
<tr>
<td>Axial Deform. of 1st Floor Column Base</td>
<td>0.105 0.010 0.041</td>
<td>0.182 0.139 0.166</td>
</tr>
<tr>
<td>Axial Deform. of 1st Floor Capital</td>
<td>0.137</td>
<td>0.069</td>
</tr>
<tr>
<td>Axial Deform. of 2nd Floor Column Base</td>
<td>0.136</td>
<td>0.010</td>
</tr>
<tr>
<td>Rocking Deform.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Damper Deformation</td>
<td>0.630 0.603 0.553</td>
<td>0.558 0.497 0.561</td>
</tr>
<tr>
<td>Other</td>
<td>0.172 0.266 0.176</td>
<td>0.252 0.119 0.152</td>
</tr>
<tr>
<td>Total</td>
<td>1.000 1.000 1.000</td>
<td>1.000 1.000 1.000</td>
</tr>
</tbody>
</table>

\[
\theta_{r1} = \frac{(\delta_{s,L} - \delta_{s,R}) + (\delta_{b1,L} - \delta_{b1,R})}{L}
\]

(1)

\[
\theta_{r2} = \theta_{r1} + \frac{(\delta_{c1,L} - \delta_{c1,R})}{L}
\]

(2)

Where, 1st parenthesis of Eq. (1) is contribution of bending deformation of ground sill, 2nd parenthesis of Eq. (1) is contribution of axial deformation of 1st floor column base, 1st parenthesis of Eq. (2) is contribution of axial deformation of 1st floor column, 2nd parenthesis of Eq. (2) is contribution of axial deformation of 1st floor capital, 3rd parenthesis of Eq. (2) is contribution of axial deformation of 2nd floor column base.

Assuming this energy dissipation wall is composed of series system of 3 springs, spring of rocking deformation, spring of damper deformation and the other spring, the contribution of each springs at peak story drift of 1st floor are listed in Table 2. The contribution of rocking deformation is calculated by Eq. (1) and (2). The contribution of damper deformation is obtained from ratio of 3 times of damper deformation to story drift. The other contribution is obtained from subtract the 2 contributions from 1.

5 CONCLUSIONS

A detailed frame model of 1-story timber structure which is proposed by Matsuda et al. (2011) is expanded to be able to apply to 2-story timber structure. Major finding are:

- Differences from 1-story are, elongating the holddown bolt length of 2nd floor wall and obtaining reaction force of the holddown bolt from 1st floor column. Because each members are modeled individually, the modification points are minimized. The frame model is able to duplicate the shake table test of 2-story timber frame (Sakata et al. 2008) in terms of not only global behavior but also local behavior with a high degree of accuracy.

- It is confirmed that the check configuration of energy dissipation wall is effective for energy absorption performance. And it is confirmed that controlling the rocking deformation is very significant for this energy dissipation wall.

REFERENCES


WOOD PRODUCTS FOR THE ITALIAN CONSTRUCTION INDUSTRY – AN LCA-BASED SUSTAINABILITY EVALUATION

Nadia Villa¹, Francesco Pittau², Enrico De Angelis³, Giuliana Iannaccone⁴, Giovanni Dotelli⁵, Luca Zampori⁶

ABSTRACT: First, from our point of view, LCA is definitively the most effective tool to measure environmental impacts. Second, in order to compare different construction products (how much wood and other products are green) we have to assess them as part of the whole Life Cycle Impact of the buildings they will be part of. Nevertheless, to do this, we need reliable effective data. Life Cycle Assessment, in fact, is a bit more complicated, for wood: tree species, yield of forest, time from planting trees to log harvest, harvesting procedure, length and width of forest roads or total area and land use have a strong influence on the environmental results. Other questions concern local electricity profile and national transport policies, the local procedures for wood wastes (disposal or energy recover) at the end of life of a building and the same choice of the functional unit for the many impacts. For these reasons, standardized data are not yet available, as well as average world-wide data, from the most diffused LCA databases, are very likely different from real product characteristics.

The aim of the study is to report the results of the assessment (LCA method) of an Italian “short chain” manufacturing unit for wood-based construction products. To get to this point, we had to face all the methodological questions for the assessment of the Life Cycle of a wood product, from data gathering to the choice of the most proper end of life scenario. The first result is a methodological answer: how to measure actual carbon storage and positive environmental performance of wood. The second is a reference for local-wood based products and a comparison of our results with standard EU and OECD, in order to quantify “short chain” consequences on environmental impacts.

KEYWORDS: Life cycle analysis, Wood, Forest, Product LCA.

1 INTRODUCTION

Nowadays more and more care on environmental aspects connected to construction sector is gathering, considering also that buildings are responsible of more than one third of environmental emissions. Many efforts in this direction have been performed, but going through the whole building to its elements, many other efforts have to be taken into account, such as the choice of construction materials and technological services that depends on the usage of the building, followed by cost, availability and architectural needs. Furthermore, materials and services involved in construction keep an environmental sustainability that designer cannot neglect.

The material able to gain the most positive environmental profile is wood, thanks to its carbon neutrality that brings a good CO₂ storage potential. Moreover, market share is rapidly increasing thanks to a renewed interest in wood based products and their outstanding mechanical properties. Italian market is also supported by industries that, following this new environmental trend, try to reach a “short chain”, even near “zero kilometer”, by concentrating production, as far as possible, in their own country.

Besides, for the production of wooden products, energy for cutting, smoothing and planning is the only resource used: in this scenario energy balance for a wooden building becomes particularly sustainable.

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Literature studies [3] report that 1 m³ of wood used saves about 1.1 t of CO₂ emissions. The results of a Life Cycle Assessment “from-cradle-to-gate” of a CLT panel produced in Italy supports the thesis that stimulates a great exploitation of wood and forests to create building products, in order to promote a sustainable management of natural environment. Wood is proved to be a good choice in place of other traditional building materials, such as concrete and brick.

2 PRODUCTION CHAIN

CLT panel production begins with the supply of raw materials to the sawmill (Factory 1), where logs are cut, and it ends when the axes are stacked, ready to be transferred by lorry to the Factory 2 where the CLT panel is assembled. In Factory 1 there is a great amount of wooden wastes (biomass), that are sold to a heat producing plant. The presence of two different products from a unique process creates a problem of allocation. Since there is not a unique method to divide energy and environmental loads among co-products, and wood is a material that is more sensible to some indicators (i.e. GWP), a great importance, within the case study, is given to the deepening of the different methodologies purposed in standard UNI EN ISO 14044 [7]:

- expansion of the system boundaries: a unique product is considered (CLT panel), because biomass is entirely used to produce heat, in replacement of fossil fuel usage;
- mass allocation: environmental loads are divided between the principal product (wood axes) and the co-product (biomass) following mass relationships; 54% attributed to CLT panel and 46% to biomass.
- economic allocation: loads are distributed between the principal product and the co-product on the basis of their Italian market values. 98.7% of environmental loads to CLT panel and the rest to biomass.

3 METHODOLOGY

The functional unit chosen is 1 m³ of CLT panel. Primary data for materials, transports, water and electrical energy consumptions were gathered directly from the two factories. The Ecoinvente Database was used for other data. SimaPro 7.1 software was used and for the impact analysis the indicators adopted were:

- GWP100: global warming potential over 100 years for the evaluation of emissions and savings of CO₂eq in atmosphere [kg CO₂eq];
- CED: cumulative energy demand to calculate the renewable and non renewable energy consumptions of the process considering also feedstock energy [MJ eq].

3.1 WOOD

The wood species used to produce CLT panel in Italy is douglas and it is supplied “at forest road”. Cutting, pruning, rotation cycle of the forest and sowing are not considered in the analysis because these operations vary with the changing of the kind of forest management. Wood data are from Ecoinvent.

3.2 ELECTRICITY

Electrical energy consumption for the production of CLT panels were quantified thanks to primary data and they are referred to the Italian energy system, considering national performances and the various sources of origin (Table 1).

Table 1: Electricity consumptions

<table>
<thead>
<tr>
<th>Production chain</th>
<th>[kWh/m³]</th>
<th>[kWh/yr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fact.1 Sawmill</td>
<td>30.96</td>
<td>26'932.00</td>
</tr>
<tr>
<td>Drying</td>
<td>30.96</td>
<td>26'932.00</td>
</tr>
<tr>
<td>Parting</td>
<td>30.96</td>
<td>26'932.00</td>
</tr>
<tr>
<td>Factory 2 Splicer</td>
<td>60.29</td>
<td>120'575.65</td>
</tr>
<tr>
<td>Planer</td>
<td>47.48</td>
<td>94'953.33</td>
</tr>
<tr>
<td>Gluer</td>
<td>0.03</td>
<td>60.00</td>
</tr>
<tr>
<td>Press</td>
<td>6.78</td>
<td>13'564.76</td>
</tr>
<tr>
<td>Pantograph</td>
<td>23.36</td>
<td>46'723.06</td>
</tr>
<tr>
<td>Compressed air</td>
<td>15.19</td>
<td>30'385.06</td>
</tr>
<tr>
<td>Crane</td>
<td>1.90</td>
<td>3'798.13</td>
</tr>
<tr>
<td>Total</td>
<td>185.99</td>
<td>336'991.99</td>
</tr>
</tbody>
</table>

3.3 TRANSPORTS TO THE FOREST

The length of the route considered in the case study is about 53 km, using lorry of 12,5 t and 27 t, Euro 4. On the basis of the fact that forests are more widespread abroad than in Italy, another hypothesis has been done, in order to quantify, in term of environmental impact, the difference exiting between 53 km (Italy) and 600 km (abroad), using the same transports for logs.

4 WASTE MANAGEMENT

The wood wastes produced, that are exploitable through the reuse, recycling or energy recovery, make wood a multifunctional material, also thanks to the careful management (sorting and separation) producing also a low quantity of garbage. About half of the input wood becomes waste due to the presence of failure and of the cutting needs; in the analysis wood wastes are supposed to be sold to produce heat (non becoming a co-product), whilst wastes that are unsuitable to produce heat, because some glue still remains in them, are conducted to a incineration plant, where the process efficiency can guarantee an environmental recovery. In the waste disposal phase of all the three scenarios settled, the wood that is incinerated or used for internal
heating of the Factory 2 allows a heat recovery in replacement of other fossil fuels.

5 RESULTS

5.1 GLOBAL WARMING POTENTIAL: GWP

CO$_2$eq emissions related to the production of 1 m$^3$ of CLT panel are shown in Table 2.

Table 2: GWP emissions for 1 m$^3$ CLT panel

<table>
<thead>
<tr>
<th>Allocation method</th>
<th>kg CO$_2$eq total</th>
<th>kg CO$_2$eq replacement fossil fuels</th>
<th>kg CO$_2$eq technology inputs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion</td>
<td>-1'670</td>
<td>-886</td>
<td>+243</td>
</tr>
<tr>
<td>Mass</td>
<td>-936</td>
<td>-124</td>
<td>+214</td>
</tr>
<tr>
<td>Economic</td>
<td>-958</td>
<td>-124</td>
<td>+242</td>
</tr>
</tbody>
</table>

The wood coming from managed forests brings a CO$_2$eq credit near 895 kg CO$_2$eq per 1 m$^3$ of raw material in input to the sawmill [source Ecoinvent]; through subsequent processing, the quantity of carbon dioxide stocked is reduced due to the emission generated by process machines and to the wastes disposed. 1 m$^3$ of CLT panel is made from 3.29 m$^3$ of input logs.

Figure 1-3 show positive and negative CO$_2$eq flows, in input and in output, following the three allocative scenarios purposed. The GWP$_{100}$ values of technological inputs and biomass vary in function of the allocative scenario used. In “expansion of the system boundaries” scenario, the final GWP$_{100}$ result is given for a large portion by fossil fuels savings.

5.2 CUMULATIVE ENERGY DEMAND: CED

CED analysis is founded on energy resources use, subdivided into two macro-categories:

- non renewable energy (fossil and nuclear);
- renewable energy (wind, solar, geothermal, hydroelectric and from biomass).

In the case study renewable energy is almost entirely attributed to the wood feedstock energy that is stocked in the CLT panel, while the non renewable one is related to electric energy consumptions of machines (without considering “expansion of the system boundaries” scenario): Table 3.
Table 3: CED for 1 m$^3$ CLT panel

<table>
<thead>
<tr>
<th>Allocation</th>
<th>Renewable energy [MJ$_{eq}$]</th>
<th>Non renewable energy [MJ$_{eq}$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion</td>
<td>33’461</td>
<td>-11’531</td>
</tr>
<tr>
<td>Mass</td>
<td>18’158</td>
<td>801</td>
</tr>
<tr>
<td>Economic</td>
<td>19’293</td>
<td>1’356</td>
</tr>
</tbody>
</table>

Renewable energy requirement is about 20’000 MJ$_{eq}$, whilst the benefit gains for “expansion of the system boundaries” scenario is ~ 33’000 MJ$_{eq}$. From the difference between the total renewable energy attributed to “expansion of the system” (~ 33’000 MJ$_{eq}$) and the total renewable energy of the other allocation methods (~ 20’000 MJ$_{eq}$), a result of ~ 13’000 MJ$_{eq}$ is obtained; this value, if multiplied for an efficiency of 0.8, generates ~10’000 MJ$_{eq}$ that is the contribution of non renewable energy of “expansion of the system” and indicates the energy bonus (negative value) due to the fossil fuel not exploited and left in nature.

6 EU DATA COMPARISON

At a practical level, given the lack of a unique and consolidated methodology, that can be shared and applied to a broad range of products, and from the evaluation of other researches, it emerges that the analyzed phases, the hypothesis adopted and the data quality in LCA studies are not always well and completely explained. Some authors show their evaluation on the basis of their line of research, other evaluate only methodological aspects, and other again focus only on disposal choices or on allocation procedures [5]. The building of a complete product LCA scenario seems to be difficult to realize due to the heterogeneity and the variability of the analysis conducted in this field.

The thread that make Italian LCA on CLT panel similar to other literature evaluation on wood is the best environmental profile, if compared to other material that guarantee the same performances, thank to the low requirements of wood of non renewable energy to product the good and to the CO$_2$ storage. Particularly, LCA on an Italian chipboard (Scimia [12]) gives a greenhouse benefit of -650 kg CO$_2$ per m$^3$, value that is similar to the one obtained for the CLT panel analyzed (Paragraph 5.1).

Rivela [10,11] makes a Characterization and Damage Assessment of a MDF and a particleboard panel, without evaluating GWP$_{100}$ but both the studies quantify in detail energy consumption of machines, that results to be for MDF panel twice the requirement needed for CLT panel (Paragraph 5.2) and for particleboard similar to CLT (without considering dryers consumptions).

7 CONCLUSIONS

The “from-cradle-to-gate” LCA of a CLT panel brings to light not only methodological problems, but also other issues connected to wood production chain, i.e. the different kinds of resource uses and the various energy involved in the process.

From the methodological point of view, following UNI EN ISO 14040-44 [6,7], the first approach purposed provides, for multifunctional systems, the “expansion of system boundaries”; in the case study only the panel is considered as a product, whereas the waste biomass is addressed to replace the fossil fuels not used. However this choice implies some randomness connected to the real use of this biomass, that cannot be neglected. Biomass can be used to replace fossil fuels, but it can also become the raw material for the production of chipboard panels or any other different kind of glued panel. These criticalities lead to the conclusion that, although the “expansion of the system” is the standard suggested choice, the use of mass or economic allocation procedures cannot be neglected.

Given the sensibility of wooden products to the widespread GWP$_{100}$ indicator, it is strictly suggested that a transparent LCA study should provide at least two different allocation method.

From the point of view of the different kinds of resource uses and the various energies involved in the process, the LCA of the Italian CLT panel has underlined that the “short chain” exploits the local productivity, with clear benefits either at macroeconomic and at environmental level, thanks to the reduced distances between the unit processes.

It is certainly true that for certain traditional building technologies the activation of “short chain” production is objectively difficult to promote, due to the difficult retrieval of the natural resource needed to realize the chief products (concrete, brick, steel, etc.), but wooden building technique, bound only to forest resources, seems to have a great potential thanks to the widespread presence of forest areas, often controlled by a certified and sustainable forest management. The increasing of the stock of wooden products has therefore the effect of reducing emissions due to CO$_2$ storage in the material itself, to the lower quantity of energy required (significantly lower than any other material) and to the further gain provided by the usage of waste biomass.

ACKNOWLEDGEMENT

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AN EFFECTIVENESS OF SEISMIC PERFORMANCES ASSESSMENT FOR SUSTAINABLE TIMBER HOUSES ON MICROTREMORS MEASUREMENT

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ABSTRACT: The microtremors measurement was done, and limit proof stress design was done about the timber building where it got natural frequency. Statistics processing was done about the relations of the natural frequency of that building and the rigidity or the proof stress which could be got from the limit proof stress calculation of the building. Then, the rigidity or the proof stress of the building were estimated from the natural frequency. Moreover, some full-scale vibration experiments were compared with a natural frequency and a result of a limit proof stress calculation, and that reliability was ascertained. A building could be diagnosed in the comparatively simple method of the microtremors measurement, and the rigidity and the proof stress of the building could be estimated. The quantitative evaluation by the microtremors measurement becomes a reliable explanation to the client.

KEYWORDS: Sustainable timber house, Microtremors measurement, Limit strength calculation, Retrofitting

1 INTRODUCTION
It is the policy which sets the earthquake-proof-ization rate of the wooden building at 90% by 2015 in Japan. But, the earthquake-proof-ization rate of the wooden building is only 75%, and 11,500,000 houses of circa are in the wooden building which must have earthquake-proof-ization at 2005 years. Therefore, a rapid earthquake-proof diagnosis is necessary. And microtremors measurement is proposed by this thesis as one technique of the quantitative and rapid earthquake-proof diagnosis. The seismic performances assessment for the sustainable timber houses based on the microtremors measurement is evaluate. Because a seismic diagnosis and a seismic reinforcement don't proceed, a quantitative explanation to the client is necessary for the existent timber building by the simple seismic diagnosis. A quantitative result can be seen by measuring the existent timber buildings before and after the improvement by the microtremors measurement. Because it is decided by static loading examination of the structure element, a difference from the dynamic characteristics can't be reflected clearly in the design. Therefore, because quantitative verification is possible, microtremors measurement can become effective vibration appraisal method.

2 METHOD OF MEASURINGS
Our research group has measured timber structures by a portable vibration measurement machine of SPC-51 with the six velocity-meters of VSE-15D of the servo-mechanism. We set up the six sensors on the first and second floors at the north-south and east-west directions. The full range of measuring is 100 [mm/sec], and the sampling frequency is a Rene 100 [Hz]. we compute the natural frequency and the damping ratio by calculation with a fast Fourier transform with the steady 1024 points data which we have got at measuring. The second floor spectrum was divided by the first floor spectrum to consider ground vibration. In the fast Fourier transform, we use ten times smoothing. We calculated damping ratio by the half power method.
3  METHOD OF SEISMIC EVALUATION

The structure is evaluated by a limit strength calculation. The limit strength calculation is an evaluation method by columns and bearing walls which building strength in each story shear force-drift and stiffness. This calculates transformation and does building evaluation. It is used for the evaluation of the Japanese traditional architecture. By a result of microtremors measurement and a result of a limit strength calculation in the sixteen houses, it is studied that the relation to the natural frequency and the stiffness or the maximum shear force. The relation to the natural frequency and the stiffness is shown figure 1. The relation to the natural frequency and the maximum shear force is shown figure 2.

![Figure 1: Relation to Natural Frequency and Stiffness](image1)

![Figure 2: Relation to Natural Frequency and Maximum Shear Force](image2)

3.1  FOR FULL-SCALE SHAKING TABLE TESTS

Some results of full-scale shaking table tests are applied this method of seismic evaluation. It is shown figure 1. The black line means a result of full-scale shaking table tests, the blue line means a limit strength calculation, the bi-liner line means a result of microtremors measurement. A result of a limit strength calculation and a microtremors measurement grasps actual action.

3.2  FOR RETROFITTINGS

Some results of retrofitting buildings are applied this method of seismic evaluation. It is shown Case 1-3. The bi-liner line means a result of microtremors measurement. A result of microtremors measurement reach to estimate actual action in the earthquake. A

![Figure 3: Experimental and calculated P-D Curve](image3)

4  CASE 1 – M-HOUSE

The M-house is constructed in 1930. It is a typical “Min-ka” type of the Japanese traditional architecture. The Plan and elevations of the M-house are shown figure 4. The Calculated P-D Curve of the M-house before retrofitting is shown figure 5. The Calculated P-D Curve of M-house after retrofitting is shown figure 6.

The M-house before retrofitting does not have seismic performance for severe earthquake. The story drift is 8 cm for building code earthquake. The M-house after retrofitting has seismic performance for severe earthquake. The story drift is 20 cm for severe earthquake. The story drift is 4 cm for building code earthquake.

The seismic performance of the Japanese traditional architecture is able to estimate on the microtremors measurement.
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Figure 5: Calculated P-D Curve of M-HOUSE Before Retrofitting

Figure 6: Calculated P-D Curve of M-HOUSE After Retrofitting

5 CASE 2 – K-STATION

The K-station is constructed in 1926. It is a Japanese traditional architecture. The Plan and elevations of the K-station are shown figure 7. The Calculated P-D Curve of the K-station before retrofitting is shown figure 8.

The K-station before retrofitting does not have seismic performance for severe earthquake. The story drift is 10 cm for building code earthquake.

The seismic performance of the Japanese traditional architecture is able to estimate on the microtremors measurement.

The K-station will be retrofit in the not-too-distant future.

6 CASE 3 – S-HOUSE

The S-house is constructed in 1918. It is a huge type of the Japanese traditional architecture. The Plan and elevations of the S-house are shown figure 9. The Calculated P-D Curve of the S-house before retrofitting is shown figure 10.

The seismic performance of the Japanese traditional architecture is able to estimate on the microtremors measurement.

The S-house will be retrofit in the not-too-distant future.

Figure 7: Plan and Elevations of K-STATION

Figure 8: Calculated P-D Curve of K-STATION

Figure 9: Plan and Elevations of S-HOUSE
7 CONCLUSIONS

By vibration performance, the seismic performance is estimated. The rise of the natural frequency could be confirmed after seismic retrofitting in both ways. Because the client could be shown value after seismic retrofitting by microtremor measurement, it was possible to have consented to a client in any case. We were able to propose effective seismic retrofit based on microtremor measurement of timber structure.

The seismic performances of the Japanese traditional architectures are able to estimate on the microtremors measurement.

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STRAIN ANALYSIS IN DRIED GREEN WOOD: EXPERIMENTATION AND MODELING APPROACHES

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ABSTRACT: In this work, the strains due to the natural drying of green wood slice are investigated by experimental and numerical methods. The shrinkage of green slice in Douglas is carried out in a room test where the environmental conditions are constant. The mass loss is measured according to a high-sensitivity balance until the internal moisture content of 11\%. According to the marked posts on the slice, the displacements are recorded with a video camera allowing the evaluation of the Point Saturation Fibre. In order to take into account the stress relaxation coupled with hygrothermal effects, mechano-sorptive elastic behaviour, including shrinkage-swelling effects with orthotropic properties in the radial and transversal directions, is considered. The results obtained by experimental approach are compared with numerical modelling to predict the shrinkage process and the efficiency of the proposed model is proved.

KEYWORDS: Green wood slice, drying process, strain analysis, finite element method

1 INTRODUCTION

The natural or artificial shrinkage due to drying is commonly responsible of crack appears in wood pieces. These defects are even more marked on the logs pieces cut up and stored in the open air during a long time. In this fact, it is essential to understand the different processes that lead to these phenomena in the green woods. In the literature, the deformations and the evolution of drying induced stresses in wood are studied based on analytical methods of elastic response [1] [2]. In the same time, the experimental approach [3] [4] and a finite element model which takes into account the dependence of mechanical properties of wood during drying phase has been investigated [5] [6]. Also, several authors have presented a model of drying that permits evaluation of moisture content distribution in wood during the drying rate periods with the moisture content at the body surface reaches the fibre saturation point (FSP) [5]. In this work, the radial and tangential strains due to the natural drying of green wood slice are studied according to analytical, experimental and numerical methods.

The first part of the paper recalls the experimental setup composed of a green wood slice in Douglas, the balance and a video camera. The device is based on the mark tracking method composed of an acquisition system data recording the displacement of black target posted on the wood slice during the drying phase. The second part details the analysis of experimental results; in this case, the evolution of moisture content versus time is posted in order to obtain the PSF. The radial
and transversal strains are computed according to an analytical approach based on a mathematical model coming from cylindrical strain calculations. In the last part, this analytical method is coupling with the swelling-shrinkage tensor. The finite element calculation is limited, in this paper, at an elastic behavior integrating a mechano-sorptive effect.

2 EXPERIMENTAL RESULTS

2.1 EXPERIMENTAL SETUP

The experimental protocol is based on a green wood slice in Douglas fir with a thickness of 30mm just peeled, Figure 1. The sample is initially conditioned in water in order to assure its saturation. The preparation of the specimen consists on draw black marks according to a polar reference centred at the sample heart. In a second step, this sample is placed in the experimental chamber in which the temperature of 22°C and a relative humidity of 33%RH are maintained constant. The sample is placed on a precision balance in order to measure the mass evolution during the drying process, Figure 2. In the same time, black marks allow performing a marking tracking protocol in order to record displacements of these marks using the videometric technology.

Figure 1: Experimental device

2.2 EXPERIMENTAL RESULT ANALYSIS

2.2.1 Moisture content evolution

The time moisture content, shown in Figure 3, is characterized by a first linear evolution corresponding to the free water migration. At the end of test, we can observe the moisture content equilibrium around 8%. While shrinkage swelling effect start below the fiber point saturation, it is necessary to detect this point.

Figure 3: Moisture content evolution versus time.

Figure 2: Green wood slice, black marks and notification of radius and rind.

Figure 4: Fiber point saturation detection.
Assuming that the fiber point saturation gives a non-linearity in the time moisture content curve, a zoom of the figure 3, highlights a critical moisture content value of 30%, see figure 4.

### 2.2.2 Referential polar coordinate

In order to identify each black mark, Figure 2 proposes a notification by the intersection between radiuses and rings. The radial strain is show by figure 5 a, where $\Delta n_{1-2}$ is the length between $M_1$ and $M_2$. The tangential strain is defined by figure 5b, where $\Delta \theta_{n-2}$ is the distance between $M_1$ and $M_2$ that $\vec{r}$ is their symmetric axis. $r_d$ is the distance between the referential origin and the projection of the two points on $\vec{F}$.

![Referential polar coordinate diagram](image)

Figure 5: Referential discrete derivation. a – radial representation. b – tangential representation.

For example, the mark $M_i'$ traduce the point at the intersection between the radius $R_i$ and the ring $C_j$. Let us note that radial and tangential strains are defined at the middle point comprised between $M_i$ and $M_2$, figure 5. With this consideration, the radial strain is calculated at the point $M_2$ corresponding to a point placed in the radius n°2 and between rings 1 and 2. With the same way, the tangential strain is calculated at the point $M_{2-3}$ placed in the ring n°2 and between radiuses 2 and 3.

### 2.2.3 Strain evolution

Figure 6 presents radial strains along radius n°3. This figure shows non-homogeneity of radial shrinkage deformations with higher values for near centre points. According to images of Figure 2, the radius n°6 is placed in a normal wood region while radius n°3 is placed in a tension wood vicinity. This fact is illustrated by differences of strains of external points ($M_{2-3}$ and $M_{1-4}$). These observations could be explained by an heterogeneity of elastic and expansion coefficient properties.

![Radial strain along radius n°3](image)

Figure 6: Radial strain along radius n°3.

Figure 7 shows the radial shrinkage strains plotted for the crown n°3 of the specimen of figure 2. This result illustrates that strain heterogeneity does not dependent on radius but it is sensible to regions. In the same time, the observation of this figure shows us that radial strains decrease with the distance of specimen centre.
3 FINITE ELEMENT APPROACH

3.1 INCREMENTAL LAW AND DRYING MODEL

The coupling between moisture content variations and mechanical behaviour is based on the addition of shrinkage-swelling response and mechano-sorptive behaviour. During drying process, Gril et al. [7] and Husson et al. [11] has putted in evidence one hygro-lock effect affecting elastic and viscoelastic responses. In this study, let us limit our investigation for elastic properties with a time decreasing of elastic modulus with an uncoupling between elastic behaviour and shrinkage process represented by the elastic and shrinkage deformations \( \varepsilon_e \) and \( \varepsilon_w \). In this case, the mechanical model can be assimilated at a Bazant's model according to a hardening behaviour during drying phases [8]. The mechanical behaviour representing the drying model written in terms of strain rate \( \dot{\varepsilon}(t) \) as follow:

\[
\dot{\varepsilon}(t) = C(t) \cdot \dot{\sigma}(t) + \alpha \cdot \dot{w}(t) \quad (1)
\]

The total stress rate tensor is called \( \dot{\sigma} \). \( C(t) \) is the four order compliance tensor adapted for a cylindrical orthotropic properties. In a 2D representation, \( C(w) \) can be defined according to elastic moduli such as:

\[
C(w) = \begin{bmatrix}
\frac{1}{E_R(w)} & -\frac{v_{RT}}{E_R(w)} & 0 \\
-\frac{v_{RT}}{E_R(w)} & \frac{1}{E_T(w)} & 0 \\
0 & 0 & \frac{1}{G_{RT}(w)}
\end{bmatrix} \quad (2)
\]

\( E_R(w), E_T(w) \) and \( G_{RT}(w) \) are the radial, tangential and shear modulus, respectively. \( v_{RT}(w) \) is the Poisson's ratio.

\( \alpha \) is the shrinkage-swelling expanse tensor. In the orthotropic referential, this tensor is diagonal. In this study, we assume a constant tensor in the hygroscopic domain. \( \dot{w}(t) \) is the moisture content rate.

The implementation of the behaviour law, expressions (1), requests its moisture content integration in the global RT base. By analogy with Randriambololona's developments [9], the equation (1) can be solved using the following finite derivation method solving according to Ghazlan approach [10,11]

\[
\Delta \varepsilon(t_n) = \left[ \frac{1}{2} \cdot C(t_n) + C(t_{n+1}) \right] \cdot \Delta \sigma(t_n) + \alpha \cdot \Delta w(t_n) \quad (3)
\]

where \( \Delta \sigma \) and \( \Delta \varepsilon \) are the stress and the strain increment, respectively.

In order to solve equation (3), the following finite element formulation, proposed by Ghazlan, is applied:

\[
K_T \cdot \{ \Delta \dot{u}(t_n) \} = \left\{ \Delta F_{ext}^\sigma(t_n) \right\} + \left\{ F^w(t_n) \right\} \quad (4)
\]

Where \( K_T \) is the tangent matrix defined by using the Jacobian matrix \( B \) connecting displacements and strains and the equivalent compliance tensor present in expression (3) such as:

\[
K_T = \int_{\Omega} B^T \cdot \left[ \frac{1}{2} \Delta C(t_n) + C(t_{n-1}) \right]^{-1} \cdot B \, d\Omega \quad (5)
\]
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\[ \{ \Delta F^e_{\text{ext}} \} (t_n) \] is the increment of nodal force vector induced by the external increment loading and incremental stresses \( \Delta \sigma \). \[ \{ \Delta F^w \} (t_n) \] designates the supplementary moisture content load vector representing the effect of moisture content variations in the time step.

3.2 FINITE ELEMENT ALGORITHM

In order to implement the incremental law (3) in finite element software Castem, the following subroutine is applied. Firstly, the referential state corresponding to the fiber saturation point is fixed. If we ignore growth constraints and relaxation effects after the tree felling, we can impose \( \varepsilon (0) = 0 \) and \( \sigma (0) = 0 \). In the same time, we admit an initial moisture content homogeneity in the disk surface and in this thickness. We note its value \( w_{\text{ini}} \). For each algorithm step, we consider, firstly, that mechanical state is known at time \( t_{n+1} \) and, in the other hand, that moisture content variation is measured by weighing for the time \( t_n \). About moisture content, we assume its homogeneity in the sample.

The compliance tensor \( C (w) \) is updated according to the moisture content \( w (t_n) \). All terms of the equilibrium equation (3) are defined. The tangent matrix (5) is calculated and the balance equation (4) is solved by the finite element process. Mechanical state is updated in terms of displacements, strain and stress tensors. The crack growth initiation is detected by comparing the transverse component of the stress tensor and its critical value.

4 APPLICATIONS

4.1 RADIAL STRAINS

In the numerical process, the Poisson’s ratio is assumed to be constant. The elastic properties of Douglas fir at \( w=12\% \) are given by the Table 1.

<table>
<thead>
<tr>
<th>( E_T^0 )</th>
<th>( E_T^0 )</th>
<th>( G_{GR}^0 )</th>
<th>( v_{GR}^0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1300 MPa</td>
<td>900 MPa</td>
<td>125 MPa</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Also, the shrinkage - swelling coefficients values are given by Guitard [12]: \( \alpha_w = 0.08\% / \% \) and \( \alpha_f = 0.35\% / \% \).

\( \circ \) : experimental measurements
\( \times \) : numerical calculus

Figure 8 presents the comparison between radial strains given by finite element approach and experimental data. In order to clarify visibility, we choose to represent experimental measurements for extreme rings for radius n°3.

Figure 8: a – Finite element mesh of slice. b - Comparison between experimental and numerical radial strains along radius n°3.

We can observe small differences which can be explain by heterogeneity of material and by the existence of growth stresses which could modify material properties.

4.2 FRACTURE INITIATION

The crack growth initiation is observed below 8% of moisture, figure 9. With these
considerations, we can admit that the fracture of the free disk and the localization of the crack growth initiation are induced by the combination of a shear mode configuration added to a tangential tension strain state.

Figure 9: Shear strain mapping at 8% of moisture content.

5 CONCLUSIONS

The strain evolutions have investigated during the drying process in this work. The changes of radial strains versus the distance to the slice centre for different moisture have been studied. Radial strains are computed by defining a local orthotropic referential. During the evolution of moisture content between the FSP level and its critical value inducing the crack initiation (w = 11.48%) the numerical and experimental results illustrate a very good agreement. In the same time, the finite element model allows the localization of the crack initiation during drying process. In the coming work, the fracture analysis will apply and coupling with the viscoelastic model and the drying model in order to study the behaviour of wood material under environmental variations.

REFERENCES

A NUMERICAL AND EXPERIMENTAL STUDY OF STRESS
AND CRACK DEVELOPMENT IN KILN-DRIED WOOD

Finn Larsen¹, Sigurdur Ormarsson²

ABSTRACT: Numerical and experimental investigations were carried out on well defined log-disc samples of Norway spruce consisting of both heartwood and sapwood, with the aim of gaining more adequate knowledge of stress and fracture generation during the drying process. Use of thin discs of a log enabled a well-controlled and simplified drying history of the samples to be obtained. Experiments supported by the numerical simulations showed the heartwood to dry below the fibre saturation point (FSP), much earlier than the sapwood, and thus to start shrinking at a much earlier stage. The variation in the moisture histories for different parts of a cross section was the main reason for the stress development during the drying process.

KEYWORDS: Aramis, wood drying, FE-simulation, drying strains, drying stresses

1 INTRODUCTION

Wood used for structural purposes and for furniture can develop considerable stresses during the drying process from green condition down to a moisture content, beneath FSP, needed for further handling. Drying of wood is done either by natural drying or by kiln drying, which is when the wood is heated and dried under controlled climate conditions. Solid wood stiffness properties are strongly affected by the moisture content (MC) and of the temperature. The stiffness properties increase when MC is reduced, while the same properties reduce as the temperature increases. The drying history is likewise complex through drying occurring faster above than below the FSP, which shown by [19] and by [10]. Further numerical investigations of drying above the FSP have been carried out by [16][17].

Mechano-sorptive strains that develop during the combined occurrence of moisture change and stress generation can reduce the occurrence of stresses related to drying. How mechano-sorption parameters affect the distortion of solid timber has been investigated numerically by [15] and by [7], for example. Differences between shrinkage coefficients in the radial and the tangential directions have been studied experimentally and been found to strongly affect the stresses that occur during the drying process [5].

Moisture related stress and strain fields in nonhomogeneous cross sections are extremely difficult to predict. Several numerical investigations taking different approaches to this have been conducted: [6] modelled stress and strain development in thin quadratic cross sections (discs). [4] used discs taken from logs to investigate shrinkages properties, primarily in the radial and the tangential directions. [3] constructed a 2-zone model for analysing the drying of wood containing both heart and sapwood, [9] developed a mathematical model for studying strains and stress development based on differences between MC in heartwood and in sapwood. The numerical results presented showed stress variations of interest within the cross sections.

The experiments reported in the present paper were conducted to examine strain field development during the drying of thin discs taken from timber logs. Since the drying of wood occurs much faster in the longitudinal direction than in any other directions, using thin discs enables the experimental time to be reduced appreciably and the temperature to be distributed homogeneously. The specimens dried, was controlled in a special designed climate chamber, allowing the transversal strain fields within the cross sections to be measured during the experiments. The climate, under which the specimens dried, was controlled in a special designed climate chamber, allowing the transversal strain fields to be measured during the drying process by means of an optical photogrammetric measuring system, Aramis [2]. The experimental samples were also simulated by use of a 3D model developed by [14], the moisture content history and the

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shrinkage properties of the samples being obtained in the experimental study.

2 MATERIALS AND METHODS

The strain results from the experimental study were used to calibrate the used finite element model [15].

2.1 TEST SPECIMENS

The test specimens were selected from 20 years old Norway spruce trees from North Zealand in Denmark. The trees were felled in the winter periods of the years 2009-2012, where logs with a diameter of 200 mm were removed from the trees and placed in a freezer for conservation purposes. The specimens were cut out from the frozen parts to avoid moisture loss during preparations. The disc specimens were selected from areas free from defects.

2.2 EXPERIMENTAL METHOD

The moisture related strain field was measured online by use of Aramis system, which was placed outside the climate chamber, as shown in Figure 1.

![Figure 1: Climate chamber with special designed window, allowing Aramis to measure strain field developments on specimens placed inside the chamber.](image)

This new and specially designed climate chamber developed at the Department of Civil Engineering of DTU has a temperature range from -70°C to 180°C and a controlled humidity range from 10% to 98% RH, within a temperature range of 10 to 95°C. The specimens were placed on online weighting arrangements inside the chamber, where they were dried down under well controlled climate conditions.

2.3 MODELLING

Simulation of the studied disc specimen was performed by the finite element software Abaqus [1]. It was a 3D stress analysis using 8-node linear brick elements of type C3D8. The specimen studied was a circular 15 mm thick disc divided into 20 geometrical parts consisting of a small pith area and 19 annual rings. Each annual ring was partitioned into 4 quarters. These parts were connected together with so called tie-constraint which allows the disc to function as an inhomogeneous continuum. Every quarter of the annual rings and the pith, have their own set of material parameters and moisture and temperature histories. The material model used is a spiral-orthotropic one taking account of elastic, shrinkage, and mechano-sorption strains and of material inhomogeneity. The moisture content histories used as input to the model were accumulated from test results.

3 RESULTS AND DISCUSSIONS

3.1 STRAIN AND STRESS RESULTS

Figure 2 shows the MC variation in green condition in the test log. The MC was determined from a number of oven dried (103°C) sticks that were selected from a radial path across the cross section of the timber log. The MC varies from 40% to 200%. Similar variations were found in [11] [13].

![Figure 2: Variation of initial (green) MC from pith to bark. The curve represents average MC based on the measured values marked in the figure.](image)

Discs with slit from bark to pith were examined during drying at 23°C and a relative humidity RH=63%. The slit was sawn in green condition to reduce the geometric constrains closed annual rings could generate during the drying process.

After 25 hours of drying, the heartwood reached equilibrium moisture content (EMC) of approximately 12%, whereas the sapwood still had MC close to the FSP. Shrinkage of the heartwood showed the total strain, consisting of elastic, free shrinkage and mechano-sorptive strains, to be considerable.

The slit shown in Figure 3 (B), which originally was 1.5 mm wide, had closed in the sapwood area and become wider than 1.5 mm in the heartwood area. This occurred because of the tangential shrinkage of the heartwood material striving to open the slit, at the same time as the opening of it was suppressed by the sapwood, which had not yet started to shrink, while the radial shrinkage of the heartwood area dragged the sapwood towards the pith, resulting in closing of the slit in the sapwood area.
Simulations showed there to be marked tensile stress during the drying process in the transition zones between the heartwood and the sapwood, and relatively big compression stresses when the wood sample had reached its EMC. The tensile stresses were rather close to reaching the ultimate tensile strength of the wood, whereas the compressive stresses did not reach as critical level. This experiment showed critical stress levels due to the moisture gradient between the sapwood with MC larger than or equal to FSP and the transition- and heartwood with MC below FSP.

### 3.2 CRACK DEVELOPMENTS

Moisture content below FSP can be controlled by regulating the humidity. High relative humidity (RH) gives a high EMC within the wood material and vice versa. Expressions describing the relations between EMC, RH and temperature are e.g. presented in [18]. Figure 4 shows this relationship at four chosen temperatures.

Figure 4 shows markedly decrease in EMC when the temperature increases. EMC is approximately MC=3.5% at 90°C and RH=30%, while EMC is MC=6% at 20°C at the same humidity. The temperature has also influence on the FSP, i.e. the FSP becomes lower when the temperature increases. In [15] this relationship is assumed to be linear. The temperature related reduction of the FSP is roughly of the same size as for the EMC which lead to almost the same moisture related shrinkage (i.e. same percentage of moisture loss) regardless the temperature level.

Stress and strain field and crack observation were examined numerical and experimental examined for number of drying disc samples exposed to different constant climate conditions. The chosen temperatures were 20, 60, 70, 80 and 90°C and the RH levels were chosen with the aim to find the minimum value where the discs will not crack during the whole drying process. Figure 5 shows a summary from the study.

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**Figure 3: Strain and stress development in a disc containing a slit:**

A) measured (total) tangential strain after 25 hours of drying, B) simulated tangential stress (S33) after 25 hours of drying, C) measured (total) tangential strain when EMC was reached, D) simulated tangential stress (S33) when EMC was reached.

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**Figure 4: EMC’s dependency of temperature and humidity.**
Figure 5 shows, amongst other, that discs dried at 20°C and RH=50% cracked, while discs at 60°C and RH=50% and at 90°C and RH=30% did not crack, even though the shrinkage from FSP to EMC was almost the same. One reason for this is that the stresses reduce slightly because of reduced modulus of elasticity and increased mechano-sorption effect when the temperature rises. In the model the modulus of elasticity and the mechano-sorption parameters are calculated with Eqs. (1) and (2) presented in [15]:

\[
E_i = E_0 \left(1 + \frac{E_{\text{m}}}{E_{\text{r}} T_0} \right) + E_r \left(1 + \frac{E_{\text{m}}}{E_{\text{r}} T_0} \right) a
\]

\[
m_i = m_0 \left(1 + \frac{E_{\text{m}}}{E_{\text{r}} T_0} \right)
\]

where the index \(i\) represents the material directions (\(l, r, t\)) and \(E_{\text{m}}=220\ \text{MPa}, E_{\text{r}}=400\ \text{MPa}, E_{T_0}=0.013\ \text{°C}^{-1}\), \(E_{r_T}=1300\ \text{MPa}, E_{r_T}=2200\ \text{MPa}, T_0=20\ \text{°C}, w_{f_0}=0.30, w_{r_T}=0.033, m_{r_0}=0.2\ \text{MPa}^{-1}, m_{r_T}=0.15\ \text{MPa}^{-1}, m_{T_0}=m_{T_T}=-0.01\ \text{°C}^{-1}\). \(T\) is the temperature and \(w_0\) is the moisture content.

Table 1 shows both the marked reduction of elastic modulus and the increase of the mechano-sorption parameters when the temperature rises.

\begin{table}
\centering
\begin{tabular}{|c|c|c|c|c|}
\hline
Temp. [°C] & 20 & 60 & 90 \\
\hline
RH [%] & 50 & 50 & 30 \\
\hline
EMC [%] & 9.1 & 8.4 & 3.6 \\
\hline
\textbf{\(w_0\)} [\%] & 0.091 & 0.084 & 0.036 \\
\hline
\textbf{\(E_{\text{rT}}\)} [MPa] & 492 & 335 & 273 \\
\hline
\textbf{\(E_{\text{rt}}\)} [MPa] & 860 & 580 & 464 \\
\hline
\textbf{\(m_{\text{r}}\)} [MPa^{-1}] & 0.2 & 0.28 & 0.34 \\
\hline
\textbf{\(m_{\text{r}}\)} [MPa^{-1}] & 0.15 & 0.21 & 0.255 \\
\hline
\end{tabular}
\caption{Stiffness and mechano-sorption values for different climate conditions after the EMC state is obtained.}
\end{table}

The reduced stresses would, if tensile strength was independent of temperature, reduce the risks of cracks markedly at increased temperatures. An investigation presented in [8] shows that tensile strength reduces as temperature rises, just as the strength rises with decreased MC. These results are confirmed by simulations based on results from the drying tests in the climate chamber.

Simulations based on moisture histories from the disc tests, with and without crack developments, were analysed to find the level of tangential tensile failure stress for each temperature level. Table 2 shows the tensile stresses just before when the cracks occurred.

\begin{table}
\centering
\begin{tabular}{|c|c|c|c|}
\hline
Temp. [°C] & 20 & 60 & 90 \\
\hline
\textbf{\(\sigma_{\text{fT}}\)} [MPa] & 2.2 & 1.75 & 1.65 \\
\hline
\textbf{MC [%]} & 12 & 11 & 6 \\
\hline
\end{tabular}
\caption{Tangential failure stresses in tension at different moisture and temperature states.}
\end{table}

The results clearly show reduction of the tensile strength although it was found at different moisture content states.

\section{4 CONCLUSIONS}

The experimental strain field results (by Aramis) for the studied disc specimen were used to verify the simulation model. Material parameters from earlier model investigations for Norway spruce were used with a adequate progress. The model was used to simulate the moisture related strain and stress distribution within the disc specimens using moisture histories based on different drying schedules similar to schedules used in practical kiln drying. The simulations emphasize clearly that drying schedules had a significant influence on the stress development during the drying process.

\section{5 FUTURE WORK}

A more detailed knowledge of how the tangential tensile strength depends on temperature and MC has to be investigated more thoroughly. New experiments and modelling work are planned with the aim to develop better drying schedules in the attempt to reduce stresses and related fractures caused by the drying process.

\section{ACKNOWLEDGEMENT}

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\section{REFERENCES}

THE PHENOMENOLOGICAL FRACTURE CRITERIA AND THE STRESS INTEGRATION VOLUMES IN HETEROGENEOUS MODELS OF WOOD

Pablo Guindos¹, Manuel Guaita²

ABSTRACT: This paper presents 64 different deterministic approaches and criteria to predict the failure load in mesostructurally heterogeneous models of wood. The knots and the grain deviation can generate singular stress concentrations in wood as the notches or the cracks would do it in fracture mechanics. The average stress approach proposed by Landaelius and Masuda, seems to be the best way to deal with fracture prediction in heterogeneous models. A photogrammetrically validated finite element model which accounts the grain deviation and the knots in tree-dimensions was used to analyze the differences between criteria and approaches. It has concluded that the use of conservative criteria in larger stress integration volumes is the best method to accurately predict the failure load when considering the heterogeneity.

KEYWORDS: Multiscale modelling, Phenomenological fracture criteria, average stress approach, failure prediction.

1 INTRODUCTION

There are many approaches and criteria to predict failure of wood [1-3]. Normally they are grouped according their stochastic or deterministic basis, and also if fracture toughness is considered or not. Stochastic basis allows heterogeneous wood conception, but the rupture phenomenon is not explained.

From the deterministic point of view, wood is usually modelled as a continuum homogenous material. Fracture is predicted and evaluated according several physical parameters, as the multiaxial stress state, stress intensity factors, energy release rates or crack lengths. However it becomes difficult to obtain accurate predictions by this approach due to the inherent heterogeneity of wood. Conversely, in recent times other modelling paradigms [2] have been developed, as the heterogeneous finite element models, the morphologically-based models and the lattice models. These last approaches allow at once a heterogeneous wood conception, and also an explanation about initiation and propagation of failure.

According the conventional deterministic approach if no initial cracks or notches are present in a wood specimen, failure initiation is predicted by using one of the multiple phenomenological failure criteria. Conversely, in case of singular stress concentrations the failure prediction is performed by using a crack growth criterion under the theory of fracture mechanics. However it was demonstrated by means of a heterogeneous finite element model [4], which accounts the effect of knots and the local and global grain deviation in the three-dimensional space, that the structures from the mesoscale can generate singular stress concentration as the notches and the cracks would do it in fracture mechanics. Unfortunately in this case, the conventional point strength based criterion generate inaccurate predictions, and the application of the conventional theories from fracture mechanics are not viable, since the source of stress singularity must be well known, however in a specimen with multiple knots this turns very complicated.

In recent years Masuda [5], Landaeluis [6] and later Aicher et al. [7] proposed the average stress approach which generalizes the application of the fracture mechanics since no initial cracks or notches are required. This theory consists of to apply a conventional strength based criterion using the average stresses of a region, rather than the conventional stresses of discrete points. This premise allows merging the predictions of the phenomenological stress criterion in homogeneous stress states, with the predictions of the fracture mechanics in case of singular stress concentrations.

In this research this method was not applied in clear members or fracture mechanics specimens, but in heterogeneous models wood which accounts the effect of knots and grain deviation. The key aspects for the application of the average stress approach are which is the phenomenological failure criteria used in for the
computation of the stresses and what is stress integration size. Although both authors introduced this theory with similar phenomenological criterion the proposed sizes are very different. In this work 64 different methods were proposed, including 8 different phenomenological failure criteria, 7 different sizes aside from the conventional stress point criterion.

2 MATERIAL AND METHODS

The commercial software Ansys Multiphysics v11, Ansys Inc., Canonsburg, PA, USA, was used to build the heterogeneous model and compare failure approaches. The model accounted the shape of knots as elliptical, oblique and rotated cones and truncated cones. The 3D local and global grain deviation was obtained by applying the theory of the flow-grain analogy proposed by Phillips et al. [8] regardless of mesh. Wood was modeled as a transverse isotropic material with anisotropic plasticity, with initial yield surface of Hill, model of Valliappan et al. The validation of the model with the generalizations of Shih et al. and hardening modeled as a transverse isotropic material with

Local and global grain deviation was obtained by oblique and rotated cones and truncated cones. The 3D beams of 3000×150×50 mm in size, were proposed, including 8 different phenomenological failure criteria, 7 different sizes aside from the


<table>
<thead>
<tr>
<th>Strength</th>
<th>Value (Nmm⁻²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal tension (f₁,₀)</td>
<td>89.0</td>
</tr>
<tr>
<td>Longitudinal compression (f₁,₀)</td>
<td>57.0</td>
</tr>
<tr>
<td>Transverse tension (f₁,₀)</td>
<td>4.0</td>
</tr>
<tr>
<td>Transverse compression (f₁,₀)</td>
<td>7.6</td>
</tr>
<tr>
<td>Longitudinal shear (f₁,₀)</td>
<td>9.5</td>
</tr>
<tr>
<td>Transverse shear (f₁,₀)</td>
<td>13.3</td>
</tr>
<tr>
<td>45° Off-axis tension (f₁,₄₅)</td>
<td>4.5</td>
</tr>
</tbody>
</table>

2.1 PHENOMENOLOGICAL FAILURE CRITERIA

The next phenomenological failure criteria were computed in their transversely isotropic formulation:

Tsai-Hill Criterion

Tsai [12] reported that equation as a strength theory for composite materials as follows:

\[ \left( \sigma_1 \right)^2 + \left( \sigma_2 \right)^2 + \left( \sigma_3 \right)^2 + \left( \sigma_1 \sigma_2 \right) = 1 \]

where \( \sigma_1, \sigma_2, \sigma_3 \) are the uniaxial and shearing stress components in the respective directions and sections and \( A, B, C, D, E \) and \( F \) are coefficients dependent upon uniaxial and shearing strengths.

Tsai-Azzi Criterion

Next, Tsai and Azzi [13] simplified the Tsai-Hill criterion for plane stress conditions using different strength in tension and compression, by only changing the uniaxial strengths according to the sign of the stresses:

\[ \frac{\sigma_1}{f_L} + \frac{\sigma_2}{f_R} + \frac{\tau_{LR}}{f_{LR}} = 1 \]

where \( f_L, f_R \) and \( f_{LR} \) are the uniaxial and shearing strengths. As reported by Nahas [14], Tsai also developed two additional equations for the two other mutually orthogonal planes, by properly interchanging the subscripts L-R for L-T and R-T in Eq. (2).

Norris Criterion

Norris [15] developed a criterion very similar to Tsai-Azzi except that the interaction terms were not biased toward one particular strength:

\[ \left( \frac{\sigma_1}{f_L} \right)^2 + \left( \frac{\sigma_2}{f_R} \right)^2 + \left( \frac{\tau_{LR}}{f_{LR}} \right)^2 = 1 \]

Extended Yamada-Sun Criterion

Yamada and Sun [16] proposed also a plane stress criterion. As shown by Murray [17], this criterion can be extended to 3D by adding the transverse shearing term in each direction:

\[ \frac{\sigma_1}{f_L} + \frac{\sigma_2}{f_R} + \frac{\tau_{LR}}{f_{LR}} + \frac{\tau_{RT}}{f_{RT}} + \frac{\tau_{TL}}{f_{TL}} = 1 \]

Hoffman Criterion

Hoffman [18] introduced 3 linear terms in the Hill criterion. Hence tensile-compressive strength differentiation was possible with only one equation:

\[ A(\sigma_L - \sigma_R)^2 + B(\sigma_R - \sigma_T)^2 + C(\sigma_T - \sigma_R)^2 + D\tau_{LR} + E\tau_{RT} + F\tau_{TL} + G\sigma_L + H\sigma_R \]

where A,B,…, I are coefficients dependent upon shearing and uniaxial compressive and tensile strengths.

Hashin Criterion

Hashin [19] proposed a transversely isotropic formulation of strengths was given, which permitted discerning 4 modes of failure, including: parallel tension,
World Conference on Timber Engineering

**2.2 THE SIZES OF THE AVERAGE STRESS APPROACH**

All the described criteria were computed with:

\[
\frac{\sigma^2}{f_{0,0}^2} + \frac{\tau^2}{f_{0,0}^2} + \frac{\tau^2}{f_{0,0}^2} - 1
\]

parallel compression,

\[
\frac{n_t}{f_{0,0}} - 1
\]

perpendicular tension

\[
\left(\frac{\sigma_t}{f_{0,0}}\right)^2 + \frac{\tau^2}{f_{0,0}^2} + \frac{\tau^2}{f_{0,0}^2} - 1
\]

and perpendicular compression,

\[
\left(\frac{\sigma_t}{f_{0,0}}\right)^2 + \frac{\tau^2}{f_{0,0}^2} + \frac{\tau^2}{f_{0,0}^2} - 1
\]

where \(f_t, f_{90}, f_{L}, f_{R}, f_{T}\) and \(f_{0,0}\) are respectively the strengths related to longitudinal tension and compression, transverse tension and compression and longitudinal and transverse shearing.

**Tsai-Wu Criterion**

The Tsai and Wu [20], under transversely isotropic premises, can be written as:

\[
F_{12} \left( \frac{\sigma_t}{f_{0,0}} \right)^2 + F_{66} \left( \frac{\sigma_t}{f_{0,0}} \right)^2 + F_{22} \left( \frac{\sigma_t}{f_{0,0}} \right)^2 + 2F_{12} \left( \frac{\sigma_t}{f_{0,0}} \right)^2 + 2F_{66} \left( \frac{\sigma_t}{f_{0,0}} \right)^2 - 1
\]

where \(F_{12}, F_{22}, F_{66}, F_{12}\) are coefficients obtained from shearing and uniaxial strengths, and \(F_{12}\) is a coefficient such that represents the biaxial wooden behavior and ensure the mathematical consistency of Eq. (10), if the subsequent expression is satisfied:

\[
\sqrt{\frac{F_{12}^2}{F_{12}}} \leq \frac{1}{\sqrt{F_{12}}}
\]

In this study 2 different values were estimated according the strength values of Table 1; first, the equation proposed by Liu [21]:

\[
i_{12} = \frac{1}{2} \left( \frac{1}{f_{0,0}} + \frac{1}{f_{90}} - \frac{1}{f_{0,0}} \right)
\]

And second, the expression proposed by Tsai and Wu for 45-degree off-axis tensile tests:

\[
F_{12} - 2\left( f_{45} \left( \frac{1}{f_{0,0}} \right) \right) \geq \left( 1 - \frac{f_{45}}{f_{0,0}} \right)^2
\]

where \(f_{45}\) is the 45-degree off-axis tensile strength. In the case study, according the values of Eq. (12) and (13) match respectively with the minimum and maximum value to ensure mathematical consistency, therefore the Tsai-Wu criterion was compared in its extreme values.

**3 RESULTS**

In the Figure 1a, it is shown the absolute error in the ultimate load prediction of the model. A great intrinsic component of the case study, regarding to the finite element model, the tested specimens and the studied species, must be considered in this graphic. However the most interesting results are presented in Figure 1b and 1c, where the qualitative and quantitative differences between the phenomenological failure criteria and approaches, under the typical stress gradients produced by the knots and the local grain deviation of wood, can be respectively extracted by means of comparing the no absolute errors.
Figure 1: Absolute error (a) and average error of each approach (b), and each criteria (c)

The theory of Tsai-Wu according the 45-degree off-axis tensile test (maximum value of the interaction factor \( F_{12} \)) provided the most conservative predictions, ranging between -36% and -18% the actual value of failure load, hence the size effect of the stress integration volume was about 18% in this criterion. The most accurate stress integration volume was 10×10×10 mm, which was the greatest size, so that the lowest sensitivity to singular stress concentrations was observed.

The criteria of Hoffman, Hashin and Tsai-Azzi produced similar errors, ranging from -24% to 0%, thus the size effect was about 24%, so it was a 6% higher than the effect on the Tsai-Wu criterion. The best integration size was 10×10×10 mm for these criteria. The extended Yamada-Sun and Norris theory were quite parallel to the previous and showed a similar size effect. Failure prediction generated errors from -21% to 3% and from -19% to +7% respectively. The best integration size was 8×8×8 mm for Yamada-Sun extended and 6×6×6 mm for the Norris criterion.

The Tsai-Hill criterion was not so parallel to the other theories. The error was about -17% and 5%, so that size effect was approximately 22%.

The Tsai-Wu criterion according the Liu’s theory (minimum value of the interaction factor \( F_{12} \)), generated errors from -12% and 14%, so that the size effect was similar to the previous criteria and considerably higher than the results obtained with the maximum value of \( F_{12} \). The optimum integration size was 2×2×2 mm. The influence of the interaction factor \( F_{12} \) was enormous, until such extent that predictions were between 24% and 32% higher than the values obtained with the maximum \( F_{12} \).

The average prediction of all the criteria respect to the actual failure load was of -22.4%, -16.3%, -10.1%, -5.4%, -4.5%, -1.2% and -0.7% in the volumes of 1×0.4×0.4 mm, 2× 0.4×0.4 mm, and in hexahedrons of 2, 4, 6, 8 and 10 mm of side length respectively.

The average prediction of the point strength-based criteria was 5.6% lower than its correspondent stress integration volume (2×2×2 mm). But it is expected that the difference between point strength-based criterion and average stress criterion was much higher if bigger stress integration volumes were considered instead.

The lowest errors in each approach were generated by the Tsai-Wu criterion according the Liu’s theory in the point strength-based criterion and in the shortest volumes, and the criterion of Norris, Yamada-Sun, Tsai-Hill and the Tsai-Azzi in volumes of 4, 6, 8 and 10 mm of side length respectively.

Accordingly, the average size effect in all the phenomenological failure criteria due to heterogeneity of wood was about 23%. In general, lower changes were observed among the different phenomenological failure criteria.

4 CONCLUSION

The average stress approach seems the best way to predict the failure load of wood from the deterministic point of view when heterogeneity is taken into account.

The best results were provided by the larger stress integration volumes. Nevertheless, these great sizes are much less sensitive to singular stress concentrations. It is strongly recommended the use of conservative failure criteria in large stress integration regions.

Although in general the predictions of the different phenomenological failure criteria were quite similar, it is recommended to recommend the use of at least two or three different criteria. The choice should be taken according the computational complexity and the necessities in the failure identification. In any case, if the criterion of Tsai-Wu is applied, a rigorous calibration of the interaction parameter is mandatory.
5 REFERENCES


A STUDY ON THE FIRE RESISTANCE PERFORMANCE OF WOOD FRAMED LIGHTWEIGHT WALL WHICH INCLUDING A MIDDLE LINTEL

In-hwan Yeo¹, Bum-yean Cho², Byung-yeol Min³, Kyung-suk Cho⁴, Myung-o Yoon⁵

ABSTRACT: In this study fire resistance performance of wood framed lightweight wall designed as a Han-ok(Korean traditional housing) wall system was assessed. The wall has a middle lintel and be constructed with dry system instead of traditional wet construction of Han-ok. Wall frame and middle lintel was made of wood, and fire resistant gypsum board most commonly used in domestic and foreign buildings or loess-board made of loess and straw, a Korean traditional building material, were used as finishing material. Fire resistance performance tests and evaluation of walls were carried out by requirements of non-load bearing wall because Han-ok is post-and-beam construction. Test results showed that all specimens secured fire resistance for 60 or more minutes. The wall finished with fire resistant gypsum board lost integrity by crack and drop out occurred on finish. And, in the case of finished only with loess, insulation has diminished while integrity was secured without crack and drop-out. It was determined that the specimen with a hybrid of loess and fire resistant gypsum board could secure integrity more effectively and improve its fire resistance performance better than the one with fire resistance gypsum board only.

KEYWORDS: Middle lintel, Loess board, Fire resistance, Lightweight wall, Fire resistant gypsum board

1 INTRODUCTION

The people’s interest towards low energy consumption and eco–friendly environment in residential area is increasing globally. Recently, Korean consumer’s attention has also been focused on wooden architecture, such as Han-ok, and its demand is increasing. Most of the wooden buildings, especially in the case of Han-ok, are single or two-storied, but multi storied and large-spaced wooden buildings become possible by development of high functional laminated lumber and wooden building technologies. Fire safety technology is considered more serious as buildings become wide, complex and multi storied. In this study, fire resistance performance of lightweight wall designed for compartment partition has been evaluated. The walls have a middle lintel and constructed with dry systems instead of a traditional wet construction of Han-ok.

2 FIRE RESISTANCE RATING OF WOOD FRAMED WALLS

2.1 KOREA

The fire resistance rating of wood framed lightweight wall structures is prescribed as have of 30 to 90 minutes based on the stud size, thickness of finished material and insulation in accordance with KS F 1611-1 [Fire resistance performance for elements of building construction – Part1:Wall and floor/ceiling, roof/ceiling assemblies of light-frame wood structures]. The composition of walls is similar to lightweight wall types in North America. The insulation material consist of 60 kg/㎥ (density) of the rock wool. The finished material consist of the fire resistant gypsum board have of 12.5 mm or 15 mm. The both side finished materials of the interior walls are arranged...
symmetrically and the exterior walls consisted of gypsum boards on inside and have a combine of engineered lumber board and moisture proof sheet on outside.

**Table 1. Fire resistance rate of wood framed lightweight wall (KS)**

<table>
<thead>
<tr>
<th>Contents</th>
<th>Stud cross section (mm)</th>
<th>Insulation</th>
<th>Finish material (Thickness, mm)</th>
<th>FRR(min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interior</td>
<td>38x89 (600)</td>
<td>Rock wool (60)</td>
<td>Gypsum board (15.0x1)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>89</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>38x140 (600)</td>
<td>Rock wool (60)</td>
<td>Gypsum board (12.5x2)</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>140</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gypsum board (12.5x2)</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gypsum board (12.5x2)</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gypsum board (15.0x1)</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gypsum board (12.5x2)</td>
<td>90</td>
</tr>
<tr>
<td>Exterior</td>
<td>38x89 (600)</td>
<td>Rock wool (60)</td>
<td>Gypsum board (15.0x1)</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>89</td>
<td>PW or OBS or PB (12x1) + EF + MS</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>38x140 (600)</td>
<td>Rock wool (60)</td>
<td>Gypsum board (12.5x2)</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>140</td>
<td>PW or OBS or PB (12x1) + EF + MS</td>
<td>60</td>
</tr>
</tbody>
</table>
| PW : Plywood, OBS : Oriented Strand Board, PB : Particle Board, EF : Exterior Finish, MS : Moisture proof Sheet

2.2 INTERNATIONAL

The various fire resistance ratings of wood framed walls depending on conditions of the bearing or non-bearing have suggested in accordance with UL (Underwriters Laboratories Inc.). Table 2 indicate an example of non-bearing walls retain a fire resistance rating of 45 minutes to 120 minutes.

**Table 2. Fire resistance rate of wood framed lightweight wall (UL)**

<table>
<thead>
<tr>
<th>Stud (mm)</th>
<th>Insulation</th>
<th>Finish material (Thickness, mm)</th>
<th>FRR(min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>38x89</td>
<td>Rockwool 76.2</td>
<td>Fire exposed side/ non exposed side</td>
<td>45</td>
</tr>
<tr>
<td>(Non bearing wall)</td>
<td></td>
<td>Mineral and fiber Board 19.1</td>
<td>U328</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cementitious Backer Units 12.7</td>
<td>U374</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gypsum Board 15.8</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Precast Autoclaved Aerated Concrete Panels 50.8</td>
<td>U213</td>
</tr>
<tr>
<td></td>
<td>Spray applied Fire resistant material</td>
<td>Fire exposed side/ non exposed side</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gypsum Board 15.8</td>
<td>U332</td>
</tr>
</tbody>
</table>

**Table 3. Fire resistance performance by protection (CAM)**

<table>
<thead>
<tr>
<th>Finish material (Thickness, mm)</th>
<th>FRR(min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5, Douglas fir plywood, Phenolic bonded</td>
<td>5</td>
</tr>
<tr>
<td>12.7, Douglas fir plywood, Phenolic bonded</td>
<td>10</td>
</tr>
<tr>
<td>16.2, Douglas fir plywood, Phenolic bonded</td>
<td>15</td>
</tr>
<tr>
<td>9.5, Gypsum board</td>
<td>10</td>
</tr>
<tr>
<td>12.7, Gypsum board</td>
<td>15</td>
</tr>
<tr>
<td>16.2, Gypsum board</td>
<td>20</td>
</tr>
<tr>
<td>12.7, Type X Gypsum board</td>
<td>25</td>
</tr>
<tr>
<td>16.2, Type X Gypsum board</td>
<td>40</td>
</tr>
<tr>
<td>Double 9.5, Gypsum board</td>
<td>25</td>
</tr>
<tr>
<td>12.7+9.5, Gypsum board</td>
<td>35</td>
</tr>
<tr>
<td>Double 12.7, Gypsum board</td>
<td>40</td>
</tr>
</tbody>
</table>

In Canada, the fire resistance performance of the wood framed wall determined from the type and thickness of the fireproofing is suggested in accordance with CAM(Component Additive Method), NRCC (National Research Council of Canada).

Table 3 shows the fire resistance performance of wall elements determined from the type and thickness of the fireproofing comply with CAM. However, it does not mean that the fire resistance performance shall be taken as a final fire resistance rating of a wall.

3 EXPERIMENT OVERVIEW

3.1 FACTORS

In this study, specimens had been made so as to know the fire resistance performance of the wood framed lightweight wall containing the middle lintel [Table 4].
The thickness of walls had less than 150 mm to be contemplated the thickness of walls of Han-Ok. The specimens were composed of 38x89(2”x4”) wood stud, 150x150 mm wood middle lintel, 12.5mm fire resistance gypsum board, and 12.0mm loess board.

Table 4. Composition of specimen

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Composition</th>
<th>Cross-sectional Drawing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen -1</td>
<td>PB 2layers + 38x89 Wood stud @600 + PB 2layers</td>
<td><img src="image1" alt="Cross-sectional Drawing" /></td>
</tr>
<tr>
<td>Specimen -2</td>
<td>PB 2layers + 38x89 Wood stud @600 + GW(THK50, 24kg/m³) + PB 2layers</td>
<td><img src="image2" alt="Cross-sectional Drawing" /></td>
</tr>
<tr>
<td>Specimen -3</td>
<td>PB 2layers + 38x89 Wood stud @600 + GW(THK89, 9kg/m³) + PB 2layers</td>
<td><img src="image3" alt="Cross-sectional Drawing" /></td>
</tr>
<tr>
<td>Specimen -4</td>
<td>LB 2layers + 38x89 Wood stud @600 + LB 2layers</td>
<td><img src="image4" alt="Cross-sectional Drawing" /></td>
</tr>
<tr>
<td>Specimen -5</td>
<td>LB 1layer + GB 1layer + 38x89 Wood stud @600 + PB 1layer + LB 1layer</td>
<td><img src="image5" alt="Cross-sectional Drawing" /></td>
</tr>
<tr>
<td>Specimen -6</td>
<td>LB 1layer + GB 1layer + 38x89 Wood stud @600 + GW(THK89, 9kg/m³) + PB 1layer + LB 1layer</td>
<td><img src="image6" alt="Cross-sectional Drawing" /></td>
</tr>
</tbody>
</table>

Middle lintel : 150x150mm, PB : THK 12.5 Fire resistant gypsum board, GW : Glass Wool, LB : THK 12.0 Loess Board

3.2 SPECIMEN

The stud and middle lintel was made of the laminate timber of Douglas-fir from North America. Wall frame was composed of stud (38x89mm) and top/bottom plate with the boundary of middle lintel (cross section 150x150mm) and finished with the fire resistant gypsum board or loess board. The insulation material of specimen-2, 3, and 6 was made of the glasswool fiberboard density of 24 kg/m³, 9 kg/m³, and 9 kg/m³ respectively. The actual size of specimens was 3000 x 3000 mm. Figure 1 indicate of the drawing of specimen and Figure 2 of the process of specimen production.

3.3 TEST METHODS

Fire resistance test was performed following ISO 834-1 and ISO 834-8. The fire resistance performance of non-bearing partition walls shall satisfy the integrity and insulation. The thermocouples had been installed on surface of non-fire exposed side of specimens, the joint of middle lintel-finish boards. And thermocouples to measure the charring rate of wood also had been installed inside of wood (middle lintel and stud). The tests had been performed until it was failed of their integrity or insulation.

Table 5: Temperature of unexposed surface and fire resistance performance

<table>
<thead>
<tr>
<th>Specimen</th>
<th>AT(PS)</th>
<th>MT(PS)</th>
<th>Fire resistance Performance(min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen-1</td>
<td>85(147)</td>
<td>94(187)</td>
<td>91</td>
</tr>
<tr>
<td>Specimen-2</td>
<td>105(145)</td>
<td>114(185)</td>
<td>97</td>
</tr>
<tr>
<td>Specimen-3</td>
<td>103(146)</td>
<td>131(186)</td>
<td>98</td>
</tr>
<tr>
<td>Specimen-4</td>
<td>107(159)</td>
<td>206(199)</td>
<td>67</td>
</tr>
<tr>
<td>Specimen-5</td>
<td>123(161)</td>
<td>266(201)</td>
<td>100</td>
</tr>
<tr>
<td>Specimen-6</td>
<td>104(157)</td>
<td>214(197)</td>
<td>98</td>
</tr>
</tbody>
</table>

AT : Average Temperature( °C), MT : Maximum Temperature( °C) PS : Performance Criteria( °C)

4 EXPERIMENT RESULTS

As it is shown in the Table 5, specimen-1,2,3 with the applications of the fire resistant gypsum boards lost their integrity in 91, 97 and 98 minutes respectively. Especially, the integrity performance has diminished with flames in the side and lower part of the middle lintel. Specimen-4, 5, 6 with applications of the loess board lost their fire resistance performance in 67, 100 and 98 minutes respectively, due to losing their insulation.

It is shown that the fire resistance performance of specimen-1, 4 and 5 without the insulation materials was 91, 67 and 100 respectively.
At the end of the experiments, specimen-5 with the mixture of loess board and fire resistant gypsum board showed the best fire resistance performance among the specimens. Specimen-4 and 5 showed higher temperature on the unexposed side than Specimen 1. It is seemed that it was caused by difference of heat conductivity through finishing material.

The test result of the fire resistance performance of Specimen-2, 3, and 6 including the insulation materials lost at 97, 98, and 98 minutes respectively. Comparing with the Specimen 1 which contains a fire resistant gypsum board only with non-insulation material, specimen 2 and 3 showed better fire resistance performance by around 6 and 7 minutes respectively. It is the level similar to the additional fire resistance performance time 5 min.) for a glass wool insulation material provided in the fire resistance performance determined from CAM(Component Additive Method) of NRCC. However, Specimen 6 showed the similar fire resistance performance compared to Specimen 5 with its loess board and a finished fire resistant gypsum board.

5 CONCLUSION

For the wood framed lightweight wall covered with fire proofing materials using fire resistant gypsum boards, the integrity was lost due to drop-out and crack in the fire resistant gypsum board. Especially, the integrity performance has diminished with flames in the near part of middle lintel.

In case of applying the loess board, the integrity was secured without any drop-out and crack, but the insulation got lower than applying the fire resistant gypsum board because of high heat conductivity than that of fire resistant gypsum board.

Fire resistance test of a mixture of loess board and fire resistant gypsum board has shown that it has better fire resistance performance than the ones with fire resistant gypsum board or Loess board only.

In the view point of insulation material, Specimen-1, 2, and 3 which have lower integrity showed better fire resistance performance enhancing when have insulation. On the other hand, specimen 5 and 6 with insulation didn't contribute to improve their fire resistance performance. Therefore, it is safe to assume that insulation material has greater influence on the improvement of integrity than on the improvement of the insulation.

REFERENCES

ARCHITECTURE AND ENGINEERING CASE STUDIES

MODELS FOR EVALUATING STRUCTURAL WOOD DAMAGED BY XYLOPHAGOUS

Takashi Yojo¹, Maria Beatriz Bacellar Monteiro², Fabiola Margoth Zambrano Figueroa³, Gonzalo Antonio Carballeira Lopez⁴, Maria José de Andrade Casimiro Miranda⁵

ABSTRACT: This paper presents a method that can be used in inspections to verify the safety of structural building components and considers the experience and knowledge of engineering, wood anatomy, entomology and mycology, acquired over the years in inspections carried out in wood structures. The basis for this work was a model of resistance related to simple bending which may extend to other types of loadings, with possible applications in structural analysis of shears, rafters, beams and pillars. Four models to estimate the loss of mechanical strength on structural wood were defined based on the theory of strength of materials and on the type and intensity of the biological attack. The proposed method has several applications and can even be used for verification of wooden structures of buildings of architectural heritage.

KEYWORDS: Decay, Wood, Assessment, Biodeterioration, Building, Termite, Wood Borer, Decay fungi.

1 INTRODUCTION

Wood is an organic material composed primarily of cellulose, lignin and hemicellulose. These molecules are arranged in such a way that constitutes a complex structure which in close interaction with water molecules confers strength to cell walls or wood fibers. However, as all organic material, wood can be attacked by organisms that use it as a food source.

Wooden components of building structures must comply with safety, performance and durability requirements that should be periodically checked. This check is done through a structural analysis which covers the inspection and detailed survey of the building.

To contribute to the diagnosis and analysis of timber structures, this paper proposes a method that should assist on inspections to verify the safety of structural building components.

The method taken into account the experience on several areas of knowledge such as engineering, wood anatomy, entomology and mycology, acquired over the years in inspections carried out in wooden structures and offers a tool of practical application.

2 MODELING AND PARAMETRIZATION

The basis of this work is a model of resistance related to simple bending which may extend to other types of loadings, with possible applications in structural analysis of shears, rafters, beams and pillars. Based on the theory of strength of materials four models were defined to estimate the loss of mechanical strength on structural wood.

The variables considered for the mechanical strength for the proposed models were the percentage of reduction on the healthy cross section of wood and the percentage of reduction on mechanical strength without reducing the original cross section.

Each model is related to a specific type of biological attack that is harmful to the integrity of the wood,
because each organism has a specific mechanism of interaction with the material wood, depending on its biology.

In Brazil, the most common types of biological attack are soft, white and brown rot fungi; subterranean and arboreal termites; dry wood termites and wood borers.

2.1 BIODETERIORATION

Biodeterioration can be defined as any undesirable change in the properties of a material caused by the activity of organisms. The process of wood biodeterioration leads to its decomposition and loss of their physical and mechanical properties.

The attack of the wood rot fungi is closely related to the presence of water. The fungi can grow on wood only when free water is available in sufficient quantity to enable the occurrence of enzymatic reactions required for digestion and absorption of food. The wood fiber saturation point (FSP), which varies around 30% of moisture on a dry basis, is important because below this moisture the fungi can’t grow.

Groups of decay fungi can be recognized by the visual appearance of their attack on wood.

The white rot fungi degrade all components of the wood, including cellulose. Wood attacked by this group of fungi loses its glossy appearance and natural color, becoming whitish. The timber acquires, in addition, a spongy consistency, and sometimes the attacked region is delimited by black lines (Figure 1).

![Figure 1: White rot attack on a piece of wood](image1)

Brown rot fungi degrade the molecules of cellulose and hemicellulose of the wood. The attacked wood has a dark brown color and cracks parallel and perpendicular to its fibers, acquiring, in addition, a brittle and crumbly consistency (Figure 2).

![Figure 2: Brown rot attack on a piece of wood](image2)

The soft rot fungi have restricted their attack to the wood surface. When attacked by this type of fungi, the surface of the wood becomes soft. When drying, the surface darkens and tends to have small cracks parallel and perpendicular to the fibers. The biodeterioration of the wood by soft rot fungi occurs in environments with high moisture content and low oxygen level. These conditions inhibit the development of both white and brown rot fungi (Figure 3).

![Figure 3: Soft rot attack on a piece of wood](image3)

Among the xylophagous insects, two groups are the main responsible for damages to wood: termites (Order Isoptera) and wood borers (Order Coleoptera).

Termites are social insects, ie, form colonies composed of different categories of individuals. Termites are mainly present in tropical regions of the world and have a variety of habits. The caste of workers is the largest category of the society and is responsible for the attack on the wood.

Termites are often grouped according to their nesting habits. They are called wood termites when their colony develops wholly within the wood; soil termites when the colony grows on soil and arboreal termites when the colony grows on some support above the ground, usually a tree trunk (Figure 4).
The attack by wood borers begins when the adult female lays its eggs from which larvae hatch. These larvae will feed on the wood until the adult stage when they pierce holes and in the wood that lead to the external environment. In general, the attack is noticed through the holes and residues left by adults when they leave the wood (Figure 5).

**2.2 BENDING THEORY**

All the models consider a wood beam submitted to simple bending in the critical cross section.

**2.3 SAFETY ASSESSMENT**

The safety of the structure was calculated imposing a condition that the capacity of the wood components must be equal or larger than its loading:

\[ f_d \geq \sigma_d \]  \hspace{1cm} (1)

where:

- \( f_d \) is the design bending strength, and
- \( \sigma_d = \gamma_m \frac{M}{W_i} \)  \hspace{1cm} (2)

where:

- \( \gamma_m \) is the partial factor for loading;
- \( M \) is the flexural moment; and
- \( W_i \) is the section modulus where each model is represented by the index \( i \).

**2.4 MODELS OF STRENGTH BASED ON TYPE AND INTENSITY OF BIOLOGICAL ATTACK**

In this study it was considered four models depending on the type of biological attack. For the sake of didactic proposal only the rectangular cross-section was used. These models consider the attack of wood decay organisms found in Brazil and can be adapted for other countries.

**2.4.1 Model 1 – External attack**

This model can be applied to pieces of wood attacked by soft rot fungi. It consists in the reduction of the section modulus due to the reduction of the outer portion of the cross section. The residual portion will be considered sound wood (Figure 6).

In this model the section modulus is calculated by:

\[ W_i = \frac{(t - 2d) \cdot (h - 2d)^2}{6} \]  \hspace{1cm} (3)

Where \( d \) is the depth of attack.

**2.4.2 Model 2 – Internal attack**

This model can be applied to pieces of wood attacked by white and brown rot fungi and dry wood termites. It consists in the reduction of the section modulus due to the reduction of the inner portion of the cross section. The residual portion will be considered sound wood (Figure 7).

In this model the section modulus is calculated by:

\[ W_i = \frac{th^2}{6} \cdot \frac{(t - 2d)(h - 2d)^3}{6 \cdot h} \]  \hspace{1cm} (4)

Where \( d \) is the thickness of sound wood.
2.4.3 Model 3 – Uniform attack
This model can be applied to pieces of wood attacked by subterranean and arboreal termites. It consists in the reduction of mechanical strength maintaining the same section modulus (Figure 8).
In this model the section modulus is calculated by:

\[ W_3 = \frac{t h^2}{6} \]  

(5)

2.4.4 Model 4 – Sapwood attack
This model can be applied to pieces of wood attacked by insects when heartwood is durable. It consists in the reduction of section modulus due to the elimination of sapwood. The residual portion will be considered sound wood (Figure 9).
In this model the section modulus is calculated by:

2.5 Intensity of biological attack
The intensity of the biological attack was divided into five groups: sound wood; initial attack (up to 15% of the cross section); moderate attack (between 15% and 30% of the cross section); intense attack (between 30% and 80% of the cross section), and destroyed (over 80% of the cross section).

3 RESULTS
The percentage of residual capacity for rectangular beams subjected to bending was calculated considering the four models and intensity of attack. The results are given on Table 1.

<table>
<thead>
<tr>
<th>Intensity of attack</th>
<th>Loadings</th>
<th>Residual capacity (%) for models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Sound</td>
<td>In both directions</td>
<td>100</td>
</tr>
<tr>
<td>Initial</td>
<td>In direction h</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>In direction t</td>
<td>75</td>
</tr>
<tr>
<td>Moderate</td>
<td>In direction h</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td>In direction t</td>
<td>53</td>
</tr>
<tr>
<td>Intense</td>
<td>In direction h</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>In direction t</td>
<td>6</td>
</tr>
<tr>
<td>Destroyed</td>
<td>In both directions</td>
<td>0</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS
The proposed method has several applications and can even be used to verify wooden structures of buildings of architectural heritage, and for the conservation of traditional building techniques.
ACKNOWLEDGEMENT

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REFERENCES


SUPPLY CHAIN MANAGEMENT STRATEGIES FOR MULTI-STOREY TIMBER CONSTRUCTION IN AUSTRALIA AND NEW ZEALAND

Matt Holmes\textsuperscript{1}, Keith Crews\textsuperscript{2}, Grace Ding\textsuperscript{3}

\textbf{ABSTRACT:} Multi-storey non-residential buildings are a potential area that timber products could be used as structural elements. Two major barriers to timber's use in non-residential construction in Australia and New Zealand (NZ) have been identified as overall designer confidence and fire performance. Another important area that poses as a potential barrier to an increase in timber use in multi-storey buildings is the current supply chain. Literature has outlined there historically hasn't been a market or demand for large timber buildings constructed over 4 storeys in Australia or NZ. As a result aspects of the supply chain needed to deliver such buildings have remained fragmented and relatively primitive in nature. The supply chain for multi-storey timber buildings in Australia and NZ includes a number of different stages as outlined below in figure 1. Between each stage a number of transportation, material and design information flows occur. This paper focuses on the issues in the supply chain from forestry through to construction and aims to outline the key areas that pose as a risk on the cost and time of multi-storey timber buildings.

\begin{center}
\includegraphics[width=\textwidth]{figure1.png}
\end{center}

\textit{Figure 1: Supply chain for multi-storey timber buildings}

A critical area of the supply chain that needs to be addressed is the fabrication process as well as the manufacturing and supply of materials to fabricators and construction sites. Building contractors have concerns with the lack of fabrication skills, long lead times and ability to deliver a quality product on schedule for multi-storey timber construction in Australia and NZ. Supply Chain Management originated in the manufacturing industry though is a
relatively new area in the construction industry. SCM views the entire supply chain instead of individual parts or processes. Its introduction has the potential to help improve transparency and alignment of the supply chains coordination and has the potential to improve time, cost and quality of construction. Time and cost are important factors that developers and clients alike typically use to measure the success of construction projects. For new building systems addressing things that have potential to impact these significant factors is crucial to its overall future success. This paper aims at reviewing the current state-of-art of SCM used in construction and discusses how it can best be adopted for multi-storey timber construction. The paper presents the results of semi-structured interviews with practitioners across the supply chain of timber construction in Australia and NZ to investigate the key problem areas and to identify opportunities. Case studies of timber buildings in NZ Finally the paper presents the SCM strategies that have been developed to address the current limitations and improvements in the timber industry.

**KEYWORDS:** Supply Chain Management, timber construction, Australia & New Zealand

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ABSTRACT: The Masonite Flexible Building (MFB) system is a complete timber building system for commercial and residential multi-storey houses. The system is subdivided into two market variants; XL and Light. The XL version is for tall and large buildings with long floor spans while the Light version is adapted for smaller buildings with lower loads. Though differing in technical performance, the functional criteria are the same for both variants. The MFB system uses prefabricated wall, floor and roof elements which are delivered in flat packages and erected on the construction site. The MFB system might be classified as a panel construction, where the load-carrying structure consists of composite light-weight timber I-beams mechanically integrated with a composite laminated wood panel called PlyBoard™. The I-beams and the panel form a strong and rigid carcass for wall and floor elements, making the system well suited for high rise construction. A key feature of the MFB system is the connection technique which enables swift erection of the system units on site. The plyboard panels are provided with a continuous slot along the periphery. The slot is used as a general connection interface for the joining of the wall elements. The floor elements are suspended and hooked onto the bearing walls using sheet steel hangers, allowing swift assembling of the floor deck and enabling direct vertical wall-to-wall load transfer parallel to grain. The paper presents the construction principles, system components and units, erection technique and functional and architectural aspects of the Masonite Building System.

KEYWORDS: Masonite Flexible Building system, multi-storey timber buildings, slotted-in connections, suspended connections, functionality requirements.

1 INTRODUCTION

Building systems and construction methods for multi-storey timber buildings have developed rapidly. Today, timber construction technology uses prefabricated units, which are highly competitive with regard to cost efficiency, environmental impact and erection. The Masonite Flexible Building (MFB) system is one of several systems based on prefabricated units.

The MFB system is an open building system in the sense that the technical solutions and the building method are free to be employed by architects, designers, manufacturers and construction companies. The system meets current requirements regarding fire safety, moisture conditions, strength and stabilization, thermal and acoustic insulation. The system is subdivided into two market variants; MFB XL and MFB Light [1]. The XL system is aimed for large multi-storey buildings up to 8 storeys high and floor spans up to 8 meter while the Light version is cost optimised for detached houses and smaller buildings with lower loads and floor spans. The main difference between the system variants is the structural design of the floor and the wall elements. The XL system is a modified panel construction featuring a special composite laminated wood panel called PlyBoard™, I-beams and I-studs whereas the Light system is a conventional stud and sheathing construction. Though differing in technical performance, the functional demands are the same.

The two variants can be combined in multi-storey building to obtain a cost-effective solution. For example the Light system can be used in the upper storeys where the vertical loads and horizontal wind forces have not yet developed in magnitude, while the XL system is used in the lower storeys. This paper focuses on the XL system.

2 THE MFB XL SYSTEM

The XL system consists of prefabricated wall, floor and roof elements which are delivered in flat packages and assembled storey by storey. The system is intended for multi-storey buildings up to 8 storeys and the floor spans up to 8 m. The system meets fire requirements of REI 60 (Swedish Building Rules, BBR 5:221) and noise requirement class A (Swedish Standard, SS 02 52 67).
Structurally XL is a panel construction with linear and plane wood-composite members. Prominent features are the plywood panel with its unified connection interface, elimination of horizontal timber members in the walls to reduce settlement to a minimum and the suspended floor-to-wall connection. The carcass structure and the high quality of system components offer architectural freedoms and simple assembling.

2.1 LOAD-BEARING STRUCTURE

External wall and floor elements use a ribbed plywood panel construction with ribs of composite I-studs and I-beams, respectively. Internal wall elements may have, depending on the magnitude of load and application, a ribbed or boxed plywood panel structure with ribs of squared timber studs. The different load-bearing structures are depicted in Figure 1. The plywood panel is attached to the I-beams/I-studs mechanically by nails. The combination of plywood and I-studs/I-beams forms a strong and rigid load-bearing structure well suited for high rise construction, see Section 2.2.1 and 2.2.2 for description of each of the structural members.

Usually in wall panel constructions, the individual studs carry the vertical loads from roof and suspended floors whereas the sheathing resists the horizontal forces due to wind and accounts for the bracing. In this case, the plywood panel is employed for carrying both vertical and horizontal loads, thus significantly increasing the load-bearing efficiency of the walls. The use of high quality engineered wood products offers new design options which improves many of earlier disadvantages for multi-storey timber buildings, such as vertical settlement due to loading perpendicular to grain of horizontal members, thermal bridges at wall-to-floor junction etcetera.

**Figure 1.** Examples of structural wall and floor carcasses used in the XL system; Top: Ribbed plywood panel with I-studs/I-beams used in external walls and floor; Middle: Double ribbed plywood panel for internal walls; Bottom: Boxed plywood panel for internal walls.

2.2 MATERIAL AND SYSTEM COMPONENTS

The structural components of the XL system are a composite light-weight I-beam and a plywood panel. Together they constitute the structural skeleton of the building elements.

2.2.1 Composite I-beams

The composite I-beams and I-studs are used as structural members in wall, floor and roof elements. The flanges are made of machine graded structural timber, and the web is made of hard fibreboard or OSB, see Table 1. The different types of beams and studs pertinent to the MFB system are compiled in Table 1. The I-beams/I-studs are manufactured by Masonite Beams AB [2].

**Table 1.** Characteristics of I-beams and I-studs in the MFB System. Height and weight of beams (H, HI, HB) and studs (R) with web of hard fibreboard or OSB.

<table>
<thead>
<tr>
<th>H [mm]</th>
<th>Type H / R</th>
<th>Weight [kg/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Board</td>
<td>OSB</td>
</tr>
<tr>
<td>200</td>
<td>3.1</td>
<td>2.8</td>
</tr>
<tr>
<td>220</td>
<td>3.2</td>
<td>3.0</td>
</tr>
<tr>
<td>250</td>
<td>3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>300</td>
<td>3.8</td>
<td>3.5</td>
</tr>
<tr>
<td>350</td>
<td>4.2</td>
<td>3.8</td>
</tr>
<tr>
<td>400</td>
<td>4.6</td>
<td>4.1</td>
</tr>
<tr>
<td>450</td>
<td>5.0</td>
<td>4.4</td>
</tr>
<tr>
<td>500</td>
<td>5.3</td>
<td>4.7</td>
</tr>
</tbody>
</table>

**Table 2.** Characteristics of I-beams and I-studs in the MFB System. Height and weight of beams (H, HI, HB) and studs (R) with web of hard fibreboard or OSB.

<table>
<thead>
<tr>
<th>Weight [kg/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type HB</td>
</tr>
<tr>
<td>Board</td>
</tr>
<tr>
<td>200</td>
</tr>
<tr>
<td>220</td>
</tr>
<tr>
<td>250</td>
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<tr>
<td>300</td>
</tr>
<tr>
<td>350</td>
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<tr>
<td>400</td>
</tr>
<tr>
<td>450</td>
</tr>
<tr>
<td>500</td>
</tr>
</tbody>
</table>

2.2.2 Plyboard panel

PlyBoard™ is a three-layered wood composite panel with a core of LVL and surface layers of hard fibreboard as shown in Table 2. The veneer plies have a thickness of 3-4 mm and the grain orientation is parallel for all plies. The material composition gives the panel high strength and stiffness, good form and dimensional stability and high diffusion resistance. The panel is available with fibre board thicknesses of 4 or 8 mm and number of LVL plies between 3 and 7. Format dimension is 1200×2400 mm. Manufacturer is IBC Group [3].

In order to achieve a continuous panel which extends the full length of the wall and floor elements the individual panels are spliced using a loose tongue joint which is glued to attain a strong joint, and for external walls also airtightness and vapour barrier, see Figure 2.

2.3 WALL ELEMENTS

Decisive for the design of the wall elements are the way the building is used, the building physics and energy and acoustic related requirements.
Table 2. Characteristics of PlyBoard, a three-layered composite panel with core of LVL and surface layers of hard fibreboard. The panel is employed as a load-bearing member in wall, floor and roof elements.

<table>
<thead>
<tr>
<th>Model</th>
<th>Thickness of board [mm]</th>
<th>No. of plies</th>
<th>Total thickness [mm]</th>
<th>Weight [kg/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P43</td>
<td>4</td>
<td>3</td>
<td>19</td>
<td>14.8</td>
</tr>
<tr>
<td>P45</td>
<td>4</td>
<td>5</td>
<td>27</td>
<td>19.7</td>
</tr>
<tr>
<td>P47</td>
<td>4</td>
<td>7</td>
<td>34</td>
<td>24.7</td>
</tr>
<tr>
<td>P83</td>
<td>8</td>
<td>3</td>
<td>27</td>
<td>22.1</td>
</tr>
<tr>
<td>P85</td>
<td>8</td>
<td>5</td>
<td>35</td>
<td>27.1</td>
</tr>
<tr>
<td>P87</td>
<td>8</td>
<td>7</td>
<td>42</td>
<td>32.0</td>
</tr>
</tbody>
</table>

Standard dimension [mm] 1200×2400 mm.

Strength class:
Board: HB.HLA2.

Figure 2. Plyboard panel; Top: Periphery slot; Bottom: Spliced panels using a glued board tongue joint. Dimensions in mm.

All the wall elements are delivered with preinstalled tubes for electric cables and power sockets. The maximum lengths of the wall elements are limited to 9 m due to what can be transported by road.

2.3.1 External wall elements
The external wall elements are delivered with cladding, doors and windows and a perimeter beam for the suspend-ed floor. The carcass is a ribbed panel structure according to Figure 1 with I-studs and a plyboard panel shown in Table 1 and Table 2.

Figure 3 shows a representative external wall construction with a description of each of the various layers and parts. The depth of the load-bearing layer is normally 200-400 mm depending on the demand of thermal insulation. The plyboard panel is attached to the inside of the I-studs. This enables the vertical loads to be transferred directly to the foundation, and is further favourable for the building physics as the dense plyboard can perform the triple functions of bracing, airtight membrane and vapour barrier provided that the panel splices are tightened and sealed.

Due to the continuous panel sheathing and the suspended floor connection, which structurally disconnects the wall from the floor, the building envelope remains completely intact and unbroken. Combined with the low thermal transmission profile of the I-studs the thermal bridges are hence reduced to a minimum, making the wall design well adapted for highly insulated buildings.

An issue of great importance with respect to high rise timber construction is the settling due to horizontally installed timber members in the wall and floor. Subsequently, the proportion of load-bearing horizontal members in the XL external walls is reduced; the top and bottom plates are replaced by head and sole strips of hard fibre board.

2.3.2 Internal wall elements
The internal walls have either a doubled ribbed panel or a boxed panel structure according to Figure 1, with squared timber studs as shown in Table 1 and Table 2 respectively.

Like the external walls, internal walls carry the loads from the roof, suspended floors and the dead load of the wall itself. Furthermore, they are used for bracing the building implying that they transfer the horizontal wind force and the wind induced uplift force. Figure 4 and
2.4 FLOOR ELEMENT

Floor structures have to offer adequate load-carrying capacity, minimal deflection and vibration, effective sound insulation and sufficient fire resistance. To satisfy these demands, the XL suspended floor elements are designed as a multi-layered structure with an embedded load-bearing structure, a floor finish and a ceiling as illustrated in Figure 6. The structural floor layer is a composite ribbed plywood panel according to Figure 1. The floor elements are prefabricated and delivered with pre-attached floor hangers and resilient ceiling.

To obtain good sound insulation and vibration properties heavyweight finishes are advantageous. The plywood panel itself adds weight and stiffness, and distributes the acoustic energy laterally enhancing the acoustic performance. A supplementary sub-floor of dry sand fill or similar material can be added to further improve the sound insulation. The floor finish consists normally of a double layer of floor plasterboard on top of a single layer of floor particleboard, see Figure 6. The ceiling is pre-attached to the floor element at manufacturing by ceiling-hangers to achieve resilient fixings. The ceiling-hangers allow the ceiling to be displaced against the bottom flange of the I-beams to enable vertical stacking of floor elements during transportation. Beneath the fixings a double-layer of acoustic plasterboards is used.

An essential issue of a suspended floor is the design of the wall connection which has to fulfil multiple demands. It should not only transfer the self-weight and the imposed load of the floor, but also eliminate acoustic flank transmission to nearby compartments, be fire resistant and allow for easy erection of the floor elements.

The MFB system uses sheet metal hangers that are pre-mounted on the ends of the floor element with mechanical fasteners and hooked on to a separate perimeter beam on the wall when the floor element is lifted in place. The design of the floor hanger is described more in Section 2.5.3. The choice of a floor-to-wall connection that is structurally disconnected from the external wall carcass is governed by four principles:

1. By placing the floor “beside” the wall, loading of horizontally installed timber members perpendicular to grain is avoided, hence there is no contribution of the floor to the settling of the wall.
2. The floor connection should not interfere with the integrity of the wall by penetrating the building envelope, causing thermal bridges and/or harm to the airtightness and vapour barrier.
3. The floor assembling should be simple and swift.
4. The connection should accommodate erection tolerances and moisture induced dimensional changes without affecting the structural integrity of the wall.

2.5 STRUCTURAL CONNECTIONS

The XL system has a unified interface for the interconnection of wall elements and a hanger attachment for the suspended floor to the load-bearing walls. Depending on the type of wall intersection, different types of slotted-in steel plates are used.
2.5.1 The slotted-in wall connection
The external and internal wall elements are spliced by slotted-in steel plates and screws. The plates are at manufacturing pre-installed in the slot of the plyboard panel along the bottom edge and one of the vertical end edges. At erection the protruding part of the plates go into the opposite slot of the already erected wall elements. When fixed in position remaining screws are fastened. The connection interface is shown in Figure 7 for an external wall and in Figure 8 for a wall junction. The periphery slot enables flexibility as the number and the location of the plates along the sides of the wall element, as well as the screw configuration readily can be adapted to prevailing loading conditions. The slotted-in plate connection resists both horizontal wind force and wind induced uplift forces. Thus, elaborate tie-down arrangements are not needed. The structural features of the slotted-in connection are described in [4].

2.5.2 Wall junction and corner connections
The slotted-in plates are also applied for wall junction and wall corner connections. Internal wall elements with a double-ribbed panel carcass use T-brackets, Figure 4. Internal wall elements with a boxed plyboard carcass use angle brackets, Figure 5 and Figure 8. The number of brackets, screw configuration and location along the wall edges depend on prevailing loading conditions. The slotted-in brackets enable a more efficient stabilization of the building by transferring the wind induced up-lift forces from shear walls to transverse walls over the height of the wall and thus avoid local force concentration.

2.5.3 The floor hanger connection
The floor-to-wall connection is critical and needs special design attention as conflicting structural and functional demands coincide, e.g. securing load transfer, avoiding acoustic flank transmission and fulfilling the fire safety regulations. The hangers are preinstalled on the ends of the floor elements and are fastened by screws to the rim beam as shown in Figure 6 and Figure 9. The internal spacing of the hangers corresponds to the spacing of the I-beams of the floor element, normally 600 mm. At erection, the hook of the hanger grabs around the perimeter beam on the bearing walls (cf. Figure 7), enabling direct vertical wall-to-wall load transfer parallel to grain.

It is noticed that the design of the hanger connection suggested by the company can be critical, since there is no redundancy and the safety of the structure depends solely on the hanger in case of failure. The present design is critically examined and evaluated in [5] together with suggestions for improvements and a proposal for an alternative design.

3 ERECTION ON SITE
The wall elements are erected in storey-high sections and stabilized before the floor elements are put in place to form the deck which becomes the working platform for the next storey. After completion of each storey, all wall splices are sealed with an airstop tape to grant an intact and continuous building envelope.
4 FUNCTION AND ARCHITECTURE

The MFB system is designed with a holistic perspective, unifying diverse design options, safety and functional requirements.

4.1 Acoustic aspects

The concept of suspended floor implies that the floor load is the same for all storeys, and that the same type of acoustic damper can be used for all the hanger connections simplifying the design significantly. The acoustic dampers are pre-attached to the hangers, no additional adjustment or complementary installation is needed on the site. Hence, the risk for mounting errors during erection is low.

4.2 Sustainability and energy performance

The fact that buildings presently account for 40 % of the energy consumption, have led the European Union (EU), because of environmental and sustainability concerns, to introduce legislation to ensure significant reduction of energy use. The EU Directive on energy performance of buildings [6] calls on the member states to make national plans that by the end of 2020 ensure that new constructions are nearly zero-energy buildings, thereby reducing both energy consumption and carbon dioxide emissions. The plyboard panel and the I-beams/I-studs constitute a strong and tight structure and building envelope characterized by efficient use of raw material, good thermal and building physical features, making the MFB system suitable for buildings with very high energy demands.

4.3 Architectural freedom and design flexibility

The prefabricated wall units have a modularized design and can be customized with respect to size and configuration of openings (number, size and location) providing flexibility in organizing rooms' shape in plan, section and size. The stiffness and strength of the floor elements, allowing free spans up to 8 m, add further design options regarding spatial form and proportions. The wall elements allow the building envelope to be optimized regarding structural and physical performance, and both the wall and floor elements can be adjusted to meet specific safety and functional requirements (e.g. thermal, acoustic and fire performance) by modifying or changing components and internal layer composition.

5 CONCLUSIONS

The Masonite Flexible Building System combines the slenderness of a timber light-frame system with the strength and robustness of a cross laminated timber system. The erection of the elements of the system is simple and swift by efficient slotted-in steel-plate connections for the walls and steel hangers for the floors. The system is well adapted for multi-storey buildings.

ACKNOWLEDGEMENT

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Figure 8. Wall junction connection between external wall (left) and internal wall (right) using steel angle brackets and screw connectors. The angle brackets transfer part of the wind induced up-lift forces to the external wall, introducing a more efficient load transfer at wall junctions.

Figure 9. The XL floor hanger connection for the suspended floor to external wall viewed through a cut in the plyboard panel.

REFERENCES

A Simplified Fire-Risk Model for Canadian Mid-Rise Wood Constructions

Ling Lu¹, Christian Dagenais², Richard Desjardins³

ABSTRACT: This study developed a simple fire risk analysis model – “Fire Risk Index Method for Canadian Mid-rise Buildings (FRIM-CMB)” – to provide authorities having jurisdiction (AHJ) and designers with a means to develop alternative solutions for using wood components in mid-rise buildings.

KEYWORDS: Fire risk analysis, Fire Risk Index Method, Mid-rise wood building.

1 INTRODUCTION

The steep decline of the North American housing market has adversely affected the Canadian wood products industry. Under the Transformative Technologies Program which was established by Natural Resources Canada, FPInnovations has been working towards increasing the use of wood-based components in building systems for multi-storey constructions. To compete with other building systems, fire safety, as one of the main issues, can be addressed by a performance-based design, also called “alternative solution” in accordance with the 2010 National Building Code of Canada (NBCC) [1]. Prescriptive solutions within the NBCC historically permit residential wood constructions for buildings up to 4 storeys, providing an active fire protection system is installed.

British Columbia has increased the maximum height for wood-framed residential construction from four to six storeys. Other provinces in Canada, like Ontario and Quebec, are also on their way towards implementing six storey wood buildings. Until then, an “alternative solution” has to be provided when designing mid-rise wood-frame buildings (5-6 storeys). The alternative solution needs to demonstrate that it meets the minimum level of performance in the areas applicable to the “deemed-to-satisfy” prescriptive solution it replaces, thus a non-combustible construction.

In Canada, the National Research Council (NRC) had developed a complex fire risk analysis tool called FiRECAM while Carleton University has been developing a complex computer fire risk analysis model called CUrisk. Both models have been proved to be good research tools for fire risk analysis, but it needs considerable fire safety engineering knowledge to understand and use the models.

To promote mid-rise wood-frame buildings, it is necessary to develop a simple, yet comprehensive, fire risk analysis tool for authorities having jurisdiction (AHJ) and designers wishing to develop an alternative solution by using wood components, without the need to have all the full qualifications in fire safety engineering, but still having a minimum of expertise and knowledge in that field of practice. The use of such model can help in evaluating the level of performance (acceptable fire risk index) of wooden buildings when alternative solutions are applied by comparing its fire risk index to the prescriptive solution it replaces.

2 PREVIOUS WORK

2.1 FIRE RISK INDEX METHOD FOR MULTI-STOREY APARTMENT BUILDINGS

In 2002, Department of Fire Safety Engineering at Lund University in Sweden published a “Fire Risk Index Method for Multi-storey Apartment Building” (FRIM-MAB) [2][3][4]. The method has a hierarchy structure with consideration of the fire safety measures in a building. Each “decision making level” of the hierarchy has different “attributes” with components accounted for an acceptable portion of fire safety. The “attributes” are established based on NFPA Fire Safety Concept Tree [5].
The Fire Risk Index Method developed at Lund University determined by a Project group and a Delphi panel. The Project group has one member from each of the Nordic countries. The Delphi panel consists of 5 experts from 4 Nordic countries, for a total of 20 experts working in fire safety from various areas. The Project group formulates questionnaires, sends them to the Delphi panel and collects and summarises the results. Then the Project group formulates another new questionnaire and sends them to the Delphi panel members, the response from the Delphi panel members were used gradually to improve the structure of the Fire Risk Index Method and the weighing factors.

The top level of the hierarchy system is “Policy” which is to provide acceptable fire safety level in multi-storey apartment buildings. The next level of the hierarchy system is “Objectives” which are to provide life safety and to provide property protection. Then the next level of the hierarchy system is “Strategies” which are to control fire growth by active means, confine fire by construction, establish safe egress, and establish safe rescue. The bottom level of the hierarchy system consists of seventeen parameters which are: linings in apartment, suppression system, fire service, compartmentation, separating structure, doors, windows, facades, attic, adjacent buildings, smoke control system, detection system, signal system, escape routes, load-bearing structure, maintenance and information, ventilation system. Some of these parameters are divided into sub-parameters. Each parameters and sub-parameters can be quantified with grades from 0 to 5, with the highest and better grade being 5. Different weights are assigned to each attributes. A risk index is then calculated by summing up the grades of the seventeen parameters times their assigned weights to evaluate the fire risk of a building and compare with similar buildings or other design options. The structure of the Fire Risk Index Method and the weights assigned to each attribute are determined by a Delphi panel.

For a specified building, the user works through the decision tables and grades every parameter and sub-parameter, on the last page of the decision tables, the grades entered by the user are multiplied by weights. The weighted grades are summed up and a risk index is calculated. The method was evaluated by comparing the risk index with the risk rank calculated by a standard quantitative risk analysis. The comparison shows a good agreement between the two methods.

2.2 VALERIE

The Fire Brigade Department of Trento (Italy) and ISAQ Studio organized a Project called “VALERIE” [6][5]. This project developed a multi-level hierarchy structure with the assignment of the appropriate weight to each fire safety measure. The fire safety measures assessed in this method is very similar to that of the FRIM-MAB method, with primary differences arising that FRIM-MAB only addresses multi-storey apartment buildings whereas VALERIE addresses all building occupancies including industrial buildings.

The weightings assigned to the various measures are similar, but the FRIM-MAB method is more favourable towards combustible construction than the VALERIE method. As an example, FRIM-MAB considers the effect of fire stops in separations between compartments, not the VALERIE method.

3 FIRE RISK INDEX METHOD FOR CANADIAN MID-RISE BUILDINGS

3.1 INTRODUCTION

Although the FRIM-MAB method was developed in Europe using European fire safety strategies and test methodologies, the fire safety measures and approaches are quite similar to those of Canada.

Based on the European Fire Risk model, especially the FRIM-MAB [7], a Fire Risk Index Method for Canadian Mid-rise Buildings (FRIM-CMB) has been developed. The hierarchy structure of the index method is retained but the attributes for the components of each level are revised according to the requirements of NBC. Hereby the seventeen parameters (Pi) identified in FRIM-MAB are retained; the weighting factor (Wi) assigned to each parameter and the methods for assigning a grade to each parameter and sub-parameter have been reviewed and revised where necessary. Additional details on the development of FRIM-CMB method can be found in [7].

3.2 TOP LEVEL OF THE HIERARCHY SYSTEM - POLICY

Multi-storey buildings shall be designed in a way to ensure sufficient life safety and property protection in accordance with the objectives in the second level of the hierarchy system.

3.3 SECOND LEVEL OF THE HIERARCHY SYSTEM - OBJECTIVES

Based on the requirements of the objective-based 2010 NBC, the objectives are to provide fire safety and fire protection of buildings.

3.3.1 OS1 - Fire safety

This objective aims at limiting the probability that a person in or adjacent to the building will be exposed to an unacceptable risk of injury due to fire caused by:

- OS1.1 – fire or explosion occurring;
- OS1.2 – fire or explosion impacting areas beyond its point of origin;
- OS1.3 – collapse of physical elements due to a fire or explosion;
- OS1.4 – fire safety systems failing to function as expected;
- OS1.5 – persons being delayed in or impeded from moving to a safe place.
3.3.2 OP1 - Fire protection of buildings
Similarly to the OS1 objective, OP1 aims at limiting the probability that the building will be exposed to an unacceptable risk of damage due to fire caused by:

- OP1.1 – fire or explosion occurring;
- OP1.2 – fire or explosion impacting areas beyond its point of origin;
- OP1.3 – collapse of physical elements due to a fire or explosion;
- OP1.4 – fire safety systems failing to function as expected.

3.4 THIRD LEVEL OF THE HIERARCHY
SYSTEM - STRATEGIES
Four Strategies are adopted from the European method and defined as:
1. S1 - Control fire growth by active means;
2. S2 - Confine fire by construction;
3. S3 - Establish safe egress;
4. S4 - Establish safe rescue.

3.5 BOTTOM LEVEL OF THE HIERARCHY
SYSTEM - PARAMETERS
The bottom level of the hierarchy system consists of 17 parameters (P_i) associated with respective weighting parameters (W_i). Since the sum of the 17 grades weightings must equal 1, if all parameters were given the same value for W_i then W_i would equal 1/17, thus 0.05884 for all W_i. The relative importance of a parameter can be judged by whether its weighting is more than 0.05884 (a more important parameter) or less than 0.05884 (a less important parameter). The weighting parameters are mainly adopted from the European Method, while some parameters are changed according to the requirements of NBCC. The weight of the parameter is also changed accordingly.

3.5.1 P1 - Linings in apartment
This parameter is to define the possibility of internal linings in an apartment to delay the ignition of the structure and to reduce fire growth. It is assigned by the performance of the worst lining materials on walls or ceiling in a dwelling.

In Subsection 3.1.13 of NBCC, the fire performance of interior finishes for walls and ceiling is expressed in terms of Flame-Spread Ratings (FSR). To determine the flame-spread rating of a material, NBCC lists FSR for generic products of wall and ceiling finish materials (see Table D-3.1.1.A of NBCC). The FSR for finishing materials from non-combustible materials such as concrete to combustible materials such as wood and engineering wood products or plastics vary from 0 up to 150. For products that are not listed in Table D-3.1.1.A, the FSR of a material or assembly shall be determined on not less than three tests conducted in conformance with CAN/ULC-S102, “Surface Burning Characteristics of Building Materials and Assemblies” [8].

If low-density materials (≤ 100 kg/m³) are employed as room linings, whether they are combustible or non-combustible, they very effectively trap heat in the apartment causing rapid fire growth and early flashover. This index method has not been calibrated to address such scenarios.

3.5.2 P2 - Suppression system
This parameter is to define the equipment and systems for suppression of fires. Parameter P2 is assigned with a grade from 0 to 5 depending on which standard is used when Sprinkler System is installed. In Canada, two NFPA standards are required to be used in accordance with Article 3.2.5.12 of NBCC when installing automatic sprinkler system:
1. NFPA 13R “Standard for the Installation of Sprinkler Systems in Residential Occupancies up to and Including Four Stories in Height” Error! Source du renvoi introuvable., and;
2. NFPA 13 “Standard for the Installation of Sprinkler Systems” Error! Source du renvoi introuvable., which is more stringent than NFPA 13R.

Portable extinguishing equipment is not required for apartment buildings in the NBCC (refer to Article 3.2.5.16) and so unlike with FRIM-MAB it is not included in Canadian system. The two standards are very specific about where sprinklers must be employed, but there are two buildings, the Vancouver Convention Centre and the FondAction Building, that have been recently constructed by using alternative solutions and both employing enhanced sprinkler protection (beyond what is called up in the NBCC and NFPA 13). Therefore, the possibility of an enhanced sprinkler system is an option.

3.5.3 P3 - Fire service
The parameter P3 is to define the possibility of fire services to save lives and to prevent further fire spread and is divided into 4 sub-parameters. The sub-parameter P3a is intended to address the capability of the fire department. The sub-parameter P3b is intended to address response time fire department response time. The sub-parameter P3c is intended to address accessibility for the fire department. P3c is graded base on NBCC requirements on the accessibility to the sides of a building.

In NBCC, Clause 3.2.7.9 requires emergency power to supply power for firefighter to use elevator so as to reach the fire origin as quick as possible, although this is a requirement for high-rise building (above 6 storeys), it would be optional for other buildings. Therefore, a sub-parameter P3d is added to consider the availability of fire department elevator.

3.5.4 P4 – Compartmentation
Parameter P4 is to define the extent to which building space is divided into fire compartments. While the European FRIM-MAB model acknowledges the importance of the size of compartments in a residential building, it does not address the building area (maximum
In Canada, Article 3.1.5.5 of NBCC states that an exterior non-loadbearing wall assembly with combustible components such as wood is allowed in non-combustible construction if:

- Building height is not more than 3-storeys, or;
- If building height is more than 3-storeys sprinklered throughout and the interior surfaces of the wall assembly are protected by a thermal barrier;
- The wall assembly satisfied the testing with CAN/ULC-S134 “Fire Test of Exterior Wall Assemblies.”

What is addressed here is whether fire spreads along the façade of the building and hence could cause fire spread to higher storeys. Parameter P8 is graded according to its compliance with NBCC.

3.5.9 P9 – Attic

Parameter P9 is to define prevention of fire spread to and in attic. Sub-parameter P9a defines adequate protection of eaves while sub-parameter P9b defines fire compartmentation in attic.

3.5.10 P10 – Adjacent buildings

NBCC considers the limiting distance to the property line, the size of unprotected openings, etc. to address the fire spread to the adjacent buildings. Parameter P10 is graded according to the extent it complies with the requirement of NBCC in regards to spatial separations.

3.5.11 P11 – Smoke control system

Parameter P11 is to define the use of equipment and systems in escape routes for limiting spread of toxic fire products. Sub-parameter P11a defines the way to active smoke control system and sub-parameter P11b defines the type of smoke control system.

3.5.12 P12 – Detection system

Parameter P12 is to define the use of equipment and systems for detecting fires. Sub-parameter P12a defines the location and amount of detectors and sub-parameter P12b defines reliability of detectors.

The NBCC requirements for detection systems in residential buildings (particularly multi-family buildings) are enforced with much rigour by an AHJ for new construction. However, it is conceivable that some small allowances could be made for renovation of an existing building. The grading is proposed based on its compliance with NBCC requirements.
3.5.14 P14 – Means of egress
Parameter P14 is to define adequacy and reliability of escape routes. Sub-parameter P14a defines the type of escape routes, P14b defines dimensions and layout of an escape route, P14c defines the kind of guidance signs, lighting and emergency lighting and P14d defines the worst lining or flooring class that is to be found in an escape route.

In NBCC, the fire performance of interior finishes for walls and ceiling is expressed in terms of Flame-Spread Ratings (FSR). As with P12 – Detection System and P13 – Signal System, the NBCC requirements for the means of egress in residential buildings (particularly multi-family buildings) are enforced with much rigour by an AHJ for new construction. Again, it is conceivable that some small allowances could be made for renovation of an existing building. The grading is proposed also based on its compliance with NBCC requirements.

3.5.15 P15 – Structure – Load-bearing
Parameter P15 is to define the structural stability of the building when exposed to a fire. Sub-parameter P15a defines load-bearing member fire resistance capability of the building. The load-bearing member is tested by CAN/ULC-S101: “Fire Endurance Tests of Building Construction and Materials” [11], and is categorized by its fire-resistance rating (30 mins, 45 mins, 1 h and 2 h). Sub-parameter P15b grades the combustibility of insulation and load-bearing member in the building. When both load-bearing member and insulation are combustible, it gets the lowest grade, when both load-bearing structure and insulation are non-combustible, it gets the highest grade.

3.5.16 P16 – Maintenance and information
Parameter P16 is to define the inspection and maintenance of fire safety equipment, escape routes, etc. and information to occupants on suppression and evacuation.

Sub-parameter P16a grades maintenance of fire safety systems, P16b grades inspection of escape routes and P16c grades how information is available to occupants on suppression and evacuation.

In Canada, NBCC governs the design and construction of a building for fire safety. The National Fire Code of Canada (NFCC) governs the operation of a building to ensure fire safety during its use. Maintenance and information are to be spelled out in a Fire Safety Plan that is to be developed by the building owner or operator in compliance with the NFCC. Usually such a plan is prepared during the design stage and reviewed by the AHJ and local fire department. The local fire department is to inspect the building occasionally during its use to ensure maintenance is being carried out in accordance with sprinkler, detection, and alarm standards. They also ensure that fire escapes are not blocked or illegally locked from the inside and that the fire safety plan (including information for evacuation) is implemented.

So what the maintenance depends upon are the resources available to the local fire department (to inspect) and the resources available to the operator of the building (to ensure ongoing compliance). The parameter P16 is graded based on availability of local fire department and on-site management.

3.5.17 P17 – Ventilation system
Parameter P17 is to define the extent to which the spread of smoke through the ventilation system is prevented.

3.6 PARAMETER SUMMARY TABLE – FIRE RISK INDEX METHOD
Grades for each parameter has been inserted in Table 1 and multiplied by the weighting factor. Maximum individual grade for each parameter is 5.00. The weighted grades for all parameters are then summed and resulted in a score with a maximum value of 5.00. The weighting factors are kept the same in Canadian Method as in Europe.

Table 1: FRIM-CMB Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Weight Grade</th>
<th>Weighted Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1 Linings in apartment</td>
<td>0.0576</td>
<td></td>
</tr>
<tr>
<td>P2 Suppression system</td>
<td>0.0668</td>
<td></td>
</tr>
<tr>
<td>P3 Fire service</td>
<td>0.0681</td>
<td></td>
</tr>
<tr>
<td>P4 Compartmentation</td>
<td>0.0666</td>
<td></td>
</tr>
<tr>
<td>P5 Structure - separating</td>
<td>0.0675</td>
<td></td>
</tr>
<tr>
<td>P6 Doors</td>
<td>0.0698</td>
<td></td>
</tr>
<tr>
<td>P7 Windows</td>
<td>0.0473</td>
<td></td>
</tr>
<tr>
<td>P8 Facades</td>
<td>0.0492</td>
<td></td>
</tr>
<tr>
<td>P9 Attic</td>
<td>0.0515</td>
<td></td>
</tr>
<tr>
<td>P10 Adjacent buildings</td>
<td>0.0396</td>
<td></td>
</tr>
<tr>
<td>P11 Smoke control system</td>
<td>0.0609</td>
<td></td>
</tr>
<tr>
<td>P12 Detection system</td>
<td>0.063</td>
<td></td>
</tr>
<tr>
<td>P13 Signal system</td>
<td>0.0512</td>
<td></td>
</tr>
<tr>
<td>P14 Escape routes</td>
<td>0.062</td>
<td></td>
</tr>
<tr>
<td>P15 Structure – load-bearing</td>
<td>0.063</td>
<td></td>
</tr>
<tr>
<td>P16 Maintenance and information</td>
<td>0.0601</td>
<td></td>
</tr>
<tr>
<td>P17 Ventilation system</td>
<td>0.0558</td>
<td></td>
</tr>
<tr>
<td>Sum</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Score (Sum of weighted grades)

RISK INDEX (= 5 – Score)
Parameters having more relative importance can also be seen from Table 1 by those having a weight greater than 0.05884. The Risk Index is defined as 5 minus the score. A low Risk Index means a low risk, thus a high fire safety level.

4 CASE STUDY

Simple fire risk analyses have been performed in order to compare various design scenarios by using “deemed-to-satisfy” prescriptive non-combustible construction requirements as set forth in NBCC.

A design example of a typical residential building protected with automatic sprinklers in accordance with NFPA 13, with fire compartment ranging from 50 to 100 m², using non-combustible structural elements with a FRR of at least 1 hour and non-combustible insulation exhibiting a FSR of less than 25, provides a fire risk index in the order of 1.121.

An alternative solution consisting of changing only the combustibility of the structural elements (sub-parameter P15b) provides a risk index of 1.162, which is very similar (about 0.8% fire risk difference), but yet not quite equivalent. It should be reminded that the building area is not considered in the FRIM-CMB method. Such an additional parameter is likely to largely influence the risk index of such alternative solution, provided that the building area is lower than the allowed area of the “deemed-to-satisfy” non-combustible solution.

A simple design adjustment using an enhanced sprinkler system (i.e. using a higher fire risk hazard per NFPA 13) provides a much better risk index of 1.095, which is actually better than the “deemed-to-satisfy” prescriptive non-combustible construction solution intended to be replaced. Another easy way to improve the risk index of the alternative solution is to enhance the escape routes (egress) above the minimum NBCC requirements. By doing so, a new risk index of 1.100 is obtained, which again seems to provide a safer solution than the prescriptive non-combustible solution.

5 CONCLUSIONS

The FRIM-CMB method developed in this study provides a simple fire risk analysis tool for Canadian authorities having jurisdiction (AHJ) and designers seeking to develop an alternative solution for using wood components beyond the current allowance in the prescriptive solutions of the NBCC, such as in mid-rise buildings.

A simple case study reveals that by only changing the combustibility of the structural elements, while keeping all other building fire safety features identical, provides a risk index about 0.08% higher than when using non-combustible structural elements. Thereby, a building designer can easily adjust his design accordingly by enhancing very few fire safety measures in order to demonstrate that the alternative solution meets the minimum level of performance (risk level) attributed to the “deemed-to-satisfy” prescriptive solution it replaces, thus a non-combustible construction.

Further case studies are to be developed within the next years in order to validate and refine the FRIM-CMB method.

ACKNOWLEDGEMENTS

Financial support for this study was provided by Natural Resources Canada (NRCan) under the Transformative Technologies Program, which was created to identify and accelerate the development and introduction of wood products in North America. FPInnovations expresses its thanks to its industry members, NRCan (Canadian Forest Service), the Provinces of British Columbia, Alberta, Saskatchewan, Manitoba, Ontario, Quebec, Nova Scotia, New Brunswick, Newfoundland and Labrador, and the Yukon Territory for their continuing guidance and financial support.

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LOW COST PHOTOGRAMMETRY AS A TOOL FOR STIFFNESS ANALYSIS AND FINITE ELEMENT VALIDATION OF TIMBER WITH KNOTS IN BENDING

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ABSTRACT: This paper presents the utility of the low cost photogrammetry as a tool for stiffness analysis and finite element validation of wood with knots in bending. Three consumer-grade digital cameras of 10 megapixels were used. The synchronization was solved by a prolonged shutter opening in darkness conditions and a high speed flash. An equipment of approximately 2900 US Dollars provided an accuracy of 1 to 6200 mm/mm. Due to the size and multiaxial stress state of the experiment no stresses or displacements could be measured. However the average displacement of 65 finite element nodes and the distortion of the bending curve caused by the knots could be reliably obtained. A methodology for a more trustworthy collection of MOE is proposed. This method distinguishes a Clear MOE (regarded to the areas with no defects), a MOE in Knots (regions of knots), and a Global MOE (encompasses the previous two). Using the Clear MOE instead of the MOE measured at the midpoint, the simulations improved up to 49%.

KEYWORDS: Low cost photogrammetry, bending test, stiffness analysis, finite element validation, knots, modulus of elasticity.

1 INTRODUCTION

Wood is a highly heterogeneous material and its mechanical properties are strongly affected by the presence of defects, variations in density, and other non-homogeneities. Hence, the determination of mechanical properties and experimental validations usually require the performance of numerous tests until one gets reliable values, involving significant economical and environmental costs, as well as large time investments. Therefore, close range photogrammetry comes up as a very useful tool in timber to be able to measure mechanical properties, 3D displacements, stresses, or deformations in many points of the same member, as proposed in various experiments [1-14].

Additionally, in recent years the consumer-grade digital cameras have been greatly improved, and wood is considerably less stiff compared with other materials. Coherently it is suggested that low-cost photogrammetry could be introduced in timber to evaluate its mechanical properties, or carry out experimental validations with high accuracy and little investment.

Accordingly, in this research an inexpensive photogrammetric equipment composed basically by 3 consumer-grade 10-megapixel cameras was used. The objective was to assess the utility of the low cost photogrammetry for stiffness analysis and finite element validation of wood numerical models.

A FE model which accounts the behavior of structural beams with knots under 4-point bending tests was used. The presented methodology and analysis include a novel determination of the MOE taking into account the influence of knots in deflection which could be useful for common flexural grading tests, such as ATSM D 198 [15].

2 MATERIAL AND METHODS

2.1 MATERIAL AND METHODS

9 beams of 3000×150×50 mm of Pinus sylvestris were tested in a 4-point bending tester; 3 consumer-grade Canon Eos 400D cameras with resolutions of 3888×2592 pixels, Canon Inc., Ōta, Tokyo, Japan; 5 extensometers; commercial FE software, Ansys
Multiphysics v11, Ansys Inc., Canonsburg, PA, USA; PhotoModeler Scanner v6 2.2.596 photogrammetric software and its coded targets, Eos Systems Inc., Vancouver, BC, Canada; black thumbtacks; a Metz 45CT flash, Metz-Werke GmbH & Co KG, Zirndorf, Bavaria, Germany; and one relay and two switches. The total investment in photogrammetric equipment was approximately $2900 US dollars.

2.2 EXPERIMENTAL PROCEDURE

An average of 65 nodes of the FE model were marked physically in the central third of each beam. Two criteria were considered in the selection of measuring points. First, in areas not influenced by the presence of knots (clear wood), the closest nodes in the FE model with respect to a regular 60×60-mm square mesh were selected. Second, in regions affected by knots, the FE nodes selected were those in which significant stress changes or the highest stress intensity was expected. A minimum distance of at least 15 mm between FE nodes was constantly maintained.

Once the measuring points were selected, the mesh was printed in true scale, and photogrammetric coded targets were located on the clear wood with black thumbtacks positioned in regions affected by knots, as shown in Figure 1, so that the FE nodes were accurately marked. The photogrammetric coded targets provided high accuracy and automated processing but low measuring density. Alternatively, the black thumbtacks did not achieve great accuracy and automation, but allowed for an increase in the measuring density and an ability to visualize the fracture initiation. Furthermore, the use of these two marking techniques allowed one to easily discern the behavior in the regions with knots and in those regions that were defect-free.

The three digital cameras were placed at approximately 1200 mm in front of each beam, taking a photo each 5 seconds. The synchronization between cameras and test machine was solved carrying out the tests in darkness and using an electrical circuit composed by a relay and two switches, which allowed a prolonged shutter opening of all cameras while a high speed flash of 1/14000 seconds was triggered, and the screen of the test machine was captured. In this way all the pictures were taken at exactly the same time, and the load of the test machine in each photogram was known.

A perfectly flat board containing 90 photogrammetric coded targets was used to orient, scale and rotate the measuring points during photogrammetric processing. The coordinates of this flat board were known after measuring with a digital caliper of 0.01 mm sensitivity. By positioning this board precisely perpendicular to and just below the middle third of each beam, all of the cameras could record this perfect plane in each frame. Of the 90 additional coded targets, 81 were used as control points by introducing their coordinates during the photogrammetric processing so that the pictures could be scaled, rotated and oriented with certainty. The accuracy of the photogrammetry was evaluated by two methods: first, it was assessed statically by contrasting the coordinates of the 9 coded targets on the board that were not used as control points, and second, it was evaluated dynamically by adhering the coded targets to 5 extensometers and comparing the measured displacements. One of them was arranged in the axial direction to the beams to evaluate the accuracy in the X direction, 3 were placed in the vertical direction (Y direction), and finally one was placed in the lateral direction (Z direction).

3 RESULTS

Accuracy, which refers to the comparison of the point coordinates that are estimated photogrammetrically with the actual values that are measured physically, was calculated as the average absolute error during the 9 bending tests. It was obtained with an error of 0.216 mm in the X direction, 0.143 mm in the Y direction and 0.141 mm in the Z direction. Given the size of the photographed plane at the beam position, the relative absolute error can be determined to follow a relationship of 1/6200 mm/mm. Accordingly, the stresses and deformations, given the multiaxial stress conditions and the size of the experiments, could not be assessed but the displacements could be reliably evaluated. In addition, the load value, the location, and the cause of the fracture initiation could be determined and utilized. This information was essential for the validation of this model since, when many knots are present it is not always easy to accurately determine which defect causes the initial failure and in what location and at which load the process begins.

By using the true MOE in the first simulations, it was found that if the FE nodal displacements were thoroughly contrasted in all of the photogrammetrically measured nodes, rather than just evaluating the deflection at the midpoint with one extensometer, the average error in all beams was 8.5% higher. However, the average error in the defect-free areas was similar to that in the regions of knots, so it is suggested that the distortion caused by knots was rightly simulated. This strong increase should be attributed similarly to the
heterogeneity in the clear wood and the presence of defects and other non-homogeneities as the knots, so that the stiffness was undeniably changed. Thus, the theoretical deflection curve can be highly distorted, and the MOE distribution can be significantly heterogeneous as well. Indeed, it was found that when the photogrammetrically measured deflection curve was compared to its homologous theoretical curve calculated from the same true MOE, the average variability in the error, estimated from the standard deviation of the percentage errors, reached a value of 1.5%. Consequently, assuming a normal distribution of error, there could be differences in the displacement of up to 6% within 95.4% of the points. That indicates that if an FE validation or MOE measurement is carried out from just one point, the results will be more-or-less distorted.

In this way to obtain a MOE which could better represent the stiffness of each beam, the known equations of strength of materials to calculate the deflections \((y)\) and angles \((\theta)\) in four point bending tests were used. By an iterative process, the MOE of each photogrammetrically measured node, can be estimated for any deflection \((y)\) and horizontal coordinate \((x)\):

\[
y_{x=a-d} = \frac{Pa(3x - 3x^2 - a^2)}{6EI} - \frac{6Pa}{5AG} \tag{1}
\]

\[
\varphi_{x=a-d} = \frac{Pa(l/2 - x)}{EI} \tag{2}
\]

Where \(P\) = load at each load point, \(a\) = length from reaction to the nearest load point, \(l\) = span length, \(x\) = distance from the supporting point, \(A\) = cross-sectional area, \(I\) = geometrical moment of inertia, \(E\) = Young’s modulus and \(G\) = longitudinal shear modulus which was estimated as \(E/16,075\).

![Figure 2: Schematic representation of Clear MOE, MOE in Knots, and Global MOE.](image)

Averaging this calculations for sequence of pictures from the early stages to the estimated proportional limit, and discerning the nodes which are not influenced by the distortions of knots (coded targets), from the remaining points (thumbtacks), it was possible to calculate an average modulus of elasticity for the free-defect areas (Clear MOE), an average MOE in the regions distorted by the presence of knots (MOE in Knots), and the average MOE in the central third of each beam (Global MOE), as schematically represented in Figure 2.

When using the Clear MOE in the simulations rather than the true MOE calculated from the deflection at the midpoint, the average improvement of estimations was 49%.

4 CONCLUSIONS

The experimental procedure described in this paper provided an accuracy of 1/6200 mm/mm with a photogrammetric equipment of only 2900 US Dollars. This did not allow contrasting stresses or deformations in a multiaxial stress state for structural sizes. However interesting possibilities for experimental validation were found: a thoroughly field of 3D displacements was obtained, aside from monitoring the value and location of failure.

The theoretical deflection curve can be highly distorted by the presence of knots and other non-homogeneities. If the MOE is measured only at the midpoint and rigorous analysis is performed (e.g. a FE validation) the conclusions could be erroneous. The photogrammetry can be very useful in such case. Consequently, by means of the presented experimental procedure, it was proposed the calculation of a Clear MOE, MOE in Knots, and Global MOE. Using the Clear MOE in the FE model rather than the conventional true MOE the predictions of improved up to 49%.

In brief the application of low cost photogrammetry has shown the following advantages: 1) the displacements (and deformation or stresses) and mechanical properties can be massively measured in 3D in any external surface, this can be specially interesting when measuring stiffness; 2) that enables to minimize effect of heterogeneity; 3) so that the uncertainty and the required number of tests is significantly decreased; 4) nevertheless, the value and location of the failure load as well as the cause of rupture can be reliably determined; 5) and the pictures can be saved for further analysis.

Finally, the main disadvantage was the low automation of the whole process, but looking to the future it is expected that this could be easily improved and, most likely, the accuracy of the digital cameras will still increasing, so that the described method could be applied to more routinely uses as the common grading tests.

REFERENCES


CASE STUDY HOUSES 2.0: MASS-CUSTOMISED MULTI-STOREY TIMBER BUILDINGS – Competitive high-rise timber structures from an architectural and engineering point of view

Magnus Larsson¹, Alex Kaiser², Ulf Arne Girhammar³

ABSTRACT: The authors present a rule-based system that applies to the architecture of innovative tall timber buildings, and provide examples of interesting multi-storey timber buildings that can be constructed through the application of those rules. In addition, they discuss how mass customisation is a driving force for this kind of architecture. Using the famous 1945-1966 Case Study Houses Program (CSHP) as a starting point for their investigations, they proceed to argue that the time has come for a new iteration of that program to be carried out: an updated, contemporary version that seeks to promote the virtues – from sustainability to cost effectiveness, from reduced foundations to flexibility and adaptability over time – of engineered timber architecture in general, and high-rise timber buildings in particular, in a way similar to the steel-and-glass structures advocated by the initiators of that original scheme. Following from this, the notion of mass customisation as a preferred method for the construction of these new “Case Study Houses 2.0” buildings is investigated in further detail, from its historical background via current-day applications through to informed speculations on how the technology could be used in the near future as part of a production chain that might support the construction of these mass-customised multi-storey timber buildings. Finally, examples of how contemporary digital technologies and manufacturing processes can further enhance the final execution of the building typologies through mass-customisation at the detail scale are discussed, showing how such implementations impact on the project.

KEYWORDS: Wood architecture, multi-storey timber buildings, Case Study Houses, mass customisation, modular systems, Martinsons CLT building system

1 BEYOND “BETTER LIVING”

Focused on the amalgamation of highly ordinary concepts into new systemic models, the Case Study Houses 2.0 (CSH 2.0) project oscillates between notions of mass customisation and the construction of a rule-based system that can be used to generate architectural designs for innovative tall timber buildings.

An architectural call to arms for “better living” has become a cliché.¹ However, using the legacy of these iconic buildings as an instrument for future speculation allows interesting comparisons to be drawn between the present scheme and one of the most famous experiments in residential architecture ever undertaken, leading to an extension of the Case Study House program into our day and age, using a Swedish timber building system.

2 THE CSH PROGRAM (1945–1966)

The Case Study House Program (CSHP) was an editorial experiment in modernist residential architecture commissioned by the avant-garde monthly magazine Arts & Architecture. The program ran intermittently from the announcement by the magazine’s editor and publisher, John Entenza, in January 1945, until 1966. It aimed to introduce the Californian middle class to the modestly-scaled beauty of modernism: reductive attitudes and simplicity of form, rational planimetric organisations, industrial materials and construction methods, seamless connections between inside and out.

Leading architects of the day were asked to design and build inexpensive and efficient model homes – experimental prototype buildings – for actual post-war clients, providing the building industry with models to be used in what Entenza viewed as an inevitable residential housing boom in the United States following the housing shortages of the Great Depression, the end of World War II, and the resulting return of millions of soldiers from Europe. The highly publicised program sought to use materials donated from manufacturers to create low-cost modern housing prototypes for the general public. The ethos behind the program was thus firmly based on and framed by industrial materials and construction systems – new possibilities that opened up
due to the increased economic prosperity and technological advances of the period.
The first six houses were built by 1948 and attracted more than 350,000 visitors. While not all 36 designs were built, a majority was, and most of them in Los Angeles; a few are in the San Francisco Bay Area, and one was built in Phoenix, Arizona. A number of them appeared in the magazine in iconic black-and-white photographs by architectural photographer Julius Shulman. The best of them are windows into the future, strikingly contemporary-looking even today, more than half a century later. Few of them (if any) served their intended function as replicable prototypes.

Entenza’s Case Study Program announcement stated that each “house must be capable of duplication and in no sense be an individual ‘performance’... It is important that the best material available be used in the best possible way in order to arrive at a ‘good’ solution of each problem, which in the overall program will be general enough to be of practical assistance to the average American in search of a home in which he can afford to live”.\(^2\)

If despite its internal variations we are to describe the underlying conceptual framework of this design research program, we soon realise that it is based on a series of rules. The buildings designed during the program’s first three years are of comparatively modest scale, bearing in mind the wealth of the clients they were built for. They are simply constructed using or pretending to use modular systems and inexpensive materials (though this is also at times a matter of style – as William W. Wurster and Theodore Bernardi famously puts it in the description of their CSH#3: “…although plywood costs more than plaster, we like it better because it looks cheaper\(^3\)). They utilise extensive glazed partitions to achieve a relaxed and pronounced indoor-outdoor emphasis. They aim to employ modular, standardised parts and apply industrial construction methods and materials to residential architecture, and are to some extent conceived as prototypes for mass production. They are constructed using the best technology of the day, and designed for a maximum degree of flexibility on the part of the users. They try to integrate house, furnishings, and landscape into a coherent whole.

In 1949, the designs of the Case Study House Program began to break away from the vocabulary of local Los Angeles modernism as defined by architects such as Irving Gill, Frank Lloyd Wright, R. M. Schindler, and Richard Neutra. To an extent, the times finally caught up with the architects: the economy turned and industrial materials such as steel were now widely available. The resulting CSHP buildings maximise and render overt the use of technology in their design and construction. A third phase is added in the 1960s, when the clients are more affluent and the buildings expand into expansive, generous, airy modernist homes that are almost sumptuous in their use of materials, clearly designed with little regard to Entenza’s initial vision of producing replicable prototypes.

The present project started with an analysis of all 36 Case Study Houses. It soon became clear that it was in those golden years leading up to and just at the beginning of the third phase that the program came closest to not only investigate but actually build full-scale models of the new, the untried, and the unchartered; ideas about how an advanced and advancing residential architecture could respond to social issues as well as how to apply the new materials and techniques of the wartime economy, including rule-based mass production of prefabricated units, to the post-war era. Such ideas are embodied, in the works of three Case Study architects – the husband-and-wife team of Ray & Charles Eames, Richard Neutra, and Pierre Koenig – which we will now discuss in further detail.

2.1 CSH#8 (EAMES HOUSE)

Charles and Ray Eames were themselves the clients for their own Case Study House, and they chose to make it an exercise in rule-based prefabrication and mass manufacturing. Designed to serve as their combined home and studio, it promoted a unique live-work lifestyle, with the couple living amongst their own projects at various states of completion.

As Charles Eames (with Eero Saarinen) initiated the design process in 1945, the idea was to construct the building entirely from "off-the-shelf" parts from steel fabricators catalogues (elements not originally made with housing in mind). However, immediately after the war this ideal of experimentation with industrial materials (steel, glass, asbestos, and Cemesto board) and construction systems was impossible to complete. It would take until 1949 for the necessary parts to become available. During these years, the initial scheme was radically changed from a dramatically cantilevered box to its final, calmer expression. Once the steel had been delivered to the site, Charles Eames decided to redesign the building, using the same components. Architecture theoretician Beatriz Colomina has commented that this idea of rearrangement runs through the entire production of the Eames’s practice.\(^4\)

Tucked sideways against a retaining concrete wall, the two-storey structure holds bedrooms on the upper level, overlooking the double-height living room. A courtyard separates the residence from the studio space, turning the building into a double-height pavilion cut into two adjacent volumes. The bold façade – strictly divided into eight + five bays separated by four bays of open space, and sitting on a perfect structural grid – holds a composition of brightly coloured panels between thin, black steel columns and braces, sharply contrasting with the surrounding landscape. Arguably it is this contrast, the programmatic live/work dichotomy, as well as the opposition between the formal stringency and the variety of textures, colours and materials introduces within its structural rigidity that makes this building a highly personal take on the prefabricated, mass manufactured catalogue house, and, in a way, a precursor to the mass customisation of our day.
2.2 CSH#20

While he was the most well-known of the CSHP architects, Richard Neutra’s early designs (the “Omega” and “Alpha” houses) had remained unbuilt. The Bailey House is extremely small, yet elegant in its simplicity. A rectangular or L-shaped plan split into sliding strips, a modest face to the street, and huge sliding glass walls opening up to the public space that is the leafy garden at the back. Glass, steel, wood, and brick.

If the raison d’être of the Eames House is to showcase the potential for individualised serialised production using prefabricated components, the Bailey House is first and foremost a programmatic exercise. The clients – a young and small family with restricted means but anticipating the possibility of growth and increased resources – commissioned Neutra to design three additions to the house over the coming years. Future expansion was thus a key word right from the start, and the original design easily accommodated these additions. Neutra wrote himself in the accompanying Arts & Architecture article about how the smallness of the building “may be stretched by skill of space arrangement and by borrowing space from the outdoors in several directions”. This directionality makes for quite a rich diversity of different outdoor spaces.

The roof is louvered over the car port, the interior panelled in “natural wood of variation in grain, colour, and finish,” and the square-section roof gutters are allowed to stick out at a liberal distance from the eaves of the flat roof. Another important feature is the prefabricated utility core, “a ready-delivered packaged mechanical unit that contains the centrally amassed plumbing and heating installations”.

While the Bailey House was designed for a middle class family, and while it has very little of the Eames House’s rearrangeable modularity, it is still a good example of how a flexibly designed rectilinear arrangement of spaces answered to Entenza’s brief for housing that catered to the average American (doctor). It also shows how even a small plot can be divided, or cut, into programmatically interlinked spaces, and how the intelligent use of implied (exterior) space can be used to make a building feel more spacious than it is.

2.3 CSH#21 AND CSH#22

Koenig designed his two Case Study Houses in 1960. Both CSH#21 and CSH#22 were constructed on dramatic sites offering great views of Los Angeles.

The charcoal-painted framework of CSH#21 stands in stark contrast to its white walls (in fact curtain walls with an exterior steel decking combined with an interior gypsum board). An island core divides the living area and the two bedrooms. The plan for this severely compact 120m2 steel-and-glass box is strictly grided into a six-by-four-units bay system, revealing a strict, geometric arrangement of horizontal surfaces, the flooring materials – concrete, tiles, mosaic tiles, carpet, brick, gravel, water – clearly divided into overlapping island-like arrangements positioned within steel frames. Prefab elements (“readily available steel shapes and products”) are used to demarcate the structure from white surfaces. All frames were shop-fabricated and then delivered to the site. It was actually created as a prototype for affordable, mass-producible housing.

That was not the case with CSH#22, which is a two-bedroom 204m2 one-off building. One of the most famous buildings in the US, it has appeared in more than 1,200 newspaper and magazine articles, journals and books, not to mention a slew of films, TV shows and commercials. Indeed, for many this house is eternally frozen in the 1960 moment when Shulman photographed two pretty girls sitting in the cantilevered section of the building, with the street lights of LA glittering in the distance.

3 CSHP 2.0 – THE ECHO OF THE ICON

The world was different in 1945 than today. But a few crucial things point toward there being similarities to the Case Study House post-war era and our political situation of today. Then as now, there was a big migration to the cities, as men returned home from the war. Today, more than half of the world’s seven billion people live in cities. The urban population jumped from 29% in 1950 to 50.5% in 2005. Then as now, the world is a pessimistic and uncertain place. Demonstrations, uprisings, and revolutions sweep the globe, the Occupy movement clog financial arteries, the Arabic rebellions spread across the Middle East, Greece’s economy is again devastated (as it was after WWII), there are riots in London and clashes in Moscow. At the same time, then as now, we live in an optimistic time of unprecedented scientific and socioeconomic progress, with progress being made at an extraordinary rate. And then as now, new building technologies inspire architects to dream up new structures for new ways of living.

However, that’s where the similarities stop. While we use the CSHP as our starting point and adopt some of its tenets (the house as reproducible prototype, brilliant spatial organisation within limited volumes, a stringency in the choices of materials and structural systems, a willingness to experiment with the latest technologies), there are others that we have chosen to update. First, a Case Study House for today has to become a Case Study Housing project; we believe cities need higher densities than the original single-family houses (Fig 1). Second,
we have hopefully done away with the sexism latent (or blatantly expressed) in the schemes of that generation. Third, environmental concerns put timber way ahead of steel. Fourth, the buildings need to become actual repeatable prototypes rather than one-offs. Fifth, we believe the main buzz word needs to change from mass manufacturing to mass customisation.

4 MASS CUSTOMISATION

The term ‘Mass customisation’ was coined by writer Stan Davis in his book Future Perfect but popularised by writer Joe Pine.9 Already ten years ago, in 2002, Bruce Sterling argued that the post-industrial model is not an assembly line, but an assembly swarm. The essence of his argument was that while in the industrial days of the factory, raw materials were banged into identical commodities, with the advent of the computer – “a flexible factory that makes ones and zeros” – the customer can be made to do the banging, as well as buying his or her own work as “consumer choice”. The more the product is “decomposed” into modular components or subsystems, the more it can be recomposed and customised at mass-production price. “Millions of potential combinations have replaced the standard product,” writes Sterling, while also pointing out the customer is “decomposed” into modular components or subsystems, the more it can be recomposed and customised at mass-production price.

In a post-industrial mass customisation society, consumers design their own products (or services) by choosing from a wide variety of features. Nothing is assembled until payment is received. The consumer becomes, in futurologist Alvin Toffler’s portmanteau, a prosumer (prosumption = production + consumption).10 Similarly, the task of design will also change,” writes Tim Crayton. “Clearly customization will have to be designed into products. The design task shifts from designing definitive invariable products to designing product platforms and architectures and the sets of design rules that define a range of product-solutions. Similarly, the new product design process will also include designing the design tools and interfaces for consumers as co-designers that will configure or determine the kinds of choices consumers will make and perhaps simulate the actual product.”12

This is a topic aligned to the thinking of architectural history professor Mario Carpo, who argues that the transition to a digital production chain is one from the production of identical copies of objects to mass customisation and design collaboration. Both the subject and the object of architectural design will have to change with the advent of digital technologies: the romantic notion of the designer in full control is dead, and the object need not be singular and specific. Designers must choose between either designing objectiles (four-dimensional algorithmic constructs with infinite variational possibilities; a term borrowed from Gilles Deleuze via Bernard Cache) or objects themselves. The objectile route is the only one offering some kind of authorial control.13

The present project possibly represents a combination of all these three arguments, in that it is based on design rules that can be programmed to produce objectiles, and that could cater for a prosumer inhabitant. In fact, mass customisation would be a driving force necessary to produce an architecture such as the one proposed here. While Charles Eames could find all elements necessary to build CSH#8 within a steel manufacturer’s catalogue, no manufacturer will hold all of the different panels produced by our system as ready-cut boards. They will each have to be cut according to a specification, developed in line with today’s streamlined relations between designing and fabricating.

5 PLAYING BY THE RULES

Ordering our environment according to rules is a way of simplifying the world around us and making it comprehensible, but it is also a way of testing ideas, generating forms, and reaching conclusions. As artist Sol LeWitt wrote, “the idea becomes a Machine that makes the art”.14 Many architects have invented self-referential rules that have to do with architecture itself, the interiority of the practice. However, architecture is also subject to socioeconomic and technological pressures that call for a continual rewriting of the rules. For instance, new materials, or technological progress, or increases in the profitability of land development have necessitated changes in architectural rules.15 It was a combination of those two factors – an internal need for organisation that would both help define forms and volumes, and create the necessary framework for an efficient use of mass customisation, and the external pressures of designing for an age in which certain environmental and social stresses call for systemic responses – that made us create our own architectural rule system to produce the first Case Study Houses 2.0 building.

6 CSH2.0 RULES: CUTTING VOLUMES

The rule system that produces our CSH2.0 buildings is entirely based on the idea of the cut as spatial generator and alternative way of creating a stacked structure. While we have previously studied different stacking methods – an additive process – as our main strategy for creating tall timber buildings, we are now focusing on investigating a subtractive paradigm, cutting away sections of an imagined spatial block that fills up an entire floor volume in order to create different spaces, circulatory paths, and service connections, as well as providing means for the building to react to external forces of different kinds and at different scales. The architectural cut is a strategic incision into a material mass (or indeed into a void) that simultaneously opens and connects. It is an incision of variable width that coincidentally fuses and separates, bridges and divides, connects and pulls apart. The cut is energetic, threatening, revealing. It can be explicit or implicit,
making an obvious mark or leaving a faint trace. It splits an integer volume into two, creates new edges, new spaces, new surfaces, new possibilities for the flow of people, energy, and light between its newly-created entities. The cut also instigates movement, allowing one body to slip along a fresh edge.

As we are working within the aesthetic context of the Case Study House Program, we are only utilising orthogonal cuts, and refraining from cutting at a vertical angle: all of our cuts are in one straight direction from top to bottom. Through manipulating the volume using varying incisions from our catalogue of architectural cuts, we achieve not only actual spatial differentiation but also possibilities for different implied demarcations of space that correlate to some of the Case Study Houses: slight shifts along the edge between horizontal surfaces call to mind the bridges and platforms in Pierre Koenig’s CSH#22, surface incisions made into the mass-customised wall panels echo the vertical composition in the Eames House, repetitive cuts create louver-like elements reminiscent of Richard Neutra’s CSH#20. As one whole floor volume has been cut through, another is placed on top, and new incisions are made. This layering of cut volumes leads to an interesting spatial configuration as one cut is stacked on top of another, either lining up with each other or being offset against one another, turning the entire volume into a vertical labyrinth of crisscrossing cuts.

Our catalogue of architectural cuts for the first Case Study House 2.0 building corresponds to external and internal forces (stresses, pressures), onto the building. The former are to do with how the building functions on a societal level, and seeks to incorporate factors such as environmental concerns, cost characteristics, social components, cultural considerations, and climate-related aspects. The latter are to do with how the building functions as an object, and integrates formal aspects, organisational principles, and aesthetic concerns. The three different cuts shown in (Fig. 2) are (left to right):

- Tapered cut (a cut wider at one end than the other, producing private/public space),
- Positive/negative cut (the negative cut slices through a volume, the positive cut through a void creates a tunnel-like space),
- Circulation cut (a cut that opens up a circulation path between volumes).

Our layered approach means we can introduce different cut strategies on different floors, and use the system to generate iterations of interesting moments of formal, programmatic, functional, and architectural significance into the building. The resulting configurations can be shifted up and down across the building envelope until we reach an optimal series of interacting levels, and the structure is vertically flexible in that it is easy to add another storey at any time.

While the panelisation of the entire building is by necessity a mass-customised process whereby each panel is cut to size under factory conditions, CNC technology allows us to further customise the panels. This allows panels to be individually manufactured in order to perform in certain ways: to produce evocative visual and acoustic effects, function as maps of the building itself, or filter light through very thin layers of wood (Fig. 3).

In this way, the cut becomes a performative conceptual tool in its own right, using the new possibilities offered by mass customisation to create features that might connect back to different internal or external forces: one panel might bring more light into the living unit, while another adds an ornamental layer to the sheet.

Another important way in which mass customisation is driving the project is in the attention to material waste. While digital fabrication allows architects to explore more complex organisations and geometries, one objectionable by-product is a lack of attention to material waste. In a reversal of the too-frequent top-down scenario where typical material sheet sizes are plugged in to the system at the end of the design process, the CSH2.0 system starts from basic material dimensions and seeks to automatically minimise material waste during the CNC cutting.

7 ARCHITECTURAL ANALYSIS

The first building to come out of our CSH 2.0 scheme is a peculiar beauty. While the overall organisation is a direct translation of the cutting-and-stacking procedure, the detailing picks up elements from the original Case Study Houses scheme and rearranges them into a new and displaced composition. The expression is original, and can differ wildly between floors as some are cut through more often or with a wider conceptual blade than others.

The stacking of cuts is a particularly interesting architectural idea, as it presents an entirely new typology much in the same way that the Case Study Houses moved away from the prevalent LA style and promoted their own aesthetic, functional, and programmatic values. While structurally remaining a very simple system (essentially a volume balancing on top of another volume), the resulting spatial intersections and cut volumes give rise to new performative possibilities, including the layering of light across several storeys, the intermingling of programs and inhabitants where cuts intersect, the amalgamation of material effects along cut edges, the implied demarcation of functions, a flexible sectional configuration (including the possibility of effectively changing the order of the storeys as the build
progresses), and innovative circulation logics. As is the case with some of the most successful Case Study Houses, it is based on a simple principle that gives rise to a wide array of intricate spatial formations.

Using the Martinson’s solid wood building system means the CSH 2.0 buildings can be made using ready-to-assemble elements that are manufactured in a production environment with low humidity, guaranteeing both consistent dimensions and quality – an absolute must when working with as precise a system as the one outlined here. On site, Martinson’s assembly system includes a weather cover that protects the entire building and creates ultimate conditions for both dry building of the cut floors, and a pleasant working environment.

Mass customisation, finally, not only allows the scheme to become buildable and economically feasible, but also adds a seamlessness to the detailing, while providing a series of new opportunities for turning the building’s panels into performative screens. As with the prefabricated elements in the precedent schemes, this embrace of a novel building technology opens up new possibilities for perfect construction processes and economies of scale.

It is clear that rule-based systems such as the one presented here can be used to construct interesting multi-storey timber buildings. An updated, contemporary version of the original Case Study House program could fuse such a strategy with the virtues of mass customisation to yield a cost-effective and sustainable new building typology for the 21st century.

ACKNOWLEDGEMENTS
The authors express sincere appreciation to the Regional Council of Västerbotten, the County Administrative Board of Västerbotten and The European Union’s Structural Funds – The Regional Fund, for their financial support.

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FROM FILE TO FACTORY: ADVANCED MANUFACTURE OF ENGINEERED WOOD ELEMENTS - Innovative design solutions for multi-storey timber buildings throughout the entire building process

Magnus Larsson¹, Alex Kaiser², Ulf Arne Girhammar³

ABSTRACT: “File-to-factory” processes of computer technologies is a contemporary way to both maximise efficiency throughout the building process, increase a building's performance, and be able to add interesting architectural possibilities throughout the design phase. Viewing the building as a parametric network of connected components that can be individually controlled through unique parameters may no longer be a novel architectural concept, but its application to multi-storey timber buildings is still a territory for which there are no maps. Allowing not only the notion of identicality in mechanically reproduced objects to be left behind, but replacing the idea of the object with that of the objectile, the authors investigate a novel approach that produces a set of building trajectories rather than a set of buildings, yet yields a series of buildable examples of those trajectories. This paper describes and evaluates how this series of stacked multi-storey timber buildings based on three Swedish timber structural systems can be both incorporated within a file-to-factory process, and how this gives rise to a range of new and interesting potentials to create innovative solutions throughout the entire design and manufacturing process.


1 INTRODUCTION

Digital manufacturing per definition creates one-off objects. Serial variations tend to come at no additional cost. Digital processes tend to merge the two production modes of complex objects such as buildings: the initial phase of the design object as a collection of notational media representations are controlled using the same software that instructs the manufacturing machine to build the final physical object. While the digital design process brings contemporary architecture closer to its primeval mode (of simultaneous conceiving and making), the digitally controlled manufacturing process turns the factory into a computer-aided version of the artisanal workshop. Digital manufacturing makes it possible for standard building materials to become non-standard architecture. File-to-factory strategies can assist this operation. In this paper, we explore how such digital tools can be applied to a series of example buildings based on three Swedish timber structural systems, described in a previous paper.¹

2 DATA-TO-SITE FEEDBACK LOOPS

In a 2011 essay, Hani Buri and Yves Weinand discuss how different technological ages have influenced the tectonics of timber structures, from the artisanal “wooden age” through to our present “digital age”. Having established that the “ability to control machines with the help of a computer code eliminates the need for serial production,” the authors continue to explain that the easy machinability of wood “makes it an ideal material for digitally controlled processing portals,” which has led to timber “taking on the status of a high-tech material”.² Add to this digitally controlled processing of a high-tech material today’s possibilities for having different systems communicate with each other, and we end up with a highly flexible process where computer-aided manufacturing (CAM) and computer numerically controlled (CNC) technologies are linked to parametric design software and, possibly, even live-fed contextual data from the site in question. This would form a file-to-factory operation that would make it possible to manufacture individualised building components in a highly economical manner. The various parts would not only be adapted to fit perfectly together, forming a network of interlinked relationships, but would also acquire a new, time-based property: as it is analysed and tested before being inserted into the scheme, the

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component adapts to analytical data; once it has been installed, if fitted with a sensor, it can continuously feed data back into the system, reporting on its on status within the framework of the building. Such a data-to-site feedback loop would not only maximise efficiency throughout the building process, but also increase the building’s performance while adding interesting possibilities of influencing the architectural palette during the extended design phase over time. Eventually, architecture could literally be published into this information network.

While this Internet of Things-based vision is beyond the scope of this paper, an exact file-to-factory system is particularly appealing when it comes to engineered timber architecture, as accuracy is of the essence. The design of a building that is to be constructed from elements that will be pre-cut off site must be finished when it leaves the architect’s office. This suggests a production strategy in mass customisation (based on file-to-factory processes). New manufacturing methods give rise to new structures, new ways of using those structures, new architectures.

3 POST-SUSTAINABLE BUILDINGS

The growing urbanization of the planet, visible in the fact that by the middle of this century some 70% percent of the population will live in cities, situates the urban question as a key issue for the global environment. As architects and engineers, we need to engage with the questions arising from current analyses of a world in which about 3.4 billion people are concentrated on roughly one percent of the earth’s surface, consuming 75 percent of the world’s energy use and producing 80 percent of our total CO2, and predictions that in 2050 the world’s population will increase to 9.3 billion and the urban population will double to 6.8 billion. Our metropolitan areas is where density can allow us to live extraordinarily efficient lives; as David Owen observed in an article in The New Yorker, “Eighty-two per cent of Manhattan residents travel to work by public transit, by bicycle, or on foot.”

We hesitate to call this a “sustainable” urbanist outlook, as we are rather seeking a post-sustainable condition. Humanity needs to move beyond practices that merely do no more harm, we need to readjust our environmental strategy to focus on continuous improvement rather than mere sustainability, and the city is the obvious place to start, not the least because the metropolis is where change is most easily implemented. To quote Edward Glaeser, urban density “creates a constant flow of new information that comes from observing others’ successes and failures... Cities make it easier to watch and listen and learn. Because the essential characteristic of humanity is our ability to learn from each other, cities make us more human.”

In a provocative 2004 essay, David Schaller points out that “a typical coffee business uses 0.2 percent of the coffee bean to produce a cup of coffee. This means 99.8 percent of the coffee bush becomes ‘waste’”. According to Schaller, there is “an abundance to nature that, in our ignorance and even even arrogance, we are only beginning to fathom... Our microbiologists, botanists, biologists, mycologists, wood chemists and geneticists are only now scratching the surface of this great diversity and plenty. What we don’t understand, we can’t possibly explain, value, or protect.”

The choice of timber for a tall structure of this massive scale is a modest proposal that could perhaps open up one line of inquiry into this unharvested natural abundance. The earth contains about one trillion tonnes of wood, which grows at a rate of ten billion tonnes per year. An abundant, carbon-neutral or even carbon-negative renewable resource, timber is the only readily available building material that stores carbon dioxide, minimises or eliminates its carbon footprint, and hence is truly post sustainable.

Urbanising literally means “burying,” but we are not interested in burying the agricultural or suburban landscape. Rather, we want to celebrate the ground datum as a counterpoint to the necessary further densification of our urban nodes. By having the structure undulate across the topography, touching the ground in as few places as possible, we can potentially achieve a dichotomy between landscape and urbanism allowing the project to reverse today’s migration patterns, bringing the city to the people living in less densified areas. It defines the notion of sprawl, turning it into a progressive tactics for linking the city infrastructure to the rural outskirts of our urban conglomerations, extending the metropolitan condition.

The tall, sprawling, habitable timber network presented here is post sustainable in three major ways: through its reinterpretation of the urban condition and introduction of a new typology that can promote new ways of living, through its materiality as explained above, and through the systemic approach at its heart, which employs file-to-factory processes to not only minimise waste but lower the degree of processing while creating a structure that is easily manufactured, assembled, and maintained.

We also view it as being post parametric. A very brief commentary might be prudent. In 1967, Marshall McLuhan claimed that transformative new technologies eventually turn superseded technologies into “counter-environments”. The counter-environment of the Romantic landscape, for instance, is the railroad and factory of the industrial age. Counter-environments change the nature of perception, and by extension the opportunities for intervention. Parametric techniques in architecture – geometric models whose geometry is a function of a finite set of parameters – have prevailed since at least the Bauhaus, or Gaudi, or the Baroque, or the Gothic, depending on how we choose to frame the concept. While still toted as the next big thing in architecture, parametric design as a conceptual framework (there is no design without parameters) should by now have been brought into a counter-environment-like relief ready to be re-assessed. What we mean by calling this building post parametric is that it seeks to harness something completely different from the BIM models being celebrated by the corporate machine. Instead: an open-source system that acquires, processes,
and integrates data into a cloud environment that allows the building to “learn” from its context and inform the system that generates its next section about this knowledge in order to optimise its own performance; a signal network that could perhaps be likened to how the human peripheral nervous system connects the central nerve system to our limbs and organs.

4 SCALE, STRUCTURE, PRECEDENTS

The building presented here is massive. It’s scale goes beyond that of the building or even the urban block, towards the city itself. It is a megastructure, an enormous, self-supporting artificial construct. It is a plug-in gridshell, a huge canopy weather-sealed by living units, an oversized tribute to that great traditional British form, the Pavilion in the Park. The hybrid structure combines three Swedish timber building systems: a CLT/glulam lattice from Martinsons (regular along the x and y axes but locally irregular along the z axis) are supported by large-scale L-beams from the Moelven Trä8 system, which also double up as circulation and service spaces. Within this timber lattice – a wooden gridshell somewhat reminiscent of a scaled-up version of Frei Otto’s 1975 Multihalle Mannheim structure, Buro Happold’s 2006 Savill Building, or maybe Barkow Leibinger’s Company restaurant/auditorium in Ditzingen11 – prefabricated, pod-like living units manufactured using the Byggma Masonite Flexible Building System are inserted, effectively hung from the gridded framework, thus challenging the notion of compressive or subtractive stacking (Fig. 1).

It is impossible not to mention Jurgen Mayer H’s giant latticed timber canopy, a part of their redevelopment of the Plaza de la Encarnacion in Seville, Spain. This scheme, finalised and opened to the public in April 2011, includes an archaeological museum, a farmers market, an elevated plaza, an aerial walkway, as well as bars and restaurants, all contained beneath and within the parasol structure. Why is it so big? Mainly because it can be. Myron Goldsmith writes about the ultimate size for structures and the principles dealing with the effects of magnitude, going back to the teachings of Galileo and Sir D’Arcy Wentworth Thompson, concluding not only that every structure has a maximum and a minimum size, but also that size and scale “have important implications environmentally and functionally. Engineering for efficiency is not the last and only determinant; it is possible to make a choice from several efficient schemes because of architectural, aesthetic, and environmental reasons. The human needs must give the directions”.12 There are three other main reasons for the vast scale of this project. The first is that we are interested in investigating this new typology of upside-down stacking at the largest end of the spectrum, where the beams making up the lattice are large enough to hold circulation paths and other programmatic features, and principles of mass manufacturing and prefabrication can be challenged to exhaustion (for instance with entire living pods being manufactured under factory conditions). The second is a challenge to the prevalent Romantic view that environmental or even social concerns are somehow connected to scenic pleasures and aesthetic appreciations of the picturesque: “nature” is not passive but has always been an active agent in the co-production of our civilization while co-developing with it.13 By creating a building large enough to hold nature – in the form of tree/forest plantations – within itself, we comment on this misapprehension. The third is an extension of the renewable material’s properties: a building made from wood is a carbon dioxide store, and building with timber (for as many inhabitants as possible) can be viewed as a form of active climate protection.

At the scale of the city, this building embodies the ideal of the Japanese Metabolists – never a group to hold back in terms of scale: a technological fervour bringing about future cities made from large-scale, flexible, and expandable structures that evoked the processes of organic growth.14 That basic unit of Metabolism, iterated to perfection by Kisho Kurokawa, the prefabricated capsule, is important also to this scheme. Kurokawa’s 1960 Neo-Tokyo Plan and grid system on stilts, Agricultural City, as well as his capsule-within-space frame 1969 Odakyu Drive-in Restaurant, and of course his 1972 Nagakin Capsule Tower, are all somehow related to the present project, as is other Metabolist proposals such as Arata Isozaki’s 1960 Clusters in the Air and perhaps the 1961 elevated Disaster Prevention City by Kiyonori Kikutake.15

5 SOME ASSEMBLY REQUIRED

Prefabrication is often referred to as “architecture’s oldest new idea”16 – and for good reason. Not only has it been around for thousands of years, it has also been touted as the next big thing in architecture for about as long. When the newly-weds played by Buster Keaton and Sybil Seely have their prefab timber house delivered in the 1920 silent short film One Week,17 they are already at the closing end of the first chapter in the history of prefabrication, following a long narrative of prefab projects including the wooden panelised buildings the British used to house their fishing fleet in the 17th

Figure 1: The structural system. The black lattice is from Martinsons, the gray L-beams from Moelven, and the white living units from Byggma.
century; the Crystal Palace in 1851; the kit house of numbered, pre-cut pieces sold by Aladdin Readi-Cut Houses from the early 20th century; the wildly popular Houses by Mail program established by Sears Roebuck & Co in 1908; Swiss architect Le Corbusier’s structural frame Maison Dom-in-o in 1914, and the same architect’s short treatise Mass Production Houses, the penultimate chapter from his famous book Vers une architecture, which in 1919 introduced the idea of the house as “a machine for living”.  

Prefabrication fascinated the Bauhaus architects, and a year after Keaton’s comedy premiered, prefabrication entered a second chapter with the Baukasten (building blocks) developed between 1922 and 1923 by Walter Gropius and Adolf Meyer. These were standardised, industrially produced building elements that functioned as a variable kit of parts, interlocking to form a near-infinite array of configurations. The architects would guide the client through this “oversized set of toy building blocks,” illustrating possible configurations with a scale model, assembling different “machines for living” according to the inhabitants’ needs.

Timber is a fitting material for prefabrication. The first prefabricated construction system was the wood-based balloon frame method developed in Chicago by builder Augustine Taylor around 1833, and soon after the timber units making up the Manning Portable Colonial Cottage for Emigrants started shipping from England to Australia. Other interesting timber prefabs include Juhani Pallasmaa’s (with Kristian Gullichsen) 1969 model industrial summer house Moduli 225, the Burst 008 beach house by Jeremy Edmiston and Douglas Gauthier, Oskar Leo Kauffman and Albert Rüf’s 2008 System 3 unit, the Breckenridge Perfect Cottage mobile home by Christopher C Deam, and MH Cooperative’s tiny Summer Container.

The use of prefabricated, mass-customised living units reduces construction times and minimises waste, thus decreasing the building’s ecological footprint.

6 SYSTEMIC DESIGN

Just as the history of prefabrication traces the history of modernism in architecture, the history of digital iteration and the ability to respond to individual client needs through the linking of fabrication with computerised design is defining part of the current dialogue. Digital modes of production open up possibilities for designing with a much closer attention to innovative details, as we now know that (almost) whatever we create on our computer screens can be fabricated digitally at an affordable price. Design strategies, at the very intersection of architecture and engineering give rise to entirely new solutions within areas such as acoustics, ventilation, and lighting. The file-to-factory process also adds another layer of flexibility in that any grid element, structural support, or living capsule can (as long as this doesn’t compromise the structure) be added or subtracted at any time.

The act of measuring turns the territorial continuum of the site into an architectural element or system. This system is then carefully indexed and turned into a topographical network representation of the ground datum, which is raised above ground in a reversal of the typically predatory consumption of land by many urbanisation processes. The matrix canopy initiates a symbiotic relationship with the terrain below.

Additions to this flexible grid call for further analyses. The system becomes a continuously moving factory, treading the suburban and rural landscape to measure and survey it in preparation for the next stretch of the scheme. The diagram of the site topography that is the gridshell next becomes the basis for the computation of a set of operational rules, yielding the living capsules through mass customisation scripts, living units that are then “stacked” as they get hung within the structural matrix. The relationship between the local conditions of the grid and the various instantiations of living units is analogous to the way a single genotype might produce a differentiated population of phenotypes in response to diverse environmental conditions.

Despite its scale, the scheme is not as dense as a city; instead of density, we get intensity: the idea of individual buildings holding collective housing gives way for a collective building holding individual housing. The lattice frame itself becomes programmable, holding aerial walkways, terraces, transportation routes. Where a beam changes direction within the lattice, new opportunities for spatial diversity is inserted. While the basic form of the gridshell is based on the site topography, we now add additional design parameters. Any material construct can be considered as resulting from a system of internal and external pressures and constraints, its physical form being determined by these pressures. Depending on the circumstances, we can decide to exert climate-related forces, apply socioeconomic pressures, or deploy formal variations and deformations at strategic points.

The resulting combinations of lattice-supports-capsules iterate over time as the system keeps updating. Only once the building components have to be manufactured do we freeze the process and collate the data. Postponing the production until the very last minute guarantees that as much information as possible goes into the final, manufactured element and volume. Over time, this material system builds up a complex reciprocity between its materiality, site conditions, form, structure, and space, the related processes of mass customisation and prefabrication, as well as the resulting performative qualities of the structure itself.

7 THE FILE-TO-FACTORY PROCESS

The present scheme does not sit easily within a common typology. It is very likely that no structure of its kind has ever been built. Non-standard architectures call for non-standard ideas about how to use strategies such as mass customisation and prefabrication in the manufacturing process, though, crucially, it does not necessarily call for non-standard building materials or building processes. While its expression, program, manufacturing process, and design approach are all atypical, it is still a building
that can be made by everyday builders from perfectly normal engineered wood.

Recent technological advances enable the materialisation of architectural forms straight from a digital file. As the necessary equipment, machinery, infrastructure, and tools are becoming available to a larger part of the building industry, the manufacturing and fabrication of mass-customised elements according to specific instructions derived from modelling software is becoming increasingly popular. Data-driven fabrication is perhaps particularly appealing when it comes to architecture made from engineered timber, as accuracy is of the essence. Designing a building constructed from elements that are pre-cut, offsite, to the architect’s specifications means everything must be finalised at the drawing board. New manufacturing methods give rise to new buildings, new ways of using the buildings, new architectures. The plug-in gridshell structure has been designed to contain a minimal amount of timber elements: beams make up the lattice, cross-laminated sheets add structural support, surfaces (boards) come together to form the living capsules. Nearly all of these components, however, differ from each other, as they are all interlinked through a system that constantly updates our 3d model – a perfect representation of the building at any one moment in time – with new data. Since the building blocks are defined parametrically, they are able to be reprogrammed into essentially carrying out each other’s functions. The machines in the factory don’t know, and don’t care, whether the boards they are instructed to manufacture is to fill the function of a wall or a floor. As Kas Oosterhuis describes it, “Architecturally, convergence sees the beauty in the purest solution, the new modern: one building, one detail”.26

While our system doesn’t quite go so far as to define a single, parametrically controlled detail, it does take the data from the system and collates it into instructions for the factory, which in turn manufactures the parts in accordance with a predefined time schedule. This makes for an extraordinarily agile building process, where parts can be immediately printed if needed, and new parts pushed to a new place within the work schedule if for some reason the construction program changes. It also completely does away with the need for conventional, two-dimensional drawings. As the design that goes into the centralised three-dimensional project model is converted into instructions that are sent straight to the factory, nothing gets lost in translation. An annotation system makes assembly a matter of simply lining up one marked-up panel against the next, and if a builder needs to double check its placement, he or she can always bring up the latest version of the 3d model on a computer or tablet. The file-to-factory process makes Alberti’s orthographic projections and ideas of identity redundant: plans and sections become useless descriptions. This enhances efficiency and speeds up the building process. As Mark Burry points out, significant economic benefits can also be derived from “automating routines and coupling them with emerging digital fabrication technologies, as time is saved at the front-end and new file-to-factory protocols can be taken advantage of”.27

8 ARCHITECTURAL ANALYSIS

Living “off the grid” is a trendy way of referring to a self-sufficient lifestyle that doesn’t rely on public utilities (such as the main electricity transmission grid). The building presented here promotes the perhaps less trendy alternative of a communal lifestyle within the grid. Over and above the obvious environmental credentials – a carbon dioxide store that uses the relatively low density and very high degree of stability (greater, in relation to its mass, than that of steel) of wood to soar above the ground – the plug-in gridshell is a programmatically interesting alternative to rural or suburban lifestyles, which, as we have seen, are rarely as energy efficient as their urban counterparts. The systemic, adaptive variation and continuous differentiation arising from the dynamic design system, which anchors the project to the site as it continues to shoot through the landscape (in a way perhaps reminiscent of Superstudio’s 1969 The Continuous Monument77) offers an interesting contrast between input data and output result, the model/building responding to external pressures according to its own predefined logic. We have called the capsules living units, but they may also hold other programs: orchards, libraries, shops. In the shade of the structure, planted seedlings grow over time into a stretch of forest. Trees are cut down to create this habitable canopy for trees to grow under. The building also creates its own, artificial landscape (Fig. 2), a play between structure and spatial unit that becomes as varied as the information fed to the system. In some places, the spatial unit will take formal and programmatic priority over the timber lattice, while in others, the matrix will come to the foreground. But this isn’t the only way in which the system accommodates its local circumstances: it can also adapt to completely different contexts – urban, suburban, rural – and could just as well hug the skyscrapers of New York as climb across the Siberian tundra.

It is an example of how file-to-factory processes can be used to drive the production of a massive structure, but
also showcases the possibilities of hybridising different timber building systems. The three Swedish systems we are working with complement each other, supporting an architectural vision that they were clearly never meant to provide for.

9 CONCLUSIONS

File-to-factory manufacturing processes can support and give rise to innovative designs that cater for specific contexts. While wood is already regarded as an excellent material for digital manufacturing, few schemes seem to utilise this fact as a structural way forward, at different scales and in order to create a constantly varying and incredibly flexible building. The timber gridshell presented here is an attempt to start filling this gap. It is also, of course, a provocation. However it is important to remember, in the face of what might at first view come across as a highly radical proposal, that there is nothing to suggest the building presented here cannot be built. All of the technologies we have described already exist, as do the manufacturing processes, the structural possibilities, and the assembly strategies. It is not even that hard to imagine how the financing might be built. All of the technologies we have described above ground? How many floors could we add inside the capsulaes? How would the services work, the circulation systems? Would inhabitants be able to customise their own living units? What external pressures should the system respond to? Where is further research needed?

The only way forward is to corroborate those questions and the others outlined or implied in this paper with projects, prototypes, and eventually actual buildings.

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Prefabricated earthquake resistant timber structures for multi-storey buildings

Kamyar Tavoussi1, Johannes Melbinger2; Wolfgang Winter3

ABSTRACT: The demand for multi-storey buildings for business as well as for living has been globally increased. The development of a prefabricated timber structure for multi-storey buildings with big round continuous columns was on the focus in this research project. The conception of this structure was based on highly sophisticated methods in the past on Nias Island (in the North West of Sumatra, Indonesia). These structures could satisfy needs for post disaster environmental reconstruction also in developing countries.

KEYWORDS: Multi-storey buildings, prefabricated construction, earthquake

1 INTRODUCTION

Prefabricated timber structures will be an important objective in the future when time frame, transport capacities, availability of technical equipment and budgets are limited.

Timber structures are advantageous in case of earthquake. The low weight and the high ductility, if well designed, are the main benefits. [1, 2, 3].

The good performance of traditional Nias house structures in Indonesia (fig. 1) in case of sever earthquakes (up to 8.7 Richter) has been analysed in several publications, e.g. in “Adaptation and earthquake resistance of traditional Nias architecture“ [4].

This fact has inspired the authors to develop prefabricated timber structures for multi-storey buildings with big round continuous columns and high seismic resistance.

Figure 1: North Nias house in Sihare’ö Siwahili [4]

Due to a limited set of light elements with 6 to 7 m maximum length, production of elements and construction works are simple. Elements may be easily produced locally, generating jobs and business opportunities also in developing countries. Based on these requirements, following construction system was developed. First dynamic analyses showed a good result.

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2 PREFABRICATED BUILDING SET TIMBER STRUCTURES

For the first case study, a 3-storey building with a floor to storey floor height of 3 m was analysed (fig. 2-4).

The construction is supported by eight big columns, four of them have fixed supports and the other four have pinned supports at the base.

The quadratic floor plates have a size of 10 m by 10 m.

The floor plates are alternately twisted horizontally by 45 degree. 8 columns are positioned in a distance of 4.24 m from the centre point (fig. 2, 3, 4).

The most important technical characteristic of this construction is the big columns which are alternately going through two storeys.

Two types of floor-column joints are designed. The first type is a hinged joint (fig. 4, 5).

In case of the second type, the column is embedded between the stiff floor plates in the middle. Some kind of fixing is produced by using wedges (fig. 4, 6).

Figure 2: 3-D Perspective of the analysed multi-storey structure

Figure 3: Plan of the analysed multi-storey structure

Figure 4: View of the analysed multi-storey structure

Figure 5: Detail 1: Concept of hinged connection of 2 columns

Figure 6: Detail 2: Concept of continuous column fixed to the roof plate by wedges
This kind of fixture will increase the stiffness of the whole structure. In case of earthquake, the plastic deformation of these wedges will increase the damping capacity. In the past, some good experiences have been collected with such connections at the Institute of Structural Design and Timber Engineering (ITI), Vienna University of Technology, see figure 7 [1, 2].

![Figure 7: beam-column connection fixed by wedges (left), plastic deformation of wedges after quasi-static tests (right)](image)

No vertical loads should be transmitted from the floor plates to the fixed columns. On every level, the floor plates should be vertically supported only by 4 columns, see figures 2 and 9. The floor plates should be assembled as grid plates by using cross laminated timber plates (CLT) and timber beams. The shear rigidity of such a plate should be granted.

Many details have to be developed, e.g. the production details of the stiff floor plates and the connection between the column and the floor plate.

The calculations were carried out for pure timber columns. If necessary, timber columns can be reinforced by flat steel sheets. Concerning this subject, some research activities are carried out at this time at ITI [5, 6].

The first vibration mode of the structure is shown in figure 8. The natural period $T = 2 \text{ s}$.

![Figure 8: first vibration mode of the structure](image)

There have been some dynamic calculations carried out for Vienna according to EC 8 [7].

The assumed input data’s for calculations are:

- constant damping value of 5%
- type 1 spectrum (relevant for Austria)
- design ground acceleration “$a_g$” of 0.87 m/s²
- behaviour factor “$q$” of 1
- live load was assumed as 3 kN/m²
- the dead load of the floor covering was assumed as 1 kN/m²
- the diameters of the round columns were chosen 40 cm
- Timber grade is C 24 according to EC 5 [8].

For the ultimate limit state (ULS) a maximum support action of 156 kN is expected, the maximum bending moment would be equal to 81 kNm, see figure 9.

A realistic behaviour factor of 2-3 will reduce especially the calculated values for bending moment.

![Figure 9: maximum support action and maximum bending moments for ULS for behaviour factor $q=1$](image)

By designing all joints as rigid joints, a higher stiffness of the whole structure could be achieved. That would reduce the deformations in case of wind loading. In return, the higher stiffness would cause higher seismic loads because of lower natural period.
3 CONCLUSION

Light, prefabricated timber structures are advantageous for multi-storey buildings in urban areas regarding transport and erection time. High loads can be transmitted with simple connections which accelerate construction time. The total weight remains very low which is advantageous in case of earthquake. These structures could satisfy needs for post disaster environmental reconstruction also in developing countries. In the future, more details ought to be developed.

REFERENCES

STRUCTURAL PERFORMANCE OF WOOD-CONCRETE COMPOSITE BEAM FOR APPLICATION TO BRIDGE SUPERSTRUCTURE

Sang-Joon Lee¹, Chang-Deuk Eom², Kwang-Mo Kim³, Joo-Saeng Park⁴

ABSTRACT: There have been many researches that deal with the wood-concrete composite system especially for the bridge superstructure. Many of the researches have been conducted to find out or improve the shear performance of the wood-concrete composites. This study was performed as the basic study of the wood-concrete composite system. The yield mode and the reference design value (Z) were derived with different anchoring length of the simple steel rebar based on the EYM (European Yield Model). Simple shear test of wood-to-wood composite was designed and performed for verification. Results show that the yield load shows over 40% of predicted Z value which would be occurred by the differences in yield strength of tested steel rebar between the actual and designed value. Further studies including the pull-out effect of the rebar and actual yield mode investigation are now in progress.

KEYWORDS: Wood-Concrete Composite, Bridge, Structural Performance, EYM (European Yield Model)

1 INTRODUCTION

It is well-known that the wood-concrete composite system can be used in the civil construction such as floors and bridges [5]. For the bridge superstructure, the main idea is using a wooden beam and a concrete slab. Two materials have different physical properties, and normally the idea that wooden part acts as the tension and concrete part acts as the compression seems to have advantages. And the durability of whole bridge can be raised up with using the concrete slab. The structural performance with composite action including the shear performance of two materials has been normally the main key point of recent researches [1, 2, 4]. Above-mentioned criterion is not a new idea. Specially, related researches started at 1930’s in the USA and recently, many wood-concrete composite bridges have been built in European countries [3]. The main researches have been dealt with the hybrid action because the natural bond between wood and concrete is not high enough. Many approaches have been done with using variety types of notches, steel rods, shear keys and gluing. However, there were little approaches theoretically and practically related with the wood-concrete composite system in Korea. There was no wooden bridge itself which can be used for the vehicles. In this paper, basic research which related to the structural performance of composite action of wood and concrete was performed for application to bridge superstructures.

2 MATERIALS AND METHODS

2.1 MATERIALS

2.1.1 Wood

Korean Pine which has 0.43g/cm³ of specific gravity and controlled to be air dried condition was used for manufacturing wood to wood composites. Wood specimens were manufactured to be 120mm (thickness), 180mm (width) and 200mm (height). And the slope for performing the shear test of composite was determined to be 18.4° (Fig. 1). Totally, eight wood-to-wood composites were used for this research.

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2.1.2 Steel Rebar
The type of the steel rebar was SD30A which is accepted by Korean Standard (KSD 3504) and the diameter of the bar was 10mm (D10). The yield load of the SD30A steel rebar was designed to have over 294MPa. With considering the nominal area of D10 bar (71.3mm²) the yield strength can be calculated to have over 21.0kN.

2.1.3 Manufacture of the Wood Composite
11mm drilling holes with eight steps of penetration length (from 20mm to 160mm with 20mm increment) were made for prepared wood specimen (acted as the side member) at center of the thickness and 50mm in length from the top face of the member (Fig. 2). The penetration length of the main member is fixed to 180mm which is the width of wooden specimen. Then the steel rebar was anchored with using the chemical anchor (FIS V 360S, Germany).

2.2 MODELING OF WOOD COMPOSITE WITH EYM
Theoretical approaches with mainly based on the EYM (European Yield Model) was performed for predicting the performance of wood composites.

3 RESULTS AND DISCUSSIONS
3.1 SHEAR PERFORMANCE OF STEEL REBAR
Double shear test was performed for SD30A D10 steel rebar before testing the wood composites. The yield load was derived from the load displacement curve (Fig. 6).

Test result showed that the yield strength of tested steel rebar was 29.8kN. This is over 40% value of expected one (21.0kN) which is regulated from the standard (KSD 3504). And the yield strength was over five times of the reference design value (Z) (5,565N) (Table 1).
3.2 MODELING RESULTS OF THE WOOD COMPOSITES WITH EYM

Table 1 shows the results of EYM with different anchoring length of wood composites. The anchoring length indicates the penetration length of steel rebar at the side member. The minimum computed yield mode value was determined for the reference design value.

Table 1: Yield mode and reference design value (Z) due to different anchoring length of wood-to-wood composites

<table>
<thead>
<tr>
<th>Anchoring Length (mm)</th>
<th>Initial Stiffness (kN/mm)</th>
<th>Maximum Load (N)</th>
<th>Maximum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.53</td>
<td>8,685</td>
<td>16.3</td>
</tr>
<tr>
<td>40</td>
<td>1.43</td>
<td>10,675</td>
<td>27.3</td>
</tr>
<tr>
<td>60</td>
<td>1.87</td>
<td>14,669</td>
<td>35.9</td>
</tr>
<tr>
<td>80</td>
<td>1.49</td>
<td>14,990</td>
<td>36.9</td>
</tr>
<tr>
<td>100</td>
<td>1.78</td>
<td>18,106</td>
<td>50.9</td>
</tr>
<tr>
<td>120</td>
<td>1.56</td>
<td>15,280</td>
<td>41.2</td>
</tr>
<tr>
<td>140</td>
<td>1.51</td>
<td>16,180</td>
<td>36.9</td>
</tr>
<tr>
<td>160</td>
<td>1.9</td>
<td>18,059</td>
<td>45.3</td>
</tr>
</tbody>
</table>

Results shows the yield at 5mm, 10mm and 15mm of anchoring length show Is mode while the yield mode over 20mm show IV mode. Derived reference design value (Z) shows around 1.6kN, 3.2kN, 4.7kN and 5.6kN for anchoring length of 5mm, 10mm, 15mm and 20mm respectively, and the yield mode and Z value shows same values over 20mm. From this result, it would be expected that the effective anchoring length for tested setup of the wood-to-wood composite is around from 15mm to 20mm.

3.3 SHEAR PERFORMANCE OF WOOD COMPOSITE

Fig. 7 and Table 2 show the results of single shear test of wood composites with different anchoring length. The initial stiffness shows 1.63±0.19kN/mm and the yield load shows around from 7kN to 8kN. And there are little differences with varying anchoring length.
REFERENCES


Development of seismograph with struck level judgment function intended for timber structure

Hiromitsu Kajikawa¹, Yuka Okada², Atsushi Osawa³, Mikihiro Uematsu⁴
Osamu Tsuruta⁵, Hiroyuki Noguchi⁶

ABSTRACT: The development of the seismograph with the struck level judgment function for the timbered house is shown with this thesis. We want to make this device cheaply, and to spread to the Japanese whole country. It is possible to contribute to the disaster correspondence greatly when this device spreads to the Japanese whole country. In addition, the seismic design of the timbered house including the subsurface layer can be improved more.

KEYWORDS: Timber house, Struck level judgment function, Earthquake damage, Seismograph

1 INTRODUCTION

If the input earthquake motion that the timbered house receives cannot be understood accurately, it is difficult to judge the struck level accurately. Especially, it is still difficult to judge it without removing exterior and interior. In addition, the subsurface layer greatly influences the input earthquake motion, and the input earthquake motion is greatly different each place. Then, we tried the development of the device that judged the struck level. The device has the seismograph. Moreover, the device does the elastoplasticity response analysis by using the data measured from the seismograph, and judges the struck level from the analytical result.

This device can be set up all of the house that we build regardless of building or the reform. And, it is thought that this device is greatly useful for various respects like disaster prevention and countermeasures against natural disaster. If the input earthquake motion that the timbered house receives cannot be understood accurately, it is difficult to judge the struck level accurately.

Table 1: JMA Intensity and Max.Acc

<table>
<thead>
<tr>
<th>No.</th>
<th>Date</th>
<th>Time</th>
<th>Misawa Seismograph(A-city)</th>
<th>Meteorological Agency</th>
<th>K-NET(K-net)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max.Acc (X)</td>
<td>JMA Intensity</td>
<td>Max.Acc (Y)</td>
</tr>
<tr>
<td>1</td>
<td>2012/3/18</td>
<td>22:26</td>
<td>1.54</td>
<td>1</td>
<td>0.31</td>
</tr>
<tr>
<td>2</td>
<td>2012/3/19</td>
<td>18:30</td>
<td>0.78</td>
<td>1</td>
<td>0.31</td>
</tr>
<tr>
<td>3</td>
<td>2012/3/20</td>
<td>10:15</td>
<td>0.34</td>
<td>1</td>
<td>0.12</td>
</tr>
<tr>
<td>4</td>
<td>2012/3/21</td>
<td>19:45</td>
<td>0.17</td>
<td>1</td>
<td>0.06</td>
</tr>
</tbody>
</table>

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calamities, etc. Moreover, it is one of the targets that this device is widely spread. Therefore, cheapness is also important.

2 SITUATION OF EARTHQUAKE DAMAGE IN TIMBER HOUSE

Shown for earthquake damage of wooden houses in here. The earthquake damage shown here is due to earthquake off the coast of northeastern Pacific Ocean that occurred March 11, 2011.

This earthquake was very big damage caused by the tsunami (Figure 2, Figure 3). Housing remain to shed little like Figure 7. As an example of mild damage of wooden houses, there is a crack in the outer wall (Figure 2), wrinkles of the wallpaper. Some, such as damage to the roof of the tiles fall as shown in Figure 3. There may be a crack in the foundation and some damage. In addition, there are many of damage the liquefaction and the ground as shown in Figure 5 and Figure 6. Damage of the ground has a large influence on the building, sufficient care must be taken.

Figure2: The wallpaper is wrinkled.

Figure3: The roof is damaged.

Figure4: The base shifts. (The base is lacked.)

Figure5: Damage of the ground (The cliff collapses)

Figure6: Damage of the ground (Liquefaction is generated.)

Figure7: The tsunami is generated.
3 STRUCK LEVEL JUDGMENT IN TIMBER HOUSE

Earthquake damage survey and diagnosis for buildings shall be done after the occurrence of earthquakes. Some researches were engaged by evaluating relationship between the finish material and the tilt angle of the buildings [4, 5], which can provide effective information of finish material of wooden house subjected to the earthquake. While the structural assessments are actually more helpful to evaluate the safety and reliability of buildings, and the structural investigations of the buildings are much more difficult than that of finish materials. A study to identify the damage relationship between finish materials and structural part for seismic evaluation is put forward. Table 5(a) and (b) give an analysis result of the collapse process of structure and finish material. Photograph and comment record are reported, respectively. According to the analysis result, damage behaviour of interior finishing wall material appeared earlier than that of the exterior wall finishing material, and the structural damage behaviour happened ultimately. Damages are linked in accordance with the drift angle controlled in the loading test. Based on the investigation of this study, damages of finishing material reflect those of structural component in a certain extent.

Table 2: Collapse process analysis (a) Photograph record

<table>
<thead>
<tr>
<th>Drift Angle</th>
<th>1/600 ~ 1/450</th>
<th>1/300</th>
<th>1/200</th>
<th>1/150</th>
<th>1/100</th>
<th>1/75</th>
<th>1/60</th>
<th>1/45</th>
<th>1/30</th>
<th>1/20</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metal Joint</td>
<td>No damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gap of corner joint</td>
</tr>
<tr>
<td>Nail Joint</td>
<td>No damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gap of corner joint</td>
</tr>
<tr>
<td>Structure Board</td>
<td>No damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Relative sliding between boards</td>
</tr>
</tbody>
</table>

(b) Comment record

<table>
<thead>
<tr>
<th>Drift Angle</th>
<th>1/600 ~ 1/450</th>
<th>1/300</th>
<th>1/200</th>
<th>1/150</th>
<th>1/100</th>
<th>1/75</th>
<th>1/60</th>
<th>1/45</th>
<th>1/30</th>
<th>1/20</th>
<th>Overall judgement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metal Joint</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>Nail Joint</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>Structure Board</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>Exterior</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>General evaluation</td>
<td>Little damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Little damage</td>
</tr>
<tr>
<td>Cement</td>
<td>Slits in openings</td>
<td></td>
<td>Extension of slits in openings</td>
<td></td>
<td>Extension of slits of other parts</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Extension of slits of other parts</td>
</tr>
<tr>
<td>Siding board (Horizontal line)</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>General evaluation</td>
<td>Little damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Little damage</td>
</tr>
<tr>
<td>Wallpaper in opening</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Negligible damage</td>
</tr>
<tr>
<td>Sash/Crescent lock</td>
<td>Negligible damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>May not open and close</td>
<td>Negligible damage</td>
</tr>
</tbody>
</table>

Overall evaluation Negligible damage Little damage Medium damage Serious damage

Possibility of collapse

Damage of diagonal bracing (Partially) Damage of diagonal bracing (Widely)

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

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Gap of the corner joint

Gap extension Damage of corner joint

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Gap extension Damage of corner joint

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Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint

Gap of the corner joint

Gap extension Damage of corner joint
4 OUTLINE OF SEISMOGRAPH WITH STRUCK LEVEL JUDGMENT FUNCTION

Figure 8 shows an overview of the seismograph that we have developed.

4.1 Device (hardware)

The measuring unit is mounted on the rising edge of the footing portion. The display unit is mounted on the interior. And the measuring unit and the display unit are connected by the cable wiring in the wall that has the capabilities of power and communication. The measuring section is equipped with the foundation, the CPU for performing the operation and the acceleration sensor. The display unit shows the degree affected the judgment result and the maximum acceleration and the seismic intensity and the date and time after the earthquake. Also, when the acceleration sensor in response to shaking, there is also a function to inform the earthquake at the buzzer. Mechanism of seismograph is a flow such as flow diagram shown in Figure 9.

4.2 Program (software)

Here, we describe a method of determining degree of disaster analysis. Analysis method is the analysis of elasto-plastic response. The model is the series model. The hysteresis characteristics is the slip-bilinear model, and the slip ratio is set to 0.8, which represents the wooden building. Building information is collected for each building. We will put the building information to each seismograph. By doing so, it is possible to calculate the result of the determination time per building affected.

5 CONCLUSIONS

It is thought that not only countermeasures against natural calamities but also the interests concerning disaster prevention and the seismic design grow if the seismograph with this struck level judgment function is achieved.

REFERENCES


MEASURED LONG DURATION STRENGTH OF TIMBER COLUMNS

Robert H. Leicester¹ and Ron G. Pearson²

ABSTRACT: This paper presents an initial analysis of the measured long duration strength of 220 small clear-wood timber columns. This is probably the first such study to be done. Both hardwood and a softwood species were used, and the moisture conditions of the columns tested were dry, green and green allowed to dry. The tests were undertaken over a period of approximately 15 years. The test data was processed to assess a theory of creep buckling that has been used for the derivation of design procedures given in the Australian timber engineering design Code. Although columns made from clear wood have some fundamental structural differences to columns made with full size structural lumber, the study is useful for assessing the effectiveness of the long duration buckling theory used, and to develop proposals for future measurements of parameters relevant to predicting long duration buckling strength.

KEYWORDS: Structures, timber, columns, strength, load duration, creep

1 INTRODUCTION

The design of timber columns to carry long term loads has been a feature of timber engineering design codes for many years, but to date there does not appear to have been any experimental data to assess the effectiveness of the design procedures used. Accordingly the authors decided to access some relevant laboratory data measured about 60 years ago by one of the authors (Pearson), and to use this data to assess current creep-buckling theories on which the Australian Standard AS1720.1 [1] is based.

Two related sets of data on column strengths were available. One set is published data on short term load tests of clear wood columns by Pearson [2]. The second is unpublished data on the long duration strength performance of similar timber columns, also undertaken by Pearson; unfortunately data for only half of the original tests were recovered. The following is based on a preliminary processing of this data.

2 LABORATORY TESTS

The columns tested were clear wood, with a depth d=39 mm and a width b=51 mm. Fitted metal caps were used to provide an effective pin-ended load with a specified eccentricity e as illustrated in Figure 1.

![Figure 1: Notation used for wood columns](image)

The lengths of the columns chosen for this study were those having slenderness ratio values L/D of 15, 20, 30 and 50. The eccentricities used were 0, L/360 and L/120. During the long duration column tests several parameters were continuously recorded, including the lateral deflection at column mid-height. For these tests, the time to failure was noted if this occurred before 7 years. Some columns were unloaded prior to failure because it looked as though all deformations had ceased; for these columns

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² Ron Pearson, Professor Emeritus, North Carolina State University, USA. Email: rgrayp@gmail.com
it was assumed that their time to failure would be in excess of 10 years. For both the short duration and long duration tests, there were typically 2-3 replications of each specimen.

For the short duration tests the species used were three Australian hardwoods. These were Karri (Eucalyptus Diversicolor) and Silver Quandong (Eleocarpus Grandis) tested dry, and Mountain Ash (Eucalyptus Regans) tested green. Data from about 110 columns was used.

For the long duration tests, the species used were the softwood Douglas fir (Pseudotsuga taxifolia) and an Australian hardwood, Yellow Stringybark (Eucalyptus Muelleriana). The moisture conditions of the column during the tests were dry, green and green allowed to dry. The green columns were sheathed in plastic bags and subjected to a water spray about once a week. About 210 columns were tested.

3 CLEAR WOOD PROPERTIES

For all columns tested, the compression strength $F_c$ and modulus of elasticity $E$, were measured through tests on specimens cut from matched pieces of timber. For other properties required in the theory of column strength, estimates were obtained from typical values for clear wood cited in the published literature [3,4]. Some of the chosen parameters are given in the following.

The short term bending strength $F_b$ was taken to given by

$$F_b = F_c/0.55 \quad (1)$$

The increment of lateral stiffness $EI$ during drying was be taken to be a factor of 0.1 for all species. The corresponding increase in axial crushing strength $F_cA$ was be taken to be 0.8 for Douglas Fir and 0.4 for Yellow stringybark respectively. The increase in lateral bending strength $F_bZ$ was taken to be 0.5 for Douglas fir and 0.3 for Yellow Stringybark respectively. These increments were intended to take into account the effects of drying on shrinkage, stiffness and strength.

4 THEORY OF CREEP BUCKLING

The basis of the column buckling equations in the Australian Standard AS1720.1 [1] is derived as follows. First it is assumed that the shape of both the unloaded and loaded pin-ended column is that of a sine curve; the deformation at the mid-height of the column, denoted by $\Delta$, has an initial value $\Delta_o$ for the unloaded column. The deformation of the column under a constant load is then given by [5]

$$\Delta = \Delta_o \left(1 + \alpha \right) e^{\alpha \xi}$$

$$+ \Delta_b \left(1 + \alpha \right) \left\{1 + \left(1 + \frac{1}{\alpha} \right) \left(e^{\alpha \xi} - 1 \right) \right\} \quad (2)$$

where $\Delta_b$ denotes the initial lateral deflection at mid-height due to the eccentricity of a load and $\xi(t)$ denotes a relative creep factor. This creep factor can be obtained through measurement of the lateral deformation of a beam under constant load; the creep factor is defined herein as the ratio of the creep deflection relative to the initial deflection of the beam.

The amplification parameter $\alpha$ is computed from

$$\alpha = \left( \frac{1}{P_{bL} - 1} \right) \quad (3)$$

where $P$ is the applied load, $P_{bL} = \pi^2E/L^2$ denotes the Euler buckling load of a pin-ended column and $I = bd^4/12$ is the cross-section moment of inertia. The initial deflection due to the axial load acting with eccentricity $e$ is

$$\Delta_b = \frac{PeL^2}{8EI} \quad (4)$$

The equation used to compute the load capacity of pin-ended columns under both short and long term loads is

$$\frac{P}{AF_c} + \frac{P \Delta}{ZF_b} \leq \beta(t) \quad (5)$$

where $P$ is the applied axial load, $A = bd$ and $Z = bd^2/6$ are the area and section modulus, $F_c$ and $F_b$ are the short term compression and bending strengths and $\beta(t)$ is a wood parameter that is intended to take into account any changes that may occur in the strength during the duration of a test. For the case of dry timber and timber kept green, a parameter similar to the traditional load-duration factor for clear wood [3] is probably appropriate. For this study, the function chosen was

$$\beta(t) = 1 - 0.067 \log_{10}(t) \quad (6)$$

For columns that are initially green, and then allowed to dry out during the test period, some account needs to be taken of the reduction in cross-section and the increase
in the material strength that occurs during drying. For this case, the parameter chosen for the material parameter was

$$\beta(t) = \{1 - 0.067\log_{10}(t)\} \{1 + 0.5[1 - \exp(-0.005T)]\}$$  

Equation (7)

A plot of equations (6) and (7) are given in Figure 2. It should be noted that equation (7) gives a value of $\beta$ greater than 1.0 as the timber dries.

Equations (2)–(7) form a set of nonlinear equations, and care must be taken in obtaining solutions thereto.

**Figure 2: Plot of assumed functions for the material parameter $\beta(t)$**

### 5 EFFECTIVE CROOKEDNESS

The data for the short duration tests reported in [2] were used to assess the effective crookedness parameter $\Delta_o$ of clear wood columns. Columns with intermediate slenderness, with L/D values of 15, 20 and 30 were selected, as these are most sensitive to crookedness parameters. Application of equations (2)-(5) with the parameters $\xi=0$ and $\beta=0$ leads to the results shown in Figure 3.

It is seen that there is a considerable scatter in the values of delta-zero ($\Delta_o$) obtained and that there is no relationship of $\Delta_o$ to the length of the column, as is usually the case with full size structural lumber. This is probably because the laboratory specimens were carefully machined, so that the geometrical imperfections were negligible. Accordingly the initial "crookedness" of the columns were probably due to non-homogeneity of the wood and in general these may not follow a true sine form. The average value of $\Delta_o$ was found to be 2.8 for the green timber and 3.3 for the dry timber.

**Figure 3: Measured crookedness delta-zero ($\Delta_o$)**

### 6 MEASURED LONG DURATION STRENGTH

The relative strength capacity of a column under long term loads is defined as $P/P_{ult}$ where $P$ is the applied load, and $P_{ult}$ is the short term strength of the column. The strength of the columns under short term loads was computed theoretically using equations (4)-(7), and taking a value of $\Delta_o = 3.0$ mm.

In processing the data, the times to failure measured from columns with all eccentricities were grouped together into one set; with this arrangement there were about 6 replicates for each group type. If there were 4 or more failures within a chosen group, then an estimate of the median strength for the group was made using a lognormal distribution assumption for the distribution of failure times.

Figure 4 shows the data obtained for the green timber columns. The difference between the test data and the clear wood strength line is an indication of the durability effect arising from creep buckling effects.

**Figure 4: Measured median values of time to failure for columns of green timber**
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7 MEASURED CREEP FACTORS

A rough assessment of creep factors was obtained by using the lateral deformations recorded during the long duration column tests. For this purpose, data from the very slender columns, having a slenderness ratio L/d=50, were used; these columns exhibit the largest deflections and are more likely to deflect in a sine shape. About 50 columns were available for this purpose.

The deformation measured during the test will be denoted by \( \Delta_p \), ie \( \Delta_p = \Delta(t) - \Delta(0) \), where \( \Delta(0) \) is the initial deflection of the loaded column. Then from equation (2) we can derive the creep factor \( \xi(t) \) from

\[
\xi(t) = \frac{1}{\alpha} \ln \left( \frac{\Delta_p(t) + B}{B} \right)
\]

(8)

where the parameter B is a constant given by

\[
B = \Delta_e \left( 1 + \alpha \right) + \Delta_b \left( 1 + \alpha \right) \left( 1 + \frac{1}{\alpha} \right)
\]

(9)

In addition, if the column is initially green and then allowed to dry, the bending stiffness EI must be increased by a small factor to take into account the changes in the modulus of elasticity and shrinkage. For this study, the effective stiffness \( EI_{\text{eff}} \) at time t was taken to be given by

\[
EI_{\text{eff}} = EI \{1 + 0.1[1-\exp(-0.005 t)]\}
\]

(10)

An example of a typical creep record is shown in Figure 5(a) and the corresponding plot of a creep factor is given in Fig 5(b).

To simplify the processing of the large quantity of creep data obtained, only the data obtained at 1, 10, 100 and 1000 days were tabulated. In addition, the creep data from all wood columns that did not show significant deformation at 100 days, have been culled.

Figure 6 is an example of the plots that arise when using this limited data. The fitted line is taken near the high end of the graphs, as it is likely that these may correspond to specimens for which the crookedness may approach a sine form.

An alternative presentation of the data is given in Figure 7; it is a mean value of all the creep data obtained. It is to be noted that the fitted curves in Figures 6 and 7 are based on different criteria and hence, as may be expected, they do not agree.
The measured creep factors, such as those shown in Figures 6 and 7, exhibit considerable scatter in values. It would be interesting to investigate further to find out if this reflects the real variability of creep, or if it is another artefact due to the incorrect assumption of an initial sine wave shape. Either way, the information given in Figures 6 and 7 does provide some idea of the magnitude of the creep factors that need to be considered in developing design rules.

8 PREDICTED TIME TO FAILURE

It is of interest to assess the ability of the theory presented to predict the time to failure of the experimental columns. In doing so it was found that a better prediction was obtained when the input creep was taken to be 50% greater than had been assessed through the previous analysis. For the case of green or dry timber, the creep function used has been assumed to be given by

$$\xi(t) = 0.6 \log_{10}(t)$$  \hspace{1cm} (10)

An example of predictions of time-to-failure is shown in Figure 8. The large uncertainty associated with these predictions is perhaps to be expected in view of the difficulties associated with attempting to use a theory developed for structural lumber to predict the performance of clear wood columns. There may also be uncertainties introduced due to experimental difficulties. For example, some difficulties were experienced in ensuring that the green columns remained completely green throughout the duration of the tests.

9 SENSITIVITY STUDIES

In the preceding, there has been considerable discussion on the difficulties inherent in the using clear wood columns to assess the effectiveness of a structural theory developed for full-size structural lumber. However, the exercise does provide some idea of the magnitude of the parameters involved. Once this has been done, computer simulations of column strength may be undertaken to assess the sensitivity of the theory to various parameters. For example, the sensitivity to changes in the creep factor function can be seen from Figure 9. This is obviously an important factor and indicates that the measurement of creep factors should be investigated in some detail; this can be done quite easily by taking long term measurements of laterally loaded beams [6]. On the other hand it was found that the time to failure was not sensitive to variations in the initial crookedness $\Delta_o$ and the eccentricity $e$, both of which are difficult to evaluate in practice.
10 CONCLUDING DISCUSSION

The availability of long duration measurements of column strength, has provided an unique and valuable opportunity to assess the structural theory of timber columns that is used in the Australian Standard AS1720.1 [1]. It was found that, in general, the concepts of the creep buckling theory were effective.

A primary difficulty found in this exercise was the fact that the theory for structural lumber assumes a neat sine wave form for the initial column shape, whereas the wood columns used in the research were accurately machined from clear wood. Thus the effective crookedness of the model column behaviours was probably due primarily to material non-homogeneity, a matter that involves somewhat different implications to geometrical crookedness.

Nevertheless the study has provided some useful insights related to column crookedness, creep and time to failure. It can also be usefully employed in providing an idea of the magnitude of the parameters involved, information that can be used to design studies of parameter sensitivity. Information on parameter sensitivity can then be used to indicate useful areas of future research to investigate parameters that will be useful for application in formulating design rules. It can also be used to provide some idea of the uncertainties associated with the use of current creep buckling formulae.

REFERENCES

A MULTI-DISCIPLINARY CANADIAN RESEARCH NETWORK TO SUPPORT USE OF TIMBER IN MULTI-STOREY AND NON-RESIDENTIAL CONSTRUCTION

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ABSTRACT: A five-year multi-disciplinary research network was established under the Natural Sciences and Engineering Research Council’s (NSERC) Forest Sector R&D Initiative in 2010 in Canada to support the development of innovative construction technologies and advanced design methodologies, and to review some of the building code barriers, with the ultimate goal of increasing the use of structural wood products in mid-rise and non-residential construction. The network, referred to as NEWBuildS, consists of researchers from 11 Canadian universities, FPInnovations, National Research Council and the Canadian Wood Council. Because a building is required to be designed to meet the requirements of a number of performance attributes, including structural, fire, serviceability, acoustic and durability, a multi-disciplinary team of researchers with these backgrounds have been assembled to conduct collaborative research projects in this Network. This will ensure that any proposed solutions will meet the key objectives of the National Building Code of Canada. This paper provides an overview of the structure of the research network and its research program, the logics behind the various research clusters, expected key results and outreach structure.

KEYWORDS: Research network, hybrid building system, mid-rise light wood frame building, cross laminated timber, building code

1 INTRODUCTION

Timber is one of the most common and oldest structural materials. It was used to build tall structures centuries ago in Europe and Asia. Some of these structures are still standing today. Nowadays, the use of timber is limited mostly in low-rise (four storeys or lower) residential buildings in Canada mainly because of restrictions imposed by the building codes developed during the 20th century. Increased emphasis on sustainable building designs and the emergence of engineered wood products and systems have led to renewed interest in using construction with wood products beyond the current low-rise residential market. Coincide with renewed interest in timber was the timely transformation of the building codes to a performance-based format in a number of countries. Under the new performance-based format, designers are offered the opportunity to apply advanced design methods and construction technologies to demonstrate that non-traditional building solutions can meet and exceed the performance requirements of the building codes. In Canada, the alternative solution provisions in the objective-based building codes facilitated the construction of a 6-storey wood building in 2009. Earlier, a similar approach has led to the design and construction of a 9-storey building constructed entirely of cross laminated timber (CLT) in the top 8 storeys in London, UK. Construction companies and building designers are exploring various design concepts of buildings taller than 9 storeys with timber alone or in hybridized systems consisting of timber in conjunction with other materials. If these concepts are to be realized in practice, continued research to develop new technologies, including innovative products, systems, design tools and construction techniques, and to remove unjustified building code barriers, is required.

2 RESEARCH NETWORK

In Canada, under the Natural Sciences and Engineering Research Council’s (NSERC) Forest Sector R&D Initiative, a five-year multi-disciplinary strategic
research network, known as NEWBuildS, was established in 2010 to support the development of such technologies and advanced design methodologies, and the review of some of the key building code barriers. The vision of the Network is to increase the use of wood products in mid-rise buildings for residential and non-residential purposes in Canada and other markets.

The research program was established in collaboration with FPInnovations, wood industry and design community. The research network consists of about 40 researchers from 11 Canadian universities, FPInnovations, National Research Council of Canada (NRC) and the Canadian Wood Council (CWC). University of New Brunswick is the host university.

NEWBuildS Research Network structure
(11 Universities, FPInnovations, NRC and CWC)

The research activities are organized into four linked research Themes:

- **Theme 1: Cross laminated timber (CLT) – material characterization and structural performance**
- **Theme 2: Hybrid building systems – structural performance**
- **Theme 3: Building systems – fire performance, acoustic and vibration serviceability**
- **Theme 4: Building systems – durability, sustainability and enhanced products**

Each theme is led by a university-based leader and a co-leader from FPInnovations. At the outset, 36 research projects were planned, with the intention that more projects would be added as the network progresses. An estimated 60 highly qualified personnel (HQP) will be trained within the 5-year duration of the network.

NEWBuildS has established an International Expert Panel. Members of the Panel not involved in conducting research within the NEWBuildS network, to provide an impartial review of research proposals and, when necessary, the direction and progress of existing projects.

### 3 RESEARCH PROGRAM

NEWBuildS is investigating the use of traditional light wood frame in mid-rise residential construction, as well as heavy systems built with timber products or with an innovative approach to combine timber with different materials (hybrid system). Because a building is required to be designed to meet the requirements of a number of performance attributes, including structural, fire, serviceability, acoustic and durability, a multidisciplinary team of researchers with these backgrounds have been assembled to conduct collaborative research projects in this network. This will ensure that any proposed solutions will meet the key objectives of the National Building Code of Canada [1].

#### 3.1 THEME 1: CROSS LAMINATED TIMBER

This theme focuses on material characterization of CLT and structural performance of CLT building assemblies. It will generate technical information, such as design properties, product evaluation procedures and system-based uses, in support of the development of a national manufacturing industry for CLT and building applications in Canada. This research will also develop CLT product designs that utilize Canadian forest resources in a sustainable manner.

CLT was developed and used in Europe for over a decade. Each producer has developed its proprietary approach to manufacture the product according to an European Technical Approval (ETA) report. Currently, efforts are underway to develop a European (EN) CLT standard. CLT is attracting considerable attention in North America as this engineered wood product can be an alternative to concrete and steel in non-residential construction as well as non-traditional high-rise wood construction.


Under NEWBuildS, complementary work is being undertaken for further expanding the use of CLT in mid-rise and non-residential construction, where the structural, fire and moisture loads can have different characteristics compared with the traditional low-rise timber frame construction. There have been substantial interactions between Canadian and European researchers who have conducted research on CLT for the past few years. FPInnovations [3] published a CLT Handbook to assist architects and engineers to design buildings with CLT. Based on these interactions and earlier research findings from FPInnovations, and Canadian universities, research topics related to material property characteristics and structural performance of CLT building systems or sub-systems in the Canadian context are identified as priority. Canadian design community is familiar with traditional post-and-beam and stick-frame construction. CLT presents a different type of structural form based on connecting plate-like sub-systems to form...
3.2 THEME 2: HYBRID BUILDING SYSTEMS

This theme focuses on structural performance of mid-rise wood frame buildings and buildings using hybrid construction systems such as heavy timber and innovative approaches that combine wood with concrete or wood with steel.

In April 2009, the province of British Columbia modified its building code to permit the construction of 5- and 6-storey light wood-frame, multi-unit residential buildings based on utilization of ‘conventional’ construction materials and methods. Figure 1 shows the first 6-storey building constructed in B.C. The wood industry and engineering professionals have identified research topics that need to be addressed to improve the design and construction of mid-rise, light wood-frame buildings in Canada. They are design guidance on floor diaphragms to transfer design loads on buildings to walls, cumulative shrinkage due to changes in moisture content of wood, reliable design models for predicting inter-storey drift and fundamental natural frequency of multi-storey wood frame buildings under lateral load and the role played by non-structural components and interaction between wood and other substructures.

Design for mid-rise heavy frame timber systems in Canada is possible but will require special structural engineering design skills on the engineers as there are no guidelines in current design standards. Figure 2 shows a 6-storey glulam structure designed using the alternative solution path in the building code. The challenge can be the design of lateral bracing systems and the interface with reinforced concrete shear walls and connections between members and between frameworks and diaphragms.

The innovative approach is to combine the use of wood with other structural materials such as steel and reinforced concrete to form an efficient hybrid structural system that utilizes the strengths of one material to
address the weaknesses of other materials employed. Buildings can be designed with lower weight of wood and with stability and lateral resistance of a steel and concrete frame. An example of such a hybrid structure is shown in Figure 3.

Research projects under Theme 2 are as follows;

- Development of techniques for multi-functional construction interfaces will identify and investigate interfacing material and system combinations in hybrid-buildings through literature study and laboratory tests.
- Field measurements of mid-rise wood frame buildings will be conducted to measure ambient building vibration and to develop empirical models to predict natural period of mid-rise wood buildings. Rational mechanics-based approach will be developed to calculate lateral drift of wood frame buildings.
- Influence of diaphragm action in light wood frame platform construction on failure and transfer of lateral load to supporting wall elements will be studied using a computer model.
- Design guidelines will be developed for hybrid bracing systems consisting of conventional shear wall and portal frames and of shear walls containing wood-based panels and gypsum wallboard under lateral loads.
- Seismic performance of hybrid building consisting of a light wood-frame sub-system and a rigid elevator core with varying degrees of connectivity between the core and wood frame will be studied using a computer model.
- Moisture-related movements and stresses in components in buildings consisting of a heavy timber frame and reinforced concrete core will be studied using a combination of laboratory testing of scaled model, field measurement and computer modelling.
- Study to develop construction details for implementing wood in-fill wall panels into a reinforced concrete frame through laboratory testing.
- Study to develop design guidelines for implementing wood structural sub-systems such as wall and floor in mid-rise heavy steel frame building through the use of computer models and component testing.
- Development of a composite long-span floor system consisting of an innovative post-tension glulam beam and reinforced concrete slab.
- Study of diaphragm action in heavy-frame ribbed-plated floor systems consisting of thick wood plate on timber, steel and RC beams through the use of finite element modelling and laboratory tests.

3.3 THEME 3: BUILDING SYSTEMS – FIRE PERFORMANCE, ACOUSTIC AND VIBRATION SERVEABILITY

This theme focuses mainly on fire performance of mid-rise and non-residential buildings. It also covers projects addressing vibrational serviceability and acoustic performance of building systems since construction details affect fire performance often have an impact on sound and vibration transmission between compartments.

The objective-based format was introduced in the National Fire Code [4] in 2005 as part of the overhaul of the Canadian building code system. Objectives of each provision in the National Fire Code are specified which facilitate the application of scientific principles and engineering tools to develop alternative solutions that meet the performance objectives and ensure that these alternative solutions provide an acceptable level of fire safety to building occupants. This engineering-based approach allows the selection of building assemblies to be based on actual performance and predicted impact on life safety. To fully capitalize on such an opportunity, there is a need for engineering tools, data, design guidelines and highly trained professionals to be developed.

Canadian researchers have been conducting leading research on vibrational serviceability of wood floor systems for many years and the development of design procedures against excessive vibration in traditional ribbed-plate type wood floor assemblies. There is a need to transfer that knowledge to study the vibrational performance of floor systems built with new engineered wood products such as CLT. Projects to study the acoustic performance of building systems in mid-rise light wood frame and innovative systems will need to consider the construction details developed for fire resistance and structural details in other projects. Researchers working within this theme will interact regularly to ensure that recommended engineering practices and construction details will not be in conflict with each other.

Research projects are as follows;
- Further development of a previously developed fire risk model, CURisk, will be necessary to analyse the
fire performance of mid-rise buildings. The research will include the development of sub-models to predict fire and smoke development and the load-carrying capacity of CLT building assemblies during a fire event.

- Rationalization of life safety objectives and provisions in building and fire codes. The focus will be on analysing code requirements related to fire resistance and sprinklers for mid-rise buildings built with combustible or non-combustible construction and determining the level of safety through the use of the CUrisk model.
- A model previously developed to predict load-carrying capacity of traditional wood floor system during a fire event will be modified to predict the same for CLT floor system. Full-scale test data at the National Research Council will be used to validate the model predictions.
- Fire performance of timber connections used in heavy frame and CLT construction will be studied using the fire testing facility at Carleton University (Figure 4) and computer modelling. This will develop design rules to predict load-carrying capacity of connection during a fire event.
- Influence of support characteristics, such as double-span, support flexibility, and wall flexibility on vibrational performance of CLT floor systems will be studied using a combination laboratory and field testing, and computer modelling.
- Acoustic performance of traditional and innovative wood constructions will be studied to develop appropriate construction details for mid-rise buildings.

3.4 THEME 4: BUILDING SYSTEMS – DURABILITY, SUSTAINABILITY AND ENHANCED PRODUCTS

This theme studies durability and moisture related issues for wood construction in mid-rise and non-residential buildings. It also covers sustainability and enhanced wood products with coating and treatment.

In recent years, considerable interests have been shown by environmentally-conscious design professionals to consider the environmental footprint of a building which can only be assessed through detailed life cycle assessment (LCA) of the building with focus on innovative components of the buildings such as CLT, hybrid wood-steel and wood-concrete structures.

Moisture has an impact on wood such as dimensional stability, decay and mould and is affected by design of building envelope and integrity for moisture ingress and accumulation. The extension of wood construction to 6 storeys and taller, whether using platform-frame, heavy timber, CLT or hybrids, requires attention to building envelope details on moisture management. Furthermore, the moisture load from wind-driven rain and stack effect pressures will be higher due to increase of building height.

Enhanced product performance can be achieved by new finish and treatment. Intumescent coating may be a solution that can address the rapid flame spread on interior finish and rapid strength loss in fire situations. Borate treatment technology is a safe and low environment impact wood preservative that can ensure wood structure will not decay if exclusion of water cannot be assured.

Research projects are as follows:
- The environmental performance of innovative wood building systems using life-cycle assessment programs will be conducted using different databases and impact assessment methods. Various building systems, such as wood and steel construction, massive wood walls (CLT), and the integration of light-frame wood walls in hybrid construction will be assessed.
- Instrumentation and field measurement of several mid-rise buildings will lead to characterization of wind-driven rain load and the effectiveness of overhang on wind-driven rain wetting for mid-rise buildings. The end result is the development of specific recommendations on rain loading and corresponding overhang designs.
- A computational fluid dynamics (CFD) model will be developed to characterize wind-driven rain load on mid-rise buildings and predict the loads on building surfaces with different building geometries and overhang configurations.
- Hygrothermal performance of CLT wall construction in various geographic locations in Canada is investigated through experimental field study at instrumented field exposure facility (Figure 5) and validated by advanced hygrothermal model, e.g. HAMFit.
- Collaborating with FPInnovations and coating industry specialists, the use of intumescent coating to protect engineered wood products from fire damage will be investigated.
- Borate pre-treatment procedure will be developed for CLT and engineered wood-based composites used in mid-rise buildings in areas vulnerable to water ingress or condensation.
- The durability of traditional stud wall construction with various enhanced insulation systems, in accordance with the new energy performance code requirements in Canada will be studied using the test hut shown in Figure 5.

Figure 5 – Environmental test hut, University of Waterloo, Ontario
4 EXPECTED OUTCOME

NEWBuildS is divided into the four themes outlined above for administrative purposes. However, it is more effective and efficient to organize research activities by clusters. Each cluster consists of a number of projects that may deal with an issue from a single theme or multiple theme perspective. This is intended to ensure close interaction between researchers with complementary skills at the early stage of project development and technology transfer by or with FPInnovations. Examples of technology transfer will be submission of code change proposals and preparation of design guidelines and best practice codes. For most of the clusters, there are parallel, usually applied, research or technology transfer projects undertaken at FPInnovations. This formal link will ensure that the university research funded via the Network is complementary to the parallel FPInnovations projects, and to ensure that some of the specific theme objectives are achievable within the 5-year term of the Network.

The anticipated outcomes and achievements of the NEWBuildS research program are:

- To strengthen the national innovation capacity in support of the wood industry and lay the foundation for future technical activities that lead to the expansion of wood use in non-traditional building construction.

- To develop tools for the technical evaluation of CLT, and for predicting responses of selected CLT and hybrid building systems to structural strength and serviceability, fire and moisture loads. (Figure 6)

- To develop technical information in support of the use of wood-based products in mid-rise and non-residential construction for building codes, material design standards and product standards. (Figure 6)

Figure 6 – Technical outcome of NEWBuildS research.

NEWBuildS will reach out for collaborations with other international centres of expertise, create a virtual knowledge gateway and repository for the generated data, and maintain the momentum and synergy among the Network participants beyond the 5-year window of funding.

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REFERENCES

MODEL-BASED PRODUCTION FOR ENGINEERED-TO-ORDER JOINERY PRODUCTS

Niclas Björngrim, Lars Laitila, Samuel Forsman, Peter Bomark

ABSTRACT: When supplying Engineered-To-Order (ETO) joinery products to the construction industry, the manufacturer often takes responsibility for the whole value stream from request for quotation to final assembly on the construction site. There are, however, gaps in the information flow between each actor in the internal supply chain, leading to quality concerns. One of the issues comes from the lack of routines concerning documentation of both changes and additions to the original plan. To make up for the lack of documentation, the craftsmen rely on their skills and experience, often leading to unnecessary and time-consuming ad hoc solutions. Each actor in the chain spends time rediscovering previously known information instead of referring to the documentation. In this paper we suggest a model-based approach, utilizing information and communication technologies (ICT) that enables improved dissemination of relevant information to the involved actors.

KEYWORDS: ETO joinery production, Model-based production, information and communication technologies (ICT)

1 INTRODUCTION

When supplying ETO joinery products for the construction industry, the ETO joinery-products supplier often has responsibility for the whole value stream from quote/order through surveying, production preprocessing, manufacturing, logistics and final assembly on the construction site [1, 2]. Important information in the supply chain is lost and does not reach all the actors involved in the internal supply chain. Forsman et al. [2] found that there is a large amount of waste present when supplying joinery products for construction and suggests that information and standardization in communication are main areas for improvement. The joinery-product suppliers’ process could be enhanced by utilizing 3-D measurement techniques and information and communication technologies (ICT) tools such as Building Information Model (BIM) [3], Last planner [4], Line of Balance (LoB) together with 4-D CAD [5], etc. These tools could help increase dissemination of information in the joinery-products suppliers’ internal value stream. BIM is interesting for the information sharing. However, the information in 3-D BIMs does not generally contain details of a facility and therefore does not reflect how the building actually was built [6]. Joinery-products suppliers need as-built information rather than planned spatial spatial information as a basis for production. The objectives of this paper are to introduce a more efficient information carrier than 2-D drawings. A 3-D model based approach is proposed.

2 METHOD

The focus in this study has been on gaining detailed understanding of the information flow in the value stream for ETO joinery-product supplier supplying to construction. A Swedish ETO joinery-products supplier has been observed through one of its projects. The studied joinery-product supplier is a Swedish association consisting of ten production companies and a co-owned
sales company. Attention has been paid to the information flow from quote/order, surveying, production preprocessing, manufacturing and logistics to final product assembly on the construction site. During surveying, the information needed for defining the products is acquired, and this information is assumed to be of major importance. The lack of information through the internal value stream is assumed to materialise during the assembly, hence the special attention on surveying and assembly. The case study also consists of semistructured interviews with key personnel from the departments responsible for sales, production preprocessing, manufacturing, logistics and assembly. The interviews focused on how the organization relates to the surrounding actors. On-site observations were performed during surveying, during manufacturing in the production facilities and extensively during assembly and have been documented through notes, photographs and audio recordings. To improve the productivity for joinery-products companies, ways to improve the internal process through modelling of information and coordination have been explored. Potential improvements in efficiency resulting from the application of new technology such as 3-D measuring and modelling and principles of information management are discussed.

3 RESULTS AND DISCUSSION
Here the current process for the joinery-products supplier is described and ways to improve the process are discussed. Figure 1 shows the value stream of the current process.

3.1 QUOTE/ORDER
Generally, sales process quotes are made in two steps, a preliminary quote for construction contractors making a quote to a client and a final quote to the construction contractor who received the client order. This procurement process normally involves supplier competition. Sales base their quote on the architectural drawings. By using a BIM, the joinery-products supplier could have a better basis for pricing their product.

3.2 SURVEYING
The production preprocessor determined important dimensions from the architectural drawing. Those dimensions where later measured at the construction site by a surveyor. The current process relies on manual measurement techniques done with folding rulers and tape measures, with 2-D information written directly on a printout of the architectural drawing (Figure 2). The printed drawing acts as the main carrier of information. Faults in drawings can be difficult to detect until on-site assembly has begun. The geometrical information is needed to complete the production preprocessing, and complementary measurements were needed to get all important dimensions. By using 3-D digitizing equipment, the surveying would provide the production preprocessor a complete and accurate depiction of the site. The 3-D data depict the whole on-site environment and negate the need for eventual secondary on-site measurements. For capturing the correct construction site data, coordination between surveyor and construction contractor is necessary.

Figure 2: A simplified drawing that the surveyor used at the construction site to insert all important dimensions.

3.3 PRODUCTION PREPROCESSING
The surveyors’ geometrical verification in 2-D is the basis for defining the joinery products’ dimensions. The production preprocessor defines a product from the given dimensional information, develops production methods and schedules the production start. Since joinery production requires tighter tolerances than construction in general, there is a need to verify the geometrical shape of the environment and compare it to the construction drawings. The geometrical verification leads to dimensional uncertainties. The dimensional uncertainties make it necessary to produce products to be adjusted to fit during the on-site assembly. With an accurate 3-D depiction of the adjacent environment, joinery products could be produced to fit without adjustments.

3.4 MANUFACTURING
Manufacturing of the products is performed on the basis of information from preprocessing. This information is mediated mainly on 2-D CAD drawings and a manufac-
turing bill. A production plan is used to show the time demand for the manufacturing. Errors in the joinery product are hard to discover from the 2-D model. A 3-D CAD model gives the opportunity to inspect the joinery product before manufacturing and can minimize time-consuming design errors (Figure 3). 3-D CAD also gives the opportunity to automatically generate tool paths in CAM software.

Figure 3. Design error, the laminate is missed on the top of the lower shelf.

3.5 LOGISTICS

After manufacturing, the products are packed to fit on the pallet, rather than grouped to facilitate assembly. A product declaration of content is supplied with the parcel; however, observations show that this is an area of frequent failure. Lack of complete information caused delays in assembly when the personnel had to open several parcels to find the related components. There was no concern of the internal logistics on the construction site when the parcels were put together. The parcels did not fit the in-transport routes of the site, forcing the assembly personnel to open the parcels and carry the components to the right floor (Figure 4). This is not only time consuming, but also involves the risk of damaging the joinery products and causing injuries to personnel due to bad lifting positions. The shipping of parcels should be coordinated with the needs of assembly, rather than the time of manufacturing. Visualization of in-transport routes and the final product could eliminate these problems.

Figure 4: Parcel opened and carried into elevator due to parcel design not fitting in-transport route.

3.6 ASSEMBLY

The planning of the assembly is performed concurrently with production. Since projects are geographically spread, the strategy applied is to contract assembly contractors close to the construction site. The main tasks of the assembly planning are to contract assembly and coordinate with production. The product assembly is performed on site and requires time to develop understanding of the product to be assembled. During assembly, there are no instructions provided, and the information from the 2-D drawings is used as support, sometimes with additional sketches from the preprocessor. Having a 3-D model of the site would be of help for the construction personnel to visualize the joinery product and the environment where it will be assembled. Assembly instructions or exploded views were not supplied to the assembly contractor. The planning of the assembly has a start date and a stop date, but there is no daily or weekly planning. Coordination between the manufacturer and assembly personnel is needed as well as with other actors performing work on the construction site. The assembly contractor needs to communicate with the production preprocessor in order to develop an understanding of how to assemble the product. Communication was done by phone, but could have been avoided if good assembly instructions had been
provided. Because of the lack of spatial information about the construction site, products are manufactured to be adjusted to fit. This process requires lot of craftsmanship instead of assembly work. Assembly problems are solved ad hoc and the root of the problems is not investigated. Having complete spatial dimensions of the site would increase the level of prefabrication of the joinery products, thus making assembly rely less on craftsmanship and more on installation work.

3.7 MODEL BASED PRODUCTION

![Figure 5. Value stream map of the improved process and how information should be built.](image)

To enhance the process and keep the information updated and accessible for the suppliers in the whole value stream, we propose a 3-D model-based process as seen in Figure 5. Information is created through the whole value stream and information dissemination between the actors is improved. In order to improve the quality of joinery products for construction, more of the as-built spatial information needs to be known and to be more accurate than in the current process. The 3-D information must be based on as-built dimensions rather than as-planned spatial dimensions of BIMs [6]. To facilitate dissemination of project information to the different actors in the internal value stream, it is suggested that all information be retained in digital format. When sales receive the order, a visualization of the site provides a better basis for estimating the quote. Surveying with a 3-D digitizer enables acquisition of the spatial information needed for creating products with good fit and provides a tool for visualizing and planning of assembly. By using 3-D CAD, the fit of the model can be assured virtually in the 3-D scan or in a surface/solid model created from the scan data (Figure 6).

![Figure 6. 3-D model of construction site.](image)

The model consists of a wide range of information such as dimensions, surface finish, tolerances and URLs. In this stage, the model can also be prepared for machining (CAM). If the manufacturer has a better understanding of the final product, it is easier to send parcels with the right components at the right time. The manufactured parts can be sent correctly packed, in parcels that fit in-transport routes at the construction site. The visualization of the site is used to make sure that parcels will fit in-transport routes, eliminating the need for repacking of the parcels. During the assembly, the 3-D scan together with the CAD models can be used with various planning tools [3, 4] to make assembly more efficient by visualizing where a product should be placed and when.

4 CONCLUSIONS

In the current process, the main information carrier is printouts of architectural drawings with added information. This information can be difficult to interpret and needs to be further processed in order to create CAD models for production. The information in the drawings offers little support for the assembly. A higher degree of utilization of ICT and planning tools is suggested in order to create accurate and accessible information for all actors in the internal value stream. When information is available to all involved actors early in the process, a more concurrent approach is possible, which means that several tasks can be done in parallel. Providing better spatial measures results in joinery products with higher quality and better fit, which will make assembly less time consuming. The knowledge gained from different projects could be accumulated and used to facilitate future projects.

ACKNOWLEDGEMENT

The research work was carried out with the help of funding by VINNOVA and European Union Objective 2, which is very much appreciated.

REFERENCES


1 INTRODUCTION

1.1 AWARENESS OF ISSUES
After reversion of the Building Standard Law of Japan (BSL) in 2000, some wood-based Fire-Resistive structures have been developed. Wood-based structures bring social benefits, because of their low environmental impacts during producing. The other hands there are less personal benefit, because of strict requirements on fireproof regulations and the costs for fulfilling the required performance in Japan. Since 1990’s some of European countries have started to abolish limitations for maximum number of storey on wood-based buildings, and 7 to 9-storey of the buildings were built. These wood-based structures were selected as low cost structure. Both Japan and Austria allow building only up to 4-storey wood-based structures. What are the similarity and difference of situations in both countries?

1.2 METHODS AND AIMS
We tend to have certain prejudice or non-verbal knowledge on use of wood in each domestic culture, because of traditions of material. Sometimes our background educations make a fixed idea about future solution. Law can be said a kind of “written domestic common sense”. Buildings code shows and decides typical building type in each area. Comparison of “common sense on wood” can be useful not only for basic knowledge about technical transfer or exchange but also for recognition of blind spots about prejudice.

1.3 GOAL OF THIS RESEARCH
Fundamental mutual understanding for development of Wood-Based Open Building System through international joint research is goal of this research. The research has three phases for; 1) technical benchmarking, 2) social background, and 3) system development. (See Fig.1) This paper covers conclusion of phase 1 and perspective on phase 2 and 3.
2 FRAME OF DISCUSSION

In this chapter, frame of discussion is described through risk management of fire.

2.1 FRAMES AND FACTORS

Countermeasure of fire are categorised into four ways by level of damage and frequency. [1]

Figure 2 shows countermeasures of fire and related factors, and each topic in this paper is discussed based on these factors.

![Figure 2: Factors of Risk Management](image)

2.2 AXES AND ASPECTS

Group of the factors are discussed along to certain axis or aspect in each chapter.

2.2.1 Technical Benchmarking (Chapter 3)

Structural solutions on fire-resistive buildings are categorised into “Technology” on top of lower “individual” triangle in Figure 2. Comparative techniques work as “benchmarking” of further social scientific comparison.

2.2.2 Benchmark and Background (Chapter 4)

Based on benchmarking, 1) social background on fire proof philosophy and 2) division of responsibility between society and individual are being explained.

First topic treats upper “social” triangle, and second one treats an axis between “social” and “individual” triangle in Figure 2.

2.2.3 Decision-Making (Chapter 5)

Tendency of decision-makings within “social” and “individual” triangles are being analysed.

2.3 WORDING

Vocabularies for comparison are frequently used in this paper, and these wordings of this paper are given in below.

GROUPING

✓ “International” is used for group of countries, “National” is used for one country, and “Local” is used for smaller legal areas within one country.
✓ “Individual” is used for space, a person or a group what is covered same one contract or one legal obligation on construction. “Social” is used for a gathering of space cells, persons or groups.

CLASSIFICATION

✓ “Level” is used for a certain group witch is classified by physical scale or area.
✓ “Class” is used for classification of property of materials or material combinations.

COMPARISON

✓ “Similar” is used for specific techniques with same physical component or combination of items in lower level.
✓ “Equivalent” is used for theory or philosophy of technique for same aim and property.
✓ “Something (A)” and “something (J)” are used for Austrian and Japanese things.
3 TECHNICAL BENCHMARKING

Structural solutions on fire-resistive buildings as benchmark are explained in this chapter.

3.1 LEGAL SOURCE AND OVERVIEW

Scopes of comparisons on this chapter are for requirements on structural performance of wood-based buildings based on BSL, BSL enforcement ordinance (BSL-EO), Austrian Institute of Construction Engineering Guideline (OiB), and Eurocode. Target building types are apartment houses and offices as typical multi-storey buildings.

The comparisons are described in each level of building parts and types of regulations. Figure 1 shows these combinations; key words of contents and key similarity and difference.

![Figure 3: Key Similarity and Differences](image)

3.2 BUILDING LEVEL [2, 3]

3.2.1 Factors of Classifications

The both building types are classified by four factors, 1) main occupation, 2) special building or not, 3) number of floors, and 4) zone.

The buildings are categorized by main occupation, but definitions of special buildings are different. Zone exists only in Japan.

3.2.2 Classifications of Buildings

Fireproof requirements for building are fulfilled by combination of property of structures and equipment. This chapter discusses requirements for structures.

In this paper, both Quasi-Fire-Resistive Structure (J) and structures for up to GK4 (A) (up to REI-60) are called QFR-Class structure. And both Fire-Resistive Structure (J) and structures for more than GK5 (A) (more than REI-90 + A2) are called FR-Class structure. FR-Class structure is set expression with use of non-combustible material in general prescriptive codes in both countries.

For both FR- and QFR-Class structures, Japan requires fireproof from interior and exterior (double side), and Austria requires that from interior to exterior (single side).

3.2.3 Coverage of Law

QFR-Class structure can be used up to 3-storey and 3000m² buildings with fire compartments (each 500m²) in Japan. And in Austria, it can be used up to 4-storey buildings and no limitation on total area with fire compartments each 60m or 1200m².

In Japan, 60min-FR-Class structures can be used up to 4-storey from the top and no limitation on total number of floors of the building. In Austria, 90min-FR-Class structure can be used up to 7-storey buildings, and it is necessary to get special permission to build more than 8-storey.

3.3 STRUCTURE AND MATERIAL LEVEL [4]

In material and structure level, test methods are same or based on same philosophy, and these are similar or comparable, even if contents of tests or classifications are different from each other. FR-Class structures seem equivalent and QFR-Class structures can be said similar.

Differences on 1) extra requirement time on large-scale fireproof test on FR-Class structure and 2) measurement of smoke produce in material level are related to differences on building level.

3.4 CONCLUSIONS ON TECHNICAL COMPARISON

To summarize conclusions, technical solutions in structure and material level are similar or comparable, but differences in building level come from social systems and backgrounds.

3.4.1 Similar or Equivalent Techniques

Similar technique can be a benchmark of research on social system and background.

Equivalent technique can be a potential solution of construction in different background. Technical detail of FR-Class structures is being described in the other paper.

3.4.2 Differences on Building Level

There are two main relations between building and structure / material level. (See Figure 3)

The first one is an applicable area on building level. QFR-Class structures are applicable to larger buildings in Austria than in Japan.

The second one is prohibition on use of materials. In Austria, results of material tests affect to upper scale level; e.g. prohibition in material level is valid in structure / building level. In Japan, wood can be used as FR-Class structure even it is classified into out of classifications.

3.4.3 Open-Ended Question on Social Background

Differences on use in building level are considered with social scientific research in chapter 4.
4 BENCHMARK AND BACKGROUND

Outline of discussions on social background are described in this chapter.

4.1 OVERVIEW

In this section, overview of costs for “social” factors (upper triangle in Figure 2) is shown.

4.1.1 STATISTICS

“World Fire Statistics” [5] gives international statistics on fireproof costs by percentage of GDP. Focusing on sum of costs for 1) fire-fighting organization and 2) insurance (a: administrative cost of insurance company plus b: direct loss as payment of the insurance), Japan (0.52%) and Austria (0.51%) belong to same cost level. Range among ca. 20 countries from 0.12% (Singapore) to 0.60% (Belgium). From the viewpoint of major cost, Japan is categorized into “fire-fighting-major” group with the U.S. and Canada. Austria is in “insurance-major” group with U.K. and Sweden.

4.1.2 FIRE-FIGHTING

Organizations of fire fighting are part of local autonomy in both Japan and Austria. Fire fighters are categorized into full-time worker or the other. Percentage of full-time fire fighters is 15% in Japan and 0.1% in Austria. Number of fire fighters per million people is ca. 1300 in Japan and ca. 0.75 in Austria. [6] [7] [8]

4.1.3 INSURANCE

Insurances are categorised by type of object and organization. Life insurance or non-life insurance are in first category, and public insurance or private insurance are in second category. Fire insurance is non-life and private insurance in both countries. [9][10] Each insurance law in Japan [11] and in Austria [12] provides rights and obligations of insurance contract. The persons concerned and the accidents concerned are same, but coverage of compensations and ways of damage estimations different in each law. Premium of fire insurance in Japan is tax free, but that in Austria is taxed (rate: 8%) as a local tax, fire insurance tax. [13][14][6]

4.2 ANALYSIS AND FUTURE RESEARCH

Based on overview of “social” aspects, cost proportion on fire fighting and insurance are not same in each country.

4.2.1 RELATIONS

Focusing on tax and budget, fire fighting and insurance are independent to each other in Japan, and these two have economical relation through tax in Austria.

4.2.2 DIVISIONS

Focusing on cost proportion of fire fighting and insurance, divisions of obligation on society and individual seem different in each country.

4.2.3 FUTURE RESEARCH

In future research, 1) duty and obligation on insurance contract and 2) coverage of building code and fire service law will be defined on the basis of technical benchmarking in chapter 3. Expected results of this chapter are fundamental discussion for; What should be supported on building structures, and what should be supported from social systems?

5 BACKGROUND OF DECISION

Viewpoints of additional research on decision-making on buildings are informed in this chapter.

5.1 LONG-TERM AND SOCIAL

In the middle of the 19th Century, Tokyo and Vienna had around 850 thousands inhabitants, and started new city construction at that time. Tokyo grows up to 12 million-city, and Vienna stays as 1.7 million-city. [15][16] Key difference on social decision and treatment of wood will be analysed in future research from the viewpoint of history of legal frame establishments and new city constructions.

5.2 LONG-TERM AND INDIVIDUAL

The other point will be typical utilisation of wood in buildings in each city. And then, evaluation on wood-based buildings will be defined through by tax and insurance rates, and its influence on long-term decision will be analysed.

6 WOOD-BASED OPEN-BUILDING-SYSTEM

Basic idea on wood-based open-building-system is explained in this chapter. Results of technical and social scientific researches are combined into basic proposal on construction system.

6.1 WOOD BUILDING

There are potential global demands on wood-based buildings for sustainable society. On the other hand, individual have to hold additional risk on combustibility of wood, when they use or own these wood-based buildings.

6.2 WOOD- OR OPEN- BUILDING

In general, middle raised wood-based structures are expensive in Japan and Austria because of strictness of fire regulation. Also primary structural cost of planned open-building system is more than normal structure because of investments in potential flexibility.
6.3 WOOD AND OPEN BUILDING

If flexible systems work for both positive and negative change, the system can be invested as a kind of insurance. Wood-based and open-building-system can be positive combination for economical balance between primary investment and potential flexibility. Technical solution, social backgrounds and tendency of decision-makings should be count into design on this system.

6.4 PRACTICAL DEVELOPMENT

Because of rapid growth of Tokyo, we have had many chances of technical innovation on architectural components, and almost all the buildings in the city have been rebuilt after the middle of 19th Century. In the other hands, Vienna has kept a lot of historical architectures. Japan has the latest knowledge about technical aspects, but Austria seems more stable in aspects of social system. We expect to discuss proper combination of our knowledge through joint study among universities in Tokyo and Vienna.

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TIMBER BUILDING FIRE PROTECTION RESEARCH IN CHINA- PAST, PRESENT AND FUTURE

Qiu Peifang¹, Ni Zhaopeng²

ABSTRACT: After joining WTO, China’s market gradually opened to the world. Many wood councils and wood product companies of foreign countries came to China to find their opportunities. As a result, more and more timber buildings have been built all over China. In order to ensure the fire safety of the timber buildings, Tianjin Fire Research Institute of MPS (TFRI), the chief editor of the most important fire code of China, has been keeping a close eye on the safe development of timber buildings. Therefore, a series of researches on timber fire protection have been done and parts of the achievements have been integrated into the fire code. This paper introduces what we did in the last few years in timber fire protection research, what we are doing now and what we will do in the near future.

KEY WORDS: timber fire protection, fire code, building elements, research

1 INTRODUCTION

Timber buildings are very popular in Canada, USA, Australia, New Zealand and some countries in Europe. Accordingly, the building or fire codes of those countries have a series of requirements for the fire protection of timber buildings. However, masonry, reinforced concrete and steel structures were the fashion in China in 20th century, and fire protection researches were mostly focused on them, while timber fire protection was more or less neglected. Accordingly, fire code of China at that time was focused on the requirements for masonry, concrete and steel structures too. After China joined WTO, more and more wood councils and companies from North America, Europe and so on set up offices or branches in China to find market opportunities or promote cooperation with some research institutes and organizations with the purpose of studying the applicability and sustainability of wood frame buildings in China. As a result, a certain number of wood frame buildings (mostly single-family dwellings) have been built in Beijing, Tianjin, Shanghai, Hangzhou, Qingdao, Dalian, Guangzhou etc. In order to provide proper regulations for the construction and fire protection of wood frame buildings, a series of codes have been issued in China. For example, Code for Quality Acceptance of Timber Structure Engineering in 2002, Code of Design on Building Fire Protection and Prevention in 2006. However, the fire protection requirements in these codes need to be further improved and coordinated.

2 TIMBER BUILDING FIRE PROTECTION RESEARCH IN THE PAST FIVE YEARS

2.1 RESEARCH PROJECTS AND ACHIEVEMENTS

In order to ensure the fire safety of timber building in China, as the main writer of the national fire code, the Code Division of TFRI began to initiate a series of research projects related to timber fire protection. In September of 2005, fire experts from TFRI, Canada Wood (CW), AP&PA and European Wood(EW) met in Beijing to discuss issues related to timber fire protection in China. As a result, a joint technical committee was set up and a long-term research plan was agreed among cooperative parties. The joint technical committee (JTC) comprises of fire experts from the four participating parties, including officials from Ministry of Construction and the Fire Bureau of the Ministry of Public Security of China. Review of some relevant regulations and standards of Canada, USA, Germany and Sweden was the first step of the joint research. The research items of the standard comparison include the combustibility and fire resistance rating of the main structural elements, limits for height and area, compartmentation and spatial separation between buildings etc.

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Two years’ research on codes and standards of different countries was quite fruitful. As a result, one chapter has been added to the national fire code to regulate the fire protection issues of timber buildings. In this new chapter of the fire code, items such as the maximum storeys of the timber building, the fire resistance rating of the main structural elements, the permitted length and height, maximum area, spatial distance, safe evacuation and fire protection of hybrid buildings and so on have been defined. See table 1, 2, 3 for the details of these requirements[1].

**Table 1 Combustibility and FRR of the structural elements for timber buildings**

<table>
<thead>
<tr>
<th>Structural elements</th>
<th>Combustibility and FRR (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fire wall</td>
<td>Non 3.00</td>
</tr>
<tr>
<td>Load-bearing wall, wall between families, walls around the staircase</td>
<td>DC 1.00</td>
</tr>
<tr>
<td>Walls of the elevator shaft</td>
<td>Non 1.00</td>
</tr>
<tr>
<td>Exterior nonload-bearing wall, walls along the exit passage way</td>
<td>DC 0.75</td>
</tr>
<tr>
<td>Room partition wall</td>
<td>DC 0.50</td>
</tr>
<tr>
<td>Column</td>
<td>C 1.00</td>
</tr>
<tr>
<td>Beam</td>
<td>C 1.00</td>
</tr>
<tr>
<td>Floor</td>
<td>DC 0.75</td>
</tr>
<tr>
<td>Roof structural elements</td>
<td>C 0.50</td>
</tr>
<tr>
<td>Egress stairway</td>
<td>DC 0.50</td>
</tr>
<tr>
<td>Ceiling</td>
<td>DC 0.15</td>
</tr>
</tbody>
</table>

Note: Non = non-combustible  
DC = difficult combustible  
C = combustible

**Table 2 Maximum permitted storeys and building height**

<table>
<thead>
<tr>
<th>Types of timber building</th>
<th>Traditional timber building</th>
<th>Light-weight timber structure</th>
<th>Gluelam structure</th>
<th>Hybrid building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permitted story (story)</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Permitted building height (m)</td>
<td>10</td>
<td>10</td>
<td>NL</td>
<td>15</td>
</tr>
</tbody>
</table>

**Table 3 Maximum permitted length between fire walls and area of each level**

<table>
<thead>
<tr>
<th>Story (story)</th>
<th>Maximum permitted length between fire walls (m)</th>
<th>Maximum permitted area of each level between fire walls (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>1800</td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>900</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>600</td>
</tr>
</tbody>
</table>

**Table 4 Spatial distance between timber building and other civil buildings (m)**

<table>
<thead>
<tr>
<th>Building type</th>
<th>Type A, B</th>
<th>Type C</th>
<th>Timber building</th>
<th>Type D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber building</td>
<td>8</td>
<td>9</td>
<td>10</td>
<td>11</td>
</tr>
</tbody>
</table>

From the above tables, we can see that in China, the maximum permitted storeys of timber building is 3, which is lower than many other countries. However, we must notice that researches on timber fire protection have just started and timber buildings are not popular in China. Both the construction technology and fire safety management of timber buildings need to be improved in the future.
2.2 VERIFICATION TEST

In order to provide scientific support for the revision of current national fire code *Code of Design on Building Fire Protection and Prevention*, the JTC decided to conduct a series of verification tests in the lab of TFRI. The test assemblies were designed by European Wood and constructed by Canada Wood. The studs for wall assemblies, beams and columns were provided by European Wood, but all the other materials such as the gypsum-board and rock fibre etc. were made in China. These tests were done according to GB 9978 (equivalent to ISO 834) and observed by experts from Joint Technical Committee of China, Canada and Germany[2]. Twelve full-scale timber building elements were tested in the lab of Tianjin Fire Research Institute. They include 7 load-bearing or non-load-bearing exterior or interior wood stud walls, one independent ceiling, two floors, one gluelam beam and one column. The furnaces for conducting full-scale tests are shown in Figure 1, 2 and 3.

![Wall furnace](image1)

*Figure 1: Wall furnace*

![Floor and beam furnace](image2)

*Figure 2: Floor and beam furnace*

![Column furnace](image3)

*Figure 3: Column furnace*

See table 5 for the fabrication, dimension and results of the full-scale tests[3].

<table>
<thead>
<tr>
<th>Name of the assembly</th>
<th>Fabrication (mm)</th>
<th>Dimension (mm)</th>
<th>Load (kN/m)</th>
<th>Result (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-bearing exterior wall</td>
<td>Gyp-board -12; Wood stud-89; Glass wool -89; Gyp-board -12;</td>
<td>3270 × 3270 × 113</td>
<td>2.5kN/m After reaching to 60min, increase 2.5kN every 5min.</td>
<td>72</td>
</tr>
<tr>
<td>Non-bearing exterior wall</td>
<td>Gyp-board-15; Wood stud-140; Rock fiber-140; Gyp-board -15;</td>
<td>3270 × 3270 × 170</td>
<td>2.5</td>
<td>99</td>
</tr>
<tr>
<td>Load-bearing exterior wall</td>
<td>Gyp-board-15; Wood stud-140; Rock fibre-140; OSB-15</td>
<td>3600 × 3300 × 170</td>
<td>22.5</td>
<td>64</td>
</tr>
<tr>
<td>Load-bearing interior wall</td>
<td>Gyp-board-12; Wood stud-89; Glasswool-40; Gyp-board-12</td>
<td>3300 × 3600 × 113</td>
<td>11</td>
<td>47</td>
</tr>
<tr>
<td>Load-bearing exterior wall</td>
<td>Gyp-board-12; Wood stud-89; Glass fibre-80; OSB-12</td>
<td>3600 × 3300 × 113</td>
<td>12.5</td>
<td>34</td>
</tr>
<tr>
<td>Load-bearing interior wall</td>
<td>Gyp-board-15; Resilient metal slat-13; Wood stud-89; Rock fibre-80; Gyp-board-15</td>
<td>3300 × 3600 × 132</td>
<td>2.5kN/m After reaching to 60min, increase 2.5kN every 5min.</td>
<td>72</td>
</tr>
<tr>
<td>Non-bearing interior wall</td>
<td>Gyp-board-15; Wood stud-89; rock fibre-80; Gyp-board-15; Two frames put together</td>
<td>3600 × 3300 × 243</td>
<td></td>
<td>183</td>
</tr>
<tr>
<td>Floor</td>
<td>Two layers of gyp-board-12; Resilient metal slat-13 Beam-235;</td>
<td>3065 × 4500 × 290</td>
<td>2.5</td>
<td>72</td>
</tr>
</tbody>
</table>
With the help of these verification tests, we have obtained the fire resistance ratings of some typical structural elements as well as their burning behaviors under standard fire. The data obtained from these tests have agreed well with those obtained from the similar assembly tests done in Canada and Germany. It indicates that China’s national fire code can adopt the data of those timber assemblies listed in the building codes of those countries like Canada. Of course, we will do further research on timber fire protection so as to provide more scientific supports for the revision the fire code.

### 3 CURRENT TIMBER BUILDING FIRE PROTECTION RESEARCH

#### 3.1 RESEARCH ON BUILDING HEIGHT AND NUMBER OF STOREYS OF WOOD HYBRID BUILDINGS

As to the height and number of storeys of wood hybrid buildings, different countries have different requirements. Even in China, the requirements for them in some local standards are different from those of the national standards. In order to find a balance in China and to provide technical support for the national fire code, TFRI initiated a research on the comparison of fire risk of a 7-story concrete building and a 7-story (4-story concrete+3-story timber) hybrid building.

A 7-story civil building with typical layout has been chosen to be the study object. See Figure 4 and Figure 5 for the layout of the 7-story concrete building.

#### 3.2 STUDY ON FIRE PROTECTION OF TIMBER BUILDINGS WITH MORE THAN 3 STOREYS

With the help of computer modelling and event tree analysis, result has been found that the fire hazard of a 7-story concrete building is more or less the same with that of a 7-story hybrid building (4-story concrete+3-story timber), which provides a robust support for the requirements of hybrid building fire protection in the national fire code.

<table>
<thead>
<tr>
<th>Component</th>
<th>Material</th>
<th>Dimensions</th>
<th>Fire Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>Rock fibre-80; OSB-18</td>
<td>3065 × 4500 × 275</td>
<td>4, 36</td>
</tr>
<tr>
<td>Ceiling</td>
<td>Gyp-board-12; Resilient metal slat-13; Beams-235 Glass fibre&gt;60; OSB-12</td>
<td>3000 × 4500 × 289</td>
<td>43</td>
</tr>
<tr>
<td>Beam</td>
<td>Gluelam 200 × 400 × 5100</td>
<td>19, 83</td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>Gluelam 200 × 280</td>
<td>80, 90</td>
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Figure 4: FDS model of a 7-story concrete building

Figure 5: Floor plan

With the help of computer modelling and event tree analysis, result has been found that the fire hazard of a 7-story concrete building is more or less the same with that of a 7-story hybrid building (4-story concrete+3-story timber), which provides a robust support for the requirements of hybrid building fire protection in the national fire code.

As we know, story and height restrictions for timber buildings in the world changes with time. According to the survey done by Birgit Östman and his team, in Europe, most countries had strict restrictions for the number of storeys of timber buildings before 1990. However, 10 years later, timber buildings with more than 5 storeys have been permitted to be built in most European countries. A further increase in permitted use is expected in the near future. See Figure 6 for the details[4].

The ease of the restrictions for the use of timber structures for higher buildings in the national regulations of some European countries has brought great attention to the fire regulators of China. In order to study the feasibility of using timber structure for higher buildings...
in China, TFRI began to do a survey of the demo projects as well as the related research and fire protection measures for higher timber buildings at abroad from last year.

![Figure 6: Changes of maximum number of storeys in timber in Europe](image)

1) Study on active fire protection measures for timber buildings;
2) Further study on full-scale fire test of timber elements;
3) Study on joint fire protection;
4) Study on spatial distance between timber buildings and other civil buildings;
5) Study on conditions and requirements of safe evacuation measures;
6) Further study on the feasibility of the use of timber structures for higher buildings.

5 CONCLUSIONS

According to the survey, it has been found that the use of timber structures for higher buildings is just in its preliminary stage and the application of this kind is still limited. The occupancy issue, fire safety performance and fire protection features installed etc. need to be tested by time. Therefore, it is suggested that use of timber structures for higher building should be restricted.

4 RESEARCHES ON TIMBER BUILDING FIRE PROTECTION IN FUTURE

Timber building fire protection research in China is still at an initial stage. More researches should be done in the future. TFRI is the main writer of the fire code of China, therefore, we focus more on obtaining scientific technical data to improve the requirements for timber fire protection in the fire code. In order to improve the fire code, the followings should be done in the near future:

![Image](image)

REFERENCES

TIMBER FRAMED BUILDINGS AND NZS 3604

Roger Shelton¹, Graeme Beattie²

ABSTRACT: This paper briefly charts the history of the New Zealand Timber Framing Standard (NZS 3604), gives some of the background to the recent revision, and presents the initial findings of a survey conducted to establish the performance of 3604 type buildings during the recent series of earthquakes in Canterbury.

KEYWORDS: Light timber framed buildings, NZS 3604, Canterbury earthquake

1 INTRODUCTION

Stick framed timber buildings have been used for residential and low rise building structures in New Zealand for over 100 years. The essence of stick framing is the use of many, small section, timber elements of low structural capacity distributing the applied loads in conjunction with “non-structural” elements such as linings and claddings. This approach builds redundancy into the structure in two ways:

- Failure of one member allows re-distribution of loads through alternative load paths
- Practical framing arrangements provide more structure than is strictly necessary (ie walls which are ignored for bracing purposes).

Redundancy goes a long way to providing resilient buildings which are able to cope well with

This is in contrast to the philosophy behind typical “engineered” structures which use fewer, larger and stronger members, each carrying much heavier loads. The stick frame approach has evolved to suit New Zealand’s timber resource and the carpentry skills of its construction workforce. Thus, much of the development and fine tuning of the system over the years has evolved from the ground up.

Timber framed buildings now represent about 95% of structural timber usage (by volume) in New Zealand. They also constitute about 95% of the existing residential building stock, although the proportion of new buildings is slightly lower as new systems (such as light gauge steel framing) are introduced.

2 BACKGROUND TO NZS 3604


In applying engineering rationale to a well developed, “sorted” building system, the standard was ground breaking when it was first published in 1978. By placing limits on the size and scope of buildings covered, safe but not unduly conservative solutions are provided for a wide range of low-rise timber buildings without the need for the involvement of structural engineer designers. Thus the cost of the inherent conservatism is balanced against the savings in structural design fees, where the simplicity of the design does not warrant such close attention. Those limits govern the building size, height and roof slope, floor loadings, and snow loadings. Wind and earthquake loads are also limited by restricting the zone or area in which the building may be situated. For buildings outside these limits, a specific structural design is required for each building.

Both documents have been revised and updated several times since they were first written but remain at the core of timber framed building construction in New Zealand.

Over the years since 1978 most critical elements of the building structure covered in NZS 3604 have been verified by calculation where possible, or by test where appropriate. The latter is particularly true of fixings. As changes in building technology occur and new systems and techniques are developed, this testing and verification process continues.

2.1 CHANGES IN 2011

The current revision of the Standard was conceived as a "Limited technical review", and focuses on five main areas:

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² Graeme Beattie, BRANZ, 1222 Moonshine Road, Judgeford, New Zealand. Email: graeme.beattie@branz.co.nz
i. Updating to accommodate the change in the cited Loading Standard from NZS 4203 to the AS/NZS 1170 series [3].


iii. General improvement of readability and clarification

iv. Updating of durability requirements with other Standards

v. Updating to reflect new building techniques and materials, in particular engineered timber products.

The change of loading standard from NZS 4203 to AS/NZS 1170 meant revisiting all the selection tables in NZS 3604. However the net effect for users is generally quite minor. Two areas where changes were more significant were in earthquake and snow loads. Greater understanding of New Zealand’s seismicity and the effects of soil types resulted in quite major changes to the earthquake loading standard. These were treated in NZS 3604 by picking a default demand level \((z = 0.46, \text{ and soil class E})\) and providing a table for other conditions.

Changes to the snow loading standard prompted fine tuning of the treatment of snow loads in NZS 3604, and by adjusting the altitude boundaries of the snow zones and basic ground snow loads, much great coverage around New Zealand was possible with negligible penalty in terms of allowable member spans. More detailed information on these changes is described in The Engineering Basis of NZS 3604 [5].

The provisions for durability have changed to better align with other Standards, in particular the corrosion zone classifications used in ISO 9223 [6]. The aggressive nature of timber treatments high in copper towards galvanised fasteners has also been recognised, with greater requirements for stainless steel fixings in exposed situations.

Use of engineered wood products (LVL and Glulam) and proprietary roof truss systems has been extended and clarified.

Engineered wood products may be used either as direct substitutes for sawn timber, or as an equivalent proprietary system, provided the system has been engineered in accordance with NZBC B1/VM1, and the load reactions do not exceed 16 kN to avoid overloading the rest of the timber framed structure. The procedure for design checking and construction of trussed roofs were developed in conjunction with the Frame and Truss Manufacturers Association [7] and are consistent with their Code of Practice. Girder truss loads and reactions are also limited to 16 kN.

3 THE PLACE OF NZS 3604 IN THE NEW ZEALAND BUILDING INDUSTRY

NZS 3604 is a technical document, written and developed by the industry to suit New Zealand’s building environment. It is aimed primarily at “designers” although there is a lot of information relevant to builders, particularly in the area of fixings and connections.

With the introduction of the NZ Building Act in 1992 [8] and the NZ Building Code (NZBC) shortly after, NZS 3604 slotted neatly into the building control hierarchy as a series of pre-engineered solutions for the structural safety, durability and weather tightness aspects of timber framed building construction. It thus became referenced under the NZBC as an “Acceptable Solution” for timber framed buildings. This means that it is the preferred, but not the only, way of constructing timber buildings in New Zealand. Buildings, or parts of buildings, which fall outside its scope require specific structural engineering design to verify that they meet the intent of the NZBC. This has become common practise, with the basic building covered by NZS 3604 but an increasing number of specifically designed components such as engineered wood products, bracing panels, timber connectors and so on.

NZS 3604 provides a framework for all participants of the building industry, for example the wind zones are used by component manufacturers (such as windows), and have also been translated into the Building Code compliance documents for weather tightness. The Standard is also used as a benchmark for suppliers and manufacturers to design and produce their own proprietary products and systems, which are designed and tailored to fit within its design philosophy.

To aid this process, the background, engineering basis, and critical parameters are set out in [5].

Over the 34 years of NZS 3604’s existence, New Zealand building styles have changed dramatically, and this has proved to be NZS 3604’s greatest challenge. The 100 square metre rectangular box has disappeared and today’s houses are larger, have fewer walls, more windows, and are built in more exposed locations, thus placing much greater demands on the structure. There are continual requests from many quarters to include more situations, materials and systems, and as a result the document has increased in size from a modest A5 pamphlet, into a large ring binder with over 400 pages.

4 CANTERBURY EARTHQUAKES

4.1 INTRODUCCION

The revision was almost complete late in 2010 when the Standard was put its greatest test, the series of earthquakes in Canterbury lasting, with aftershocks, until the end of 2011. The majority of residential buildings in Christchurch are timber framed structures, and up to half of them would have been influenced, either wholly or in part, by
NZS 3604. Thus, its provisions and design philosophy got a good road test.

4.2 SEPTEMBER 2010
The majority of building damage after the September 2010 event resulted from liquefaction and lateral spreading of the foundation sub-soils, as shown in Figure 1. This resulted in widespread consequential damage to timber superstructures as shown in the figure. However, buildings experiencing only shaking damage fared well in general, and no collapses of timber framed buildings occurred.

![Figure 1. House severely distorted by ground movement and lateral spreading.](image)

4.3 FEBRUARY 2011
The February 2011 event produced ground shaking up to 2 times the ULS design levels, and damage to timber framed houses was much more widespread, particularly on the hillside suburbs near the epicentre to the southeast of the city. Houses on concrete floor slabs suffered again from liquefaction and lateral spreading of the foundations, although houses built on suspended timber floors with perimeter concrete foundations generally performed well. Damage that did occur was concentrated on brittle elements such as brick chimneys, veneers and roof tiles, and timber elements such as weatherboards fared particularly well. There was no loss of life in residential buildings, but there were a few building collapses, and the cost of repairs and rebuilding will be a significant drain on New Zealand’s economy for many years to come.

4.4 JUNE 2011
The effects of the major aftershocks in June were similar to those of the February event. In particular, many hillside houses in the Sumner/Redcliffs area were further damaged as this was very close to the epicentre.

4.5 BRANZ DAMAGE SURVEYS
BRANZ undertook a comprehensive survey of the performance of residential buildings after both the September and February events.

After the September event, the BRANZ surveyors accompanied the EQC Insurance Assessors, in an effort to minimise disturbance to the home owners, many of who were badly traumatised by the event. Unfortunately this gave little control over the properties visited and meant that the sample was badly skewed. This survey was almost completed when the 22 February 2011 earthquake struck.

Following the February event, BRANZ undertook another survey, this time of over 300 houses, randomly selected from within the boundaries of Christchurch city. The process involved randomly selecting a little more than 50 mesh blocks from the Statistics New Zealand database. Within each mesh block, six adjacent houses were selected for surveying at the southeast corner of each mesh block. Each property was visited by a team of two BRANZ representatives with a comprehensive survey form to gather observations about the site and its hazards (eg liquefied, rock-fall susceptible), house age, house style, construction materials and then estimates were made of the extent of damage sustained by the various elements of the structure.

Of the 314 fully completed surveys 93 houses were identified as having been built during the period of existence of NZS 3604.

The data collection process was designed to fully describe the property and its construction, and to quantify the damage experienced in the two earthquakes.

4.6 SUMMARY OF FINDINGS
4.6.1 Foundations
The predominant types of foundations used in Christchurch houses were either a suspended timber floor on concrete piles with a concrete perimeter foundation, or in the more modern houses, a concrete floor slab.

The effects of liquefaction and associated lateral spreading on the concrete floor slabs has been well documented. However if there was no ground movement on the property, the slab performed well. Timber suspended floors performed relatively well, even on soils affected by liquefaction. This performance can be attributed to the presence of the concrete perimeter foundation which provided a squat lateral force resisting element. In buildings where ground movement or lateral spreading did occur, at least the piled foundations allowed access beneath for repairs or re-levelling.

Where problems did occur was where the foundation wall supporting the veneer and the piled floor foundation were separate or not well connected together. This detail was unable to resist ground movement, as can be seen in Figure 2.
4.6.2 Walls
Timber framed walls generally performed well where the site was not affected by liquefaction. Timber claddings such as weatherboards were virtually undamaged and because of their elemental nature didn’t show the effects of minor ground distortion. Sheet cladding such as plywood or fibre-cement were also relatively undamaged. They clearly provided a bracing or stiffening function to the building even if not specifically designated as bracing walls.

Plasterboard was the predominant type of wall lining in the houses surveyed, with lath and plaster, and fibrous plaster the next most common systems. 85% of wall linings had some extent of cracking to at least the sheet joints. Diagonal cracking to sheets, usually emanating from internal corners (see Figure 3) was much less common, and complete sheet detachment was rare.

An observation was made by several owners that their houses now seemed more “flexible” as a result of the earthquakes and the numerous aftershocks. This was evidenced by vibrations and creaking during strong winds. BRANZ is currently investigating the causes of this phenomenon and possible remedial measures.

4.6.3 Roofs
Roof structures (either stick framed or nail plate trussed) were rarely damaged, even when the house was severely distorted. The continuity provided by purlins or tile battens and ceiling battens provided a diaphragm function and held the upper part of the house together. This can be seen in the severely deformed house in Figure 4. Valleys were one exception to this, and proved to be a weak point in many houses due to the loss of continuity (see Figure 5). The consequential damage can be seen in Figure 5.

This is a simple detail to rectify and has already received a positive response from practitioners spoken to by the authors.

4.6.4 General
Taking an overview of the performance of light timber framed buildings in the Canterbury earthquakes, while it is true that they performed well in general, there are some lessons to be learnt from the experience. Structures on hillsides, complex shapes, and horizontal and vertical irregularity will always provide challenges for light timber framing systems. Multiple foundation levels can make dynamic behaviour unpredictable, and take the structure outside the simple dynamic models that both AS/NZS 1170.5 and NZS 3604 are based upon. Additionally, complex plans and elevations often result in connection details that are difficult to achieve in the context of light timber framing.
Buildings with large window openings may result in structures with low local stiffness, and there were many examples in the hill suburbs in particular where lack of stiffness resulted in damage to windows and large glazing elements (see Figure 6).

Figure 6. The window was unable to accommodate the in-plane displacement of the timber structure.

One result of the Canterbury earthquake series was that the seismic hazard factor, $z$, for Christchurch has been increased from 0.22 to 0.3 to account for the greater seismicity of the Canterbury region for at least the next few years. Fortuitously, the seismic zone of NZS 3604 that encompasses Christchurch (zone 2) already included a $z$ factor of 0.3 so no alteration was required.

5 CONCLUSIONS

The initial conclusion from observations of the performance of light timber framed buildings in the Canterbury earthquakes was that they performed well if they were within the scope of NZS 3604, and there was no liquefaction at the site. This is in spite of the fact that many experienced levels of ground shaking that were beyond what they would have been designed to resist. However damage was very costly, and thought should be directed towards ways to minimise this for the future. There are already a number of lessons to be learnt from the experience which should improve performance in future events.

The results of two surveys of timber framed building performance are being analysed at the moment, and more detailed findings will be available towards the end of the year.

ACKNOWLEDGEMENTS

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The authors are grateful for assistance in conducting the surveys provided by contractors, and staff and students from Auckland and Canterbury Universities. They would also like to pay a special tribute to the people of Christchurch who willingly opened their homes and allowed surveyors access, at a time of real anguish and upheaval in their own lives.

REFERENCES

3-D MEASURING FOR AN ENGINEER-TO-ORDER SECONDARY WOOD PROCESSING INDUSTRY

Samuel Forsman¹, Lars Laitila², Niclas Björngrim³

ABSTRACT: The construction industry has been criticized for not keeping up with other production industries in terms of cost efficiency, innovation and production methods. Supplying Engineer-to-Order (ETO) joinery products to the construction industry is a novel research area within the secondary wood-processing industry. This involves processing wood into highly refined one-of-a-kind products that are engineered to fit specific needs. Before these products are manufactured, there is a need for the supplier to verify the as-built spatial information of the products adjacent environment. This is due to differences in the tolerances of the production methods in general construction and the ETO joinery products. This verification is not always sufficiently accurate or easy to perform with current methods. Further, suppliers often lack sufficient 3-D information to eliminate spatial uncertainties that affect the level of prefabrication off the construction site. Therefore, adjustments are left to be performed during assembly, and often the time required for assembly on the construction site is equal to the time required for designing and processing the products in the production plant. This work shows that current available 3-D measuring techniques, such as 3-D laser scanning equipment, coordinate measurement machines and photogrammetry techniques, have potential to improve the quality of measurements on the construction site and add spatial information that can be used in the production, planning, and assembly of neatly fitted wooden products.

KEYWORDS: 3-D measuring techniques, ETO joinery production, Supplying to construction

1 INTRODUCTION

One of two models for the distribution of joinery products is the one supplying the construction industry with tailored, one-of-a-kind products that are fitted into a given building object. This value chain is adapted to the culture in the construction industry and is still craftwork intensive and has not been able to fully utilize industrialized processes in terms of cost efficiency, innovation and production methods. Thus, this resembles the situation in the construction industry.

In media as well as in the research community, the current state of construction is under debate. The construction industry has been criticized for not keeping up with other production industries in terms of cost efficiency, innovation and production methods [1, 2]. Innovations that decrease the cost of building production and alterations have gained considerable attention in the research community and media due to their effect on the prices of the living and working environments. Increased industrialization and higher levels of prefabrication are seen as focus areas for innovation in construction. In [3] it is shown that higher predictability in the planning of construction projects is essential in incorporating Lean principles in building construction.

Engineer-to-order (ETO) joinery products are products that are prefabricated in industrialized production plants. For suppliers of joinery products, there is a need to verify spatial as-built information, since general tolerances in construction do not provide precision in parallel with their products. Despite current efforts to verify spatial as-built information, their methods cannot eliminate the spatial uncertainties, and they need to work with methods to handle spatial uncertainty, which decreases efficiency in production as well as in assembly. Thus the level, or degree, of prefabrication is restricted by the presence of spatial uncertainties. Therefore, cost-effective methods and technology for eliminating spatial uncertainty are highly interesting for this type of industry.

Manual measurements are time consuming, and the spatial information obtained from the construction site is
limited. Currently available 3-D digitizing equipment, such as laser scanners, coordinate measurement machines, photogrammetry solutions, and etcetera, could be used to gather more of the spatial information (in 3-D) from the joinery products’ adjacent environment. This information would then provide a basis for producing neatly fitting ETO joinery products.

Building Information Models (BIM) is today an accepted and widely used method for storing and sharing knowledge of a facility [4]. However, the information in the models, which often are based on CAD models, does not normally provide the supplier with as-built information; rather they show the as-planned information, and in alteration projects, this information can be based on old as-planned information. The joinery-product suppliers need as-built information as opposed to the as-planned information of BIMs.

This paper will address two research objectives: with 3-D measuring techniques such as laser scanning, coordinate measuring machines and photogrammetry to reduce the spatial as-built building uncertainties and increase the level of prefabrication and quality of the products; provide joineries with enough information to plan, produce and assemble the products efficiently.

With this background, the purpose of this paper is to evaluate laser scanner, coordinate measurement machine, and photogrammetry technology in terms of retrieving spatial 3-D as-built information in a “real world case” [5] of supplying ETO joinery products to construction. Here the objective is to identify best performing technology for this purpose and to identify areas for further studies.

2 Method

The supplying of ETO joinery products has been followed in a case study. Current routines and practices have been studied, and in parallel the use of 3-D measuring technology has been evaluated in the same environment as the real joinery-product supply process has been working. The surveyors’ work on the site was documented by notes, photos, through asking questions about the work being performed and by studying the documentation they created during their work.

In parallel with the manual measurements performed by the joinery-product supplier, three different 3-D measuring technologies were used to capture the joinery-products’ adjacent environment at the construction site. The three products used were 1) a Proliner 8 coordinate measuring machine (CMM) from Prodim⁴, 2) a photogrammetry setup with a Nikon D50 DSLR camera and Photosynth software⁵ and 3) a Leica Scan Station C10 laser scanning apparatus⁶. The laser scan was carried out by the company Mättjänst AB⁷, and the CMM measurements and single camera photogrammetry were carried out by the authors.

The Swedish construction project was an alteration project in which an entire floor in an office building in Solna, Stockholm, was altered to fit a new tenant. At the entrance of that floor, a reception area, a visitor zone and a cloakroom were designed by the Irish architects, and the Swedish joinery-products supplier provided the interior products for the project as well as the assembly of them on the construction site. In Figure 1, the ichnography of this area this is presented. The area within the red line represents the area that was surveyed.

Figure 1: Construction area under survey

2.1 PROLINER 8 CMM DATA CAPTURING

The Proliner 8 CMM consists of a main unit and a measuring stylus probe connected by a 7.5 meter long steel wire. The Proliner 8 CMM measurements were done using two different strategies (Middle and upper part in Figure 5). The first one is similar to manual measuring where the stylus probe of the Proliner 8 acquired coordinate registrations in the beginning and at the end of each straight line to represent the location of each wall segments. This type of measurement is easy to perform and gives a representation of the ichnography. However, it gives no information about the vertical alignment of the walls or if there is some curvature in the wall surface. On curved walls, the stylus probe was swept along the wall and the Proliner 8 made continuous coordinate registrations.

A second Proliner 8 CMM measurement was performed using the second strategy for acquiring the as-built information. Here the Proliner 8 stylus probe was swept all over each wall segment with continuous registration of the coordinate positions in order to have a representation of the wall plane. This second measuring strategy also gives information about the walls’ vertical alignment.

To cover all required surfaces, the limitations in the range of the Proliner 8 required moving the CMM and reconnecting to the previous measurements. This reconnection is a function of the Proliner 8 that is called leap—four markers are measured in the first position of the Proliner 8 and then again from the new position after relocation so as to reconnect to the initial measurement.

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⁴ http://prodim.eu/
⁵ http://photosynth.net/
⁶ http://hds.leica-geosystems.com/en/Leica-ScanStation-C10_79411.htm
⁷ http://www.mattjanst.se/
The data captured with the Proliner 8 was exported in DXF format to Solid Works and Siemens NX CAD software. A 3-D model was created to represent the dimensions captured in the manual measurement performed by the joinery-products supplier (Figure 5). To create 3-D geometries, walls/planes were extruded from the lines.

2.2 SINGLE CAMERA PHOTOGRAMMETRY

Photographs were taken with a Nikon D50 DSLR camera from the middle and at the corners of the room. Each picture was taken so that it overlapped the previous one by approximately 50%. Close-up pictures of the walls were also taken. To cover the area of interest, the environment was captured with a total of 113 photos. The photos were uploaded to a photosynth.net webpage, resulting in both a 3-D panorama and a point cloud. The Photosynth software does not require targets or a calibrated camera.

2.3 LASER SCANNING

The laser scanning with the Leica Scan Station C10 was performed by the company Mätjänst AB. The survey was performed from four positions in the reception, visitors’ zone and the cloakroom and then merged into a single point cloud of measured coordinates (Figure 8). Three circular targets were placed in line of sight from all scan positions. The targets were used for merging and aligning the subscans.

After performing the scanning, the measuring company supplied the authors with the raw data of the coordinate point cloud and contracted a third party, Astacus, to create 3-D CAD models that were later delivered to the authors. The authors also processed the raw data into useful information that was processed into measurable models to be compared with the models from the measuring company.

3 RESULTS AND DISCUSSION

In construction, there seem to be no customary practices to verify that the building really reflects the prescribing documents according to given dimensions in construction drawings. For this reason, there is a need for the joinery-product supplier to verify that the ordered joinery products adjacent environment in the prescribing documents reflects the as-built reality before producing the joinery products.

3.1 MANUAL MEASURING

Currently, the ETO joinery-products supplier uses manual measuring techniques in which folding rulers, tape measures and laser distance meters are used to obtain the as-built information (Figure 2).

Figure 2: Manual measuring

From the architectural drawings, important measurements were predetermined by the production preprocessor before surveying. From the predetermined list of measurements, the surveyor worked to capture wall placements, diagonals in the rooms and doorways. Measurements were noted on printouts of the architect drawings (Figure 3). The measurement were done at floor level, which means that the data is limited to a 2D representation of the site with no concern for angle between walls and floor, waviness and slope of walls and floors, et cetera. Some of the results of the manual measuring can be seen in Figure 3, where the dimensions 6900, 9190, 9400 mm, and an arc length of 2950 (mm) can be read out and compared with the results from the other measuring techniques.

Figure 3: Notes from manual measuring

These methods can be seen to have a number of error sources, and their accuracy can be questioned. For example errors can be introduced when trying to measure the center position of a pillar (left picture in Figure 2) due to the difficulty of positioning tape measures or rulers correctly, or by rounding off when reading the tape measure.

To perform the measurements, personnel from the supplier needed to travel more than 800 km to perform measurements for half a day. Still this work was not fully coordinated with the construction project, and there were walls that had not yet been built that would need to be measured. Therefore, not all the necessary spatial information could be retrieved. Further, the producer had to do complementary measurements later when it was found that some important dimensions had not been defined.
The notes from the measurements were done on paper printouts of the architectural drawings that were physically transported to the production preprocessor. The joinery-products supplier in this study spends about 1700–2000 hours annually on geometrical measurements on the adjacent environments for their products, which is equivalent to one full time employee specialized in performing spatial measurements, but this is not how they have chosen to work. Their measurements are mainly performed on a 2-D basis, thus leaving out much of the 3-D information that could be useful when modeling during the production preprocessing. This has consequences for assembly, since the components of the ETO joinery products need on-site adjustments to fit the products’ adjacent environment. The assembly work becomes more unpredictable in time and resource need due to spatial uncertainties. Therefore the level of prefabrication and predictability could be enhanced by eliminating spatial uncertainties.

### 3.2 PROLINER CMM

The measurements with the Proliner 8 CMM were performed in one hour for each of the two measuring strategies described in the method. In Figure 4, the Proliner 8 CMM measurements of the walls from the reception, visitor zone, and cloakroom are shown. To the left in the figure the method of sweeping the stylus probe over the wall surfaces is visible, and on the right the method of using only few coordinates to represent the wall position except on curved walls where the stylus probe is swept along the wall’s length. These lines and curves were the basis for the creation of a 3-D geometry CAD model that was produced in about one hour. Thus in two hours a 3-D CAD model could be produced and delivered electronically to the joinery-products supplier.

![Figure 4: Measuring information from Proliner CMM](image)

In the modeling of the Proliner Data, a DXF (Drawing Exchange Format) file was exported from the Proliner to Siemens NX, and 3-D solids were created. The process is illustrated in Figure 5. In the upper part of the figure, each plane in the DXF file represented a wall plane. Further, a combination of information about construction method and measurement information was provided to create the 3-D model representing the wall that at the time of measurement had not yet been built. In the middle part of Figure 5, the cloakroom there is a curved wall that has several different radii, and here the wall extrusion is made by several connected lines. In the lower part of Figure 5, the two separate measurements are put together in an assembly to illustrate how the two are aligned to each other.

![Figure 5: Model creation from Proliner data](image)

### 3.3 SINGLE-CAMERA PHOTOGRAMMETRY

The 113 photographs of the construction site were captured during approximately 30 minutes. Uploading and creating the Photosynth model took about 20 minutes with a wireless network with an 80-Mbit/s capacity. In the Photosynth software, the pictures were stitched together to form a 3-D panorama (Figure 6) and a point cloud (Figure 7).

![Figure 6: 3-D panorama](image)

![Figure 7: Point cloud](image)
However the point cloud from the view in Figure 6 and shown in Figure 7 was from the highest concentration of captured coordinates and not representative for the entire area being surveyed. For most of that area, the Photosynth-generated point cloud made from these photographs was not adequate to make any representative 3-D CAD models of the surveyed area.

### 3.4 LASER SCANNING

The scanning of the part of the construction site of interest was performed in approximately one hour in a mode with high density of scan coordinates. The resulting point cloud gave approximately 45 million coordinates. To increase visibility, data reduction of the point cloud was performed; the result is shown in Figure 8. The scanning is easy to perform and results in a large amount of information. However, to create representative 3-D CAD models from this information requires appropriate software tools, skill and time.

The laser scan produces a good deal of information that is not needed; e.g., there are free standing objects, such as toolboxes and other equipment pertaining to the construction work, that are not part of the room of interest (Figure 9). Therefore, the relevant information needs to be selected when processing the data, making experience of on-site measurement in combination with understanding of the planned products highly useful.

The models produced by the Astacus Company were supplied in three different formats: STL format (Standard Tessellation Language), the WRL or VRML (Virtual Reality Modeling Language) and in DWG (drawing) format (Figure 10). These three formats hold different types of information that can be used in the production preprocessing. Among Swedish ETO joinery-products suppliers, the DWG is the format most commonly used.

To compare the different measuring techniques against each other, four dimensions (A, B, C, and D) that were seen as important by the production preprocessor are retrieved from the manual measurement, the Proliner measurement and the laser scanning (Figure 11). From the laser scanning, measurements are taken from the...
three models that where supplied by the Astacus company.

Figure 11: Defined dimension in Cloakroom

In Table 1, the resulting dimensions from Figure 11 are presented for the measuring techniques tested in this paper. Note that from the photogrammetry set up no measurements could be taken. From the laser scanning, measurements were done in the three CAD models. From Table 2 one can see that there are some differences despite the fact that they came from the same raw data. Both the laser scan and the Proliner CMM show differences in the dimensions compared to the manual measurement as well as compared to each other.

Table 1: Measures of defined dimensions

<table>
<thead>
<tr>
<th>Measuring Method</th>
<th>A (mm)</th>
<th>B (mm)</th>
<th>C (mm)</th>
<th>D (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manual</td>
<td>9400</td>
<td>9190</td>
<td>6900</td>
<td>2950</td>
</tr>
<tr>
<td>STL (Scan)</td>
<td>9412.3</td>
<td>9211.3</td>
<td>6896.7</td>
<td>2923.1</td>
</tr>
<tr>
<td>WRL (Scan)</td>
<td>9413.2</td>
<td>9212.3</td>
<td>6897.1</td>
<td>2923.5</td>
</tr>
<tr>
<td>DWG (Scan)</td>
<td>9412.4</td>
<td>9211.4</td>
<td>6896.7</td>
<td>2923.2</td>
</tr>
<tr>
<td>PRT (CMM)</td>
<td>9431.5</td>
<td>9224.8</td>
<td>6897.2</td>
<td>2931.9</td>
</tr>
</tbody>
</table>

Table 2: Measure differences of defined dimensions

<table>
<thead>
<tr>
<th>Method Comparison</th>
<th>A (mm)</th>
<th>B (mm)</th>
<th>C (mm)</th>
<th>D (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manual vs. Scan Average</td>
<td>-12.63</td>
<td>-21.7</td>
<td>3.17</td>
<td>26.73</td>
</tr>
<tr>
<td>Manual vs. CMM</td>
<td>-31.5</td>
<td>-34.8</td>
<td>2.8</td>
<td>18.10</td>
</tr>
<tr>
<td>Scan (Max-Min)</td>
<td>0.9</td>
<td>1.0</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Scan vs. CMM</td>
<td>-18.7</td>
<td>-13.13</td>
<td>-0.37</td>
<td>-8.63</td>
</tr>
</tbody>
</table>

4 DISCUSSION

Currently, ETO joinery-products suppliers need to perform spatial measurements on the construction site before starting the production of the ordered products. These measurements are used to calibrate the architectural drawings against the as-built reality. In the studied case, this was done in a 2-D sense rather than a true 3-D representation of the construction scene, thus leaving a high degree of uncertainty that limits the level of prefabrication of the joinery products. Further, manual measuring shows high risk for mistakes and that the captured data might not be accurate; in this case, complementary measurement was required. These are problems that also are discussed in other research; e.g., [6] and [7].

The three 3-D measuring techniques used in this work have somewhat different conditions for how to use the information and on the limitations in precision. The Proliner CMM show rather high accuracy in each measured coordinate, but the number of coordinates usually retrieved is limited, which restricts the amount of information it can provide. Further, the Proliner CMM is wire bound, which causes problems with obstacles. It is also limited in range; therefore there is a need use a leap functionality to connect measures when moving the device, and this affects accuracy.

The photogrammetry method has the strength of being useable at limited cost by using an SLR camera on a tripod. Many of the surfaces at the construction site were gypsum boards that had few or no gradients in color or textures, and windows where the exterior surfaces of the windows also gave measurement coordinates. Therefore, the number of trustworthy coordinates in the point cloud calculated by Photosynth was limited. Still the idea of using photogrammetry seems interesting if it’s possible to achieve more measuring coordinates.

The laser scanning with the Leica C10 performed by “Mättjäst AB” was the technique that gave the greatest amount of information. The supplier of the measuring service provided the researchers with the point-cloud data (Figure 8). The process of transforming the scan information into 3-D CAD models is a service that seems limited in Sweden. The Astacus company used staff in India to make the point-cloud–to–CAD transformation. What can be seen in the provided 3-D CAD model is that it seems to be simplified; for example, information about wall–floor angles other than 90° is missing. Further, measured objects such as a sheet metal sleeper on the floor whose edges are about 1 mm are represented as 5-mm-thick material in the provided 3-D CAD model from the laser scanning.

There are dimensional differences between the manual measurements and the models created from the Proliner 8 CMM and the Leica ScanStation C10 laser scanner. There are several reasons for the dimensional differences between models in Table 1 and Table 2. The equipment used to capture the environment, modeling software, methodological approach, et cetera, affect the result of the CAD model. In the modeling of the laser-scan data, WRL, STL and DWG models are created through best-fit method from the coordinates in the point cloud. The best-fit method approximates planes to the point cloud and does not follow the topography of the geometry very well. The point cloud consists of four scans that were merged into one, which introduces accuracy errors in the point cloud.

Similarly, the data from the Proliner 8 CMM approximate curves, lines or even planes through best-fit methods. Even if the accuracy in each coordinate is high,
it’s not certain that the position of the stylus probe is representative for the curve, line or plane that is being measured, and the limited number of coordinates being registered affects the accuracy in the models from the Proliner 8 CMM. For example, a “straight” and “flat” wall at a construction site often have some horizontal and vertical curvature, and the plane often deviates from being truly vertical. This is a wall that is difficult to measure with the Proliner 8 CMM to create a representative model.

Assuming that laser scanning provides the best accuracy of the tested methods in this paper, the comparison between the manual measurement and laser scanning shows differences of 27 mm. These errors are quite worrying when the joinery-product supplier works with tolerances of less than 1 millimeter.

5 CONCLUSIONS

The three evaluated 3-D measuring techniques show that they all have potential to enhance the process of acquiring as-built information by comparison with currently used methods as regards the amount of information and 3-D relations.

In the work presented, it is shown that the traditional methods of measuring as-built dimensions differ quite considerably from CMM and laser-scan data. Spatial deviations of walls and floors are not considered at all with the methods used today. By reducing the spatial uncertainties for the ETO product, the need to perform on-site adjustments will decrease, leading to more predictable assembly work and a reduction in time and cost for assembly.

All three methods tested provide simplifications in the transformation to a 3-D model, but the reliability of the virtual reality based on the measurements leaves something to be desired. To achieve an accuracy of 1 mm in all aspects still seems difficult using the technology tested in this paper.

ACKNOWLEDGEMENT

The research work was carried out thanks to funding by VINNOVA and European Union Objective 2, which is very much appreciated. Further the ETO joinery-product suppliers studied are appreciated for giving access to the real world case and Mättjänst AB for performing the laser scanning and providing us with scan data.

REFERENCES

FEATURE RECOGNITION AND FINGERPRINT SENSING FOR GUIDING A WOOD PATCHING ROBOT

Tobias Pahlberg¹, Olle Hagman²

ABSTRACT: This paper includes a summary of a few commonly used object recognition techniques, as well as a sensitivity analysis of two feature point recognition methods. The robustness was analyzed by automatically trying to recognize 886 images of pine floorboards after applying different levels of distortions. Recognition was also tested on a subset of 5% of the boards which were both re-scanned using a line scan camera and photographed using a digital camera. Experiments showed that both the Block matching method and the SURF method are valid options for recognizing wood products covered with distinct features. The Block matching method outperformed the SURF method for small geometric distortions and moderate radiometric distortions. The SURF method, in its turn, performed better compared to the other method when faced with low resolution digital images.

KEYWORDS: Wood, Feature recognition, Fingerprint, Hol-i-Wood PR, Patching robot, Image analysis, Classification, Holonic, Sensor fusion

1 INTRODUCTION

Suppose a wooden board has gotten a crack, a few dead knots have loosened, and maybe the hue of a part of the board has changed. The question is if it is possible for a machine vision system to identify the board using only its biometric “fingerprint”? We may want to trace back through the history of the board. How was it produced and under what conditions? Or perhaps the task is merely to detect if the right piece has arrived at the right machine?

The objective of the presented work is to determine if it might be possible find a certain wooden board in a database of a thousand boards; and to determine a feasible type of descriptor that a computer can use to recognize boards. The descriptors need to be robust to both radiometric and geometrical distortions to be able to recognize a particular wood piece using different cameras and different camera setups.

The Hol-i-Wood Patching Robot project started in early 2012 and is a collaboration between Luleå University of Technology (LTU), TU Wien and TU München, as well as industrial partners; MiCROTEC, Springer, TTTech and LIP BLED. The project will have resulted after three years in several different standalone holonic modules of which the wood fingerprint recognition is one part.

The hardware will consist of a new multi-sensor scanner, a system for transport of laminated wood products which need repairing, and lastly a patching robot. The patching robot will be equipped with a vision system and should be able to recognize previously scanned wood pieces at the repair station, preferably using a non-invasive method. The patching robot will drill out certain defects and replace them with fitted wooden dowels. Defects such as dead knots, pitch pockets and geometrical defects. The identification using the fingerprint of wood is needed to inform the patching robot of which piece has arrived.

A number of laminated wooden end-products nowadays depend on a skilled machinist to repair the defects off-line, which can become a bottleneck in the production chain.

2 THEORY

The purpose of this chapter is to summarize and describe a few important concepts related to identification systems.

2.1 FINGERPRINTS

Within the field of biometrics the human fingerprint has been used for identifying people for over a century [1]. Scanning technology and algorithms have been pushed forward mainly by security and forensics applications. The procedures in fingerprint recognition software most
often have to address the following four design challenges [2]:

1. Image acquisition
2. Fingerprint representation
3. Feature extraction
4. Matching

Human fingerprint systems often rely on extracting certain minutiae or details for identifying individuals [1]. Ridge endings, ridge bifurcations and singular points are usual descriptors together with the coordinates and the orientation of these descriptors. Similar techniques can likewise be applied to identify wooden boards, where for example their knots and growth ring patterns describe “individuals” in the same way as the ridges do on a finger.

2.2 ROBUSTNESS

It can be very difficult to acquire an image under exactly the same conditions at different scanning stations. Furthermore, a lot of things can have happened to an object over time since a previous scan. Hence, a practical identification system needs to be robust to noise and distortions.

Geometric distortions can arise if for example a wooden board is transported skew past a line scan camera, if the pixels are non-square, or if the cameras have different poses at different scanning stations. Moreover, radiometric distortions introduced by different lighting conditions could be present, and at some manufacturers, for example, sawdust or dirt can occlude parts of the boards.

2.3 SPEED

A common way to reduce the problem of quickly having to search through a huge database of fingerprints to find a match is to group similar prints into bins [1]. These bins can be used for classification purposes if desired, but they can also be used to reduce the search space in a matching stage.

Bin sorting methods can on the other hand lead to problems if noise is present, where some features can become ambiguous. In such a case the sought board might have been put into the wrong bin and is therefore impossible to find.

Instead of sorting boards into discrete classes, a feature vector can be used to represent it. The Euclidean distance can then be calculated between the query vector and vectors in the database.

2.4 CLASSIFIERS

Neural networks (NN) are widely used as a classification method for fingerprints. A fingerprint system called PCASYS which utilizes NNs was developed for the FBI in the early 1990s [3].

Feature extraction and classification of wood defects was done using NNs in [4, 5]. The learning problem in those cases was solved by the use of self-organizing maps (SOM) in the early feature extraction stages. A correct classification rate of 86%, for example, was obtained using Gabor filters and adaptive color histograms.

A tree-structure support vector machine (SVM) approach was proposed in [6] to classify four types of wood knots, achieving an average correct classification rate of 96.5%. In a related thesis work a supervised classifier was trained with Adaboost to classify different stain type defects on wood in [7].

Different classifiers have different strengths and weaknesses; hence a combination of classifiers is often the best solution.

2.5 OBJECT RECOGNITION

Object detection, classification and recognition are tightly intertwined areas. The sought output is different for them, but the process of getting the output is usually similar. Often the procedure involves extraction of feature points from the objects.

A feature point is a region in an image that is likely to be recognized in other images [8]. Typical feature points include, for example, corners, blobs and T-junctions. Corners, i.e., positions in an image where there is a strong intensity change in two orthogonal directions, are very good objects to track [9, 10].

A region around the corner is usually stored. These regions, or templates, are later used for recognition of objects by finding several matching regions between images.

Criteria for good feature points, or interest points, are described in [11]:

1. Distinctness: The points should be distinguishable from their neighborhood, e.g., consist of a pronounced gradient in intensity or color.
2. Invariance: The points should be invariant with respect to expected geometric and radiometric distortions.
3. Stability: The points should be robust to noise.
4. Seldomness: There should not be several similar points in the same image to avoid confusion. (If a point is part of a repetitive pattern, the possibility for a false match is high.)
5. Interpretability: The points should preferably be interpretable, such as an edge, corner or blob.

Edges are usually not good interest points; the region information is similar along the line and hence does not fulfill the seldomness requirement. Smooth untextured regions do not uphold the distinctness requirement, etcetera.

Several corner- and feature point-detection algorithms have been produced over the years. One of the most famous corner detectors is the Harris corner detector [12]. The Harris method involves moving a search window over a grayscale image and computing the weighted Sum of Squared Difference (SSD) between the starting location and its surroundings (Figure 1). In a smooth untextured part of the image there will not be a big SSD response in any direction. At a line, or a sharp gradient change, there will be a big response in
one direction. At a corner there will be a big response in two different directions.
If a grayscale image is denoted $I$, the search window area $(u,v)$, and the shift $(x, y)$, then the weighted SSD is given by:

$$D(x, y) = \sum_{u,v} w(u,v) (I(u + x, v + y) - I(u, v))^2.$$  \hspace{1cm} (1)

The weighting factors, $w(u,v)$, are often formed to create a circular search window. Moreover, a Gaussian smoothing can be applied over the region to counter the noise introduced due to the discrete grid of pixels in an image.

By use of Taylor expansion, Equation (1) can approximately be rewritten as:

$$D(x, y) \approx (x - y) A \begin{pmatrix} x \\ y \end{pmatrix},$$ \hspace{1cm} (2)

where

$$A = \sum_{u,v} w(u,v) \begin{bmatrix} \frac{\partial^2 I(u,v)}{\partial x^2} & \frac{\partial^2 I(u,v)}{\partial x \partial y} \\ \frac{\partial^2 I(u,v)}{\partial y \partial x} & \frac{\partial^2 I(u,v)}{\partial y^2} \end{bmatrix}. \hspace{1cm} (3)$$

Second derivatives are better suited for finding lines and corners than first derivatives, i.e., gradients. Second derivatives are only non-zero at places in the image where there is a gradient change, not where there is a constant gradient. A constant gradient is usually not defined as a line.

Most corner detectors utilize the Hessian matrix, in some way, to find intensity maxima.

Through the eigenvalues, $\lambda_1$ and $\lambda_2$, of $A$, a rotationally invariant descriptor of what is going on around a point in an image is obtained.

1. If both $\lambda_1$ and $\lambda_2$ are large, a corner is present in the region.
2. If one eigenvalue is large and the other is small, then a line is present in the direction perpendicular to the eigenvector with large eigenvalue.
3. If both $\lambda_1$ and $\lambda_2$ are small, there is no a distinct edge or corner in that region.

The eigenvalues are usually not computed explicitly due to the computational cost. Instead the trace and determinant of $A$ can be used.

In the method proposed by Shi and Tomasi, a threshold is set on the smaller of the two eigenvalues which decides if a corner is accepted [13].

Another corner detector, the Features from Accelerated Segment Test (FAST, [14]), uses a different approach. The algorithm searches for patches within an image that “looks” like a corner. Hence, it does not have to calculate second derivatives or eigenvalues. As can be understood by its abbreviation, it is fast, and trades exactness for speed. Only four pixels are traversed in each patch when deciding the probability of there being a corner in it. Only in the most probable patches, a more thorough investigation is made by traversing more pixels around the circumference of the patch. The FAST corner detector is often accurate enough, as long as there is not too much noise in the image.

Lowe proposed a novel approach for feature point detection called Scale-Invariant Feature Transform (SIFT, [15]). SIFT detects the dominant gradient orientation of interest points in an image and saves the gradient information around these points. Since the dominant orientation is found, the method becomes rotationally invariant. SIFT features can also be matched at different scales since the images are downsampled iteratively while leaving the kernel size unchanged. Scales are divided into so-called octaves, where the next octave corresponds to a doubling of the length of the side of the image (Figure 2). Moreover, SIFT and its variants can handle some variation in illumination of the scene.

A fast and robust method which is in many ways similar to SIFT is the Speeded-Up Robust Features method (SURF, [16]). SURF uses integral images [17] for fast calculations of sums of intensities over rectangular regions in grayscale images. The use of integral images makes it possible to do such computations in constant time, independent of region size. In the SURF approach, the kernel sizes are resized instead of the images, as in SIFT. A Hessian matrix-based method is used for finding corners and blobs suitable as feature points.

Intensity information on feature points is stored in a way similar to SIFT. However, SURF is by default using half as much storage space, i.e., a 64-dimensional vector instead of 128. This speeds up both the feature extraction and the matching time.

For images without distinct features, a template-based approach may be more suitable. Template matching works by moving a search window over an image while calculating an error metric, e.g., SSD or normalized cross correlation between the template and search image.
The minimum error, or maximum response, is given when the template is at the correct location.

### 2.6 GEOMETRIC TRANSFORMATIONS

The geometric relation between points in space and a two-dimensional image can be described by a projective transformation [18]. If most of the points in a scene are at the same depth, and if a long focal length is used, then the affine transformation is a good approximation of the projective transformation. Affinely distorted images can be generated using the planar affine transformation. This 2D to 2D transformation skews the image and then rotates and translates it (Figure 3). The transformation preserves straight lines, i.e., lines which were parallel, will be parallel also after the transformation. Furthermore, affine transformations preserve the length ratios between parallel line segments in an image, as well as ratios between areas.

The planar affine transformation \( x' = Hx \) is defined as:

\[
\begin{bmatrix}
x' \\
y' \\
1
\end{bmatrix} =
\begin{bmatrix}
a_{11} & a_{12} & t_x \\
a_{21} & a_{22} & t_y \\
0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
x \\
y \\
1
\end{bmatrix} =
\begin{bmatrix}
A \\
t
0
\end{bmatrix}
\begin{bmatrix}
x \\
y
\end{bmatrix}.
\]

(4)

The linear transformation, \( A \) in Equation 4, consists of a deformation followed by a rotation:

\[
A = R(\theta)R(-\phi)DR(\phi).
\]

(5)

After the first rotation, \( \phi \), the image is scaled in \( x \)- and \( y \)-directions using the diagonal matrix in Equation 6, skewing the image.

\[
D = \begin{bmatrix}
\lambda_1 & 0 \\
0 & \lambda_2
\end{bmatrix}
\]

(6)

The image is then rotated back by the same amount and a different rotation, \( \theta \), is applied. Lastly, a translation by \((t_x, t_y)\) is applied to the image.

### 3 MATERIALS AND METHODS

#### 3.1 IMAGES AND IMAGE ACQUISITION

The chosen dataset consists of 886 floorboards from Scots pine (Pinus sylvestris) with the dimension 21x137 mm. The same set of floorboards was previously used in [19-21]. They are between 3007-5109 mm long, with a mean length of approximately 4500 mm. The boards were sawn from 222 logs with top diameter between 201-215 mm and were randomly collected from Bollsta sawmill in central Sweden. The boards had been planed, sanded and finished with white pigmented oil and a thin layer of varnish.

The finished floorboards were scanned in 2006 using a high resolution color line scan camera; Dalsa Trillium TR-37. It is a 3CCD camera with a 2048 pixel array for each of the colors; red, green and blue [19]. The boards were scanned at a resolution of 2.5 pixels/mm lengthwise and 10 pixels/mm across. Photocells were set up to automatically crop off 80 mm from each side to avoid edge artifacts. The images were then resized to a resolution of 1 pixel/mm in both directions using bicubic interpolation and were saved in JPEG format at 95% quality. The colors of the images had been carefully calibrated to match the colors of the boards in reality.

#### 3.2 ROBUSTNESS TESTING

To test the robustness of the two different feature-matching techniques, a two-level full factorial experiment was created. A subset of 100 floorboards was picked out randomly from the total 886. The planar affine transformation was used to distort these images using different parameter combinations. The four parameters; \( \theta, \phi, \lambda_1 \), and \( \lambda_2 \), were each assigned a “low” and a “high” value. Hence, \( 2^4 = 16 \) new image datasets containing 100 images each were created to test all combinations and to determine which factors had the most significant impact on the identification accuracy. The different low and high values can be seen in Table 1.

#### Table 1: The affine distortion parameters’ “low” and “high” value used in the robustness testing.

<table>
<thead>
<tr>
<th>( \theta (\circ) )</th>
<th>( \phi (\circ) )</th>
<th>( \lambda_1 )</th>
<th>( \lambda_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>5</td>
<td>5</td>
<td>0.9</td>
</tr>
<tr>
<td>High</td>
<td>15</td>
<td>15</td>
<td>0.7</td>
</tr>
</tbody>
</table>

The translation parameters of the affine transformation were set to zero since they would not affect the result in any case. Feature points are matched independently of their positions and only their relative positions in each image are used to sort out false matches.

The low and high values were chosen accordingly after screening experiments indicated difficulties within these ranges. Note that when the scaling parameters, \( \lambda_1 = \lambda_2 = 1 \), there is no scaling done. As can be seen in Table 1, a 10% size reduction is chosen as the low value, and a 30% size reduction as the high value.

Anti-aliasing is used to limit the impact of aliasing on the output images when resizing and transforming images. Aliasing may appear as “stair-step” patterns at lines in an image or as moiré patterns, a ripple- or wave-type artifact.

#### 3.3 RE-SCAN OF IMAGES

The same line scan camera as before was used in 2012 to re-scan 5% of the floorboards to have a subset of realistically acquired images to test the matching capabilities on. Some of the boards had during the past
years of storage become somewhat crooked or bowed. Moreover, no stabilization of the boards was carried out while the conveyor belt was transporting them past the camera at the re-scan. No color corrections were made on the images and some dust stains were present on a few of the floorboards.

This time the camera was mounted at such a distance from the conveyor belt that yielded a resolution of 5.6 pixels/mm lengthwise and 6.5 pixels/mm across. The entire lengths of the board images were kept this time, and the images were resized to 1 pixel/mm in both directions using bicubic interpolation. They were saved in JPEG format at maximum quality.

### 3.4 DIGITAL PHOTOS

To provide a last tough challenge to the recognition methods, digital photos were taken of the floorboards from a distance. The photos were acquired with a Canon 400D camera at a resolution 0.6 pixels/mm. The images were scaled up to 1 pixel/mm in order not to disadvantage the scale-dependent Block matching method.

### 3.5 FEATURE EXTRACTION AND MATCHING

Two different methods were used for feature extraction and matching in the experiments. The first is referred to as the Block matching method and the other as the SURF method. Implementations of these methods can be found as open source [22], and are often used for similar purposes in other computer vision applications.

In this paper, implementations from MATLAB® R2011b, Computer Vision System Toolbox™, have been incorporated.

#### 3.5.1 Block matching method

In the Block matching method simple square neighborhoods are extracted as features around interesting points in an image. A detailed explanation is presented in Algorithm 1.

A few settings for the FAST corner detector were altered from the default. The size of the search window for finding corners was set to 25x25 pixels, i.e., 25x25 mm due to the resolution. The corner threshold parameter was increased to 0.001 (default: 0.0005) to pass along also weaker corners. However, the intensity threshold for accepting corners was instead increased to 20 (default: 0.1) out of the maximum 127.5, to reduce the number of corners being falsely detected on the boards’ boundaries. No more than the 50 best corners were kept to be able to execute the matching step within reasonable time.

The intensity information in a 25x25 pixel neighborhood around each FAST point was extracted as feature points. This neighborhood size was chosen since it encircled most of the knots. The MATLAB-function matchFeatures carries out steps

---

**Algorithm 1:** Block matching method

1. Run FAST corner detector on query image, \( I_q \), and database image, \( I_b \).
2. **Description of feature points:**
   1. Extract the intensity information in a square patch around the corners.
   2. Patches that are on the border or completely outside the region of interest are removed.
   3. Reshape the patches into \( n_A \) and \( n_B \) column vectors and insert them as the columns of feature matrices \( A \) and \( B \).
   4. Normalize the columns of \( A \) and \( B \) to make the method more robust to radiometric differences.
3. **Matching:**
   1. Calculate the \( (n_A \times n_B) \) error matrix between all the features in \( A \) and \( B \) using SSD according to:
      \[
      E(i,j) = \sum \left( (a_i - b_j)^2 \right), \text{ where } a_i \text{ and } b_j \text{ are the column vectors of } A \text{ and } B \text{ respectively, and the operator } (.^2) \text{ squares every element in a vector.}
      \]
   2. For every row in \( E \), find the lowest column error, i.e., the best matching features between \( I_q \) and \( I_b \).
   3. Remove duplicate feature pairs.
   4. Remove weak matches which have an error above some specified threshold.

---

**Algorithm 2:** SURF method

1. Create a scale-space representation of the query image and the database image by use of integral images and box filters.
2. **Detection of feature points:**
   1. Use the local maxima of the Hessian determinant operator applied to the scale-space to obtain feature points.
   2. Keep points with a response level above a certain threshold.
   3. Refine scale and location of these candidates.
   4. SURF points which are on the border or completely outside the region of interest are removed.
3. **Description of feature points:**
   1. Calculate a dominant orientation direction using the neighborhood gradient information to make the feature points rotationally invariant.
   2. Build a 64-dimensional descriptor corresponding to the local normalized gradient histograms using Haar wavelets.
4. **Matching:**
   1. Compute the Euclidean distance between all potential matching pairs.
   2. Reduce mismatches using a nearest neighbor distance ratio criterion.
3.1-3.4 in the Block matching algorithm. There, the SSD boundary for accepting matches was increased to 2% instead of the default, 1%, to produce more potential matches. Feature pairs with a matching error above this percentage are removed.

3.5.2 SURF method
The SURF method is thoroughly described in [16], and an extensive algorithm description can be found in [23]. A shortened version is presented in Algorithm 2. The number of octaves was kept at the default, 3, but the number of scale levels within each octave was increased to 6 (default: 4). No maximum number of SURF points was specified here, only the default metric threshold for accepting points was used. The SSD boundary for accepting matches was increased to 5% instead of the default, 1%.
The MATLAB-function matchFeatures carries out steps 4.1-4.2 in the SURF algorithm.

3.6 REMOVAL OF FALSE MATCHES
Some features of wood, such as knots, can look quite similar, especially to a computer. Hence, features from one floorboard are sometimes matched to several features in another. This ambiguity can be taken care of by removing features that get a lot of matches (Seldomness requirement, Section 2.5). Features with more than three possible candidate matches were removed. A problem can also be that the best fit of features, algebraically, might not be the correct one, for different reasons. Hence, it is usually a bad idea to sort out "erroneous" matches solely on the basis of a SSD threshold.

3.6.1 Geometric Transform Estimator
MATLAB’s Geometric Transform Estimator was utilized to remove point pairs that should be considered as false matches given a certain geometric transformation. The non-reflective similarity transformation was chosen instead of the affine transformation since it, due to fewer degrees of freedom, performed more stably. To allow for some deviation and to loosen the demands on the point pairs to conform to the model, the Euclidean pixel distance threshold was raised to 100 mm. Random Sample Consensus, RANSAC [24], was chosen as the method for determining inliers, i.e., true matches. A description of the geometric transform estimator is presented in Algorithm 3.

4 RESULTS AND DISCUSSION
In this section the two different feature recognition methods are compared. Note, however, that their performances depend on parameters which do not always apply to both methods.

Test runs were carried out on a PC running Windows 7 64-bit with an Intel® Xeon® processor at 2.53 GHz. It takes about 12 seconds to acquire feature points for one query floorboard and then compare it against the database of 886 floorboards, i.e., around 14 ms to compare it against one database-file. The Block matching method and the SURF method take roughly the same amount of time.

4.1 ROBUSTNESS TESTING
For every distorted image, the best matching board in the database was chosen based on having the greatest number of unique matching feature points. Both the Block matching method and the SURF method successfully identified the correct individual in Figure 4.

The identification accuracy of the two methods for different parameter combinations can be seen in Table 2. The Block matching method performs well as long as the scaling parameters are kept at the low level. The scaling in x-direction, λ₁, has the biggest impact on the accuracy. Inspection of some of the failed cases showed that occasionally the FAST corner detector misinterpreted sections of the boards’ outer borders as corners and thus a lot of feature points were rejected. The worst case occurred at a large skewing angle, φ, in conjunction with a large scaling in x-direction, λ₂. However, neither a large rotation angle nor a large skewing angle seems to correlate significantly with bad accuracy.

The SURF method’s identification accuracy remains decent when the scaling parameters are kept at the lower level. However, the SURF method has big trouble when either λ₁ or λ₂ is at the high level. A noted problem is that certain wood features that the SURF method sees as especially interesting are found at many different scale levels. Thus, several scattered features in one image can be falsely matched together with a tight cluster of features in the other image. This effect is most evident when either of λ₁ or λ₂ is at the high level and the other one is not. The SURF method, however, does not experience the same problems of finding invalid feature points on edges of the boards.

A large number of matched feature points usually means that the correct board has been identified. If the second best matching board has almost the same amount of matched feature points, then the chances are high that the “best” might not be the correct one. To visualize the difference between the number of matched feature points for the best match and the second best match, for small geometric distortions, a box plot was created (Figure 5).
Each of the 886 floorboards was given a small geometric distortion and was then matched against the original images. The parameters of the affine transformation were assigned uniformly distributed random numbers on the intervals: $-5^\circ \leq \theta, \varphi \leq +5^\circ$ and $0.9 \leq \lambda_1, \lambda_2 \leq 1.1$. The plots show that the Block matching method has a slightly higher mean number of matched features for the best match.

### 4.2 RE-SCANNED IMAGES

An example of how one of the re-scanned images could look is shown in Figure 6. Both the Block matching method and the SURF method successfully identified the correct board also in that case.

The identification accuracy of the 44 re-scanned boards can be seen in Table 3. The Block matching method outperforms the SURF method for this particular case comprising small geometric distortions and moderate radiometric changes. Due to the smaller geometric distortions in this experiment, the drawbacks of the two methods should not negatively affect the accuracy to the same extent.

![Figure 4: Example of a simulated distorted floorboard image with $(\theta, \varphi, \lambda_1, \lambda_2) = (-5^\circ, -5^\circ, 0.9, 0.9)$ which has been correctly identified in the database. (a) Block matching method. (b) SURF method.](image)

<table>
<thead>
<tr>
<th>Block matching method</th>
<th>SURF method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accuracy (%)</td>
<td>Accuracy (%)</td>
</tr>
<tr>
<td>93</td>
<td>98</td>
</tr>
<tr>
<td>94</td>
<td>48</td>
</tr>
<tr>
<td>90</td>
<td>36</td>
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<td>78</td>
<td>75</td>
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<td>98</td>
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<td>35</td>
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<td>93</td>
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<td>98</td>
<td>53</td>
</tr>
<tr>
<td>96</td>
<td>28</td>
</tr>
<tr>
<td>82</td>
<td>79</td>
</tr>
</tbody>
</table>
A visualization of the difference between the numbers of matched features for the best match and the second best match is shown for each of the 44 boards in Figure 7. The same tendency as before can be seen there. The SURF method has a more widespread number of matched features, though there is more often a smaller difference between the best and the second best match. This means that the SURF method can be seen as less robust in that sense.

Table 3: Comparison of the identification accuracy between the feature point methods for the re-scanned images.

<table>
<thead>
<tr>
<th>Method</th>
<th>Accuracy (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block matching</td>
<td>95.5</td>
</tr>
<tr>
<td>SURF</td>
<td>86.4</td>
</tr>
</tbody>
</table>

4.3 DIGITAL PHOTOS

The last and most challenging case was that of matching low resolution digital images with the original, higher resolution, images. The digital images were cropped and scaled to match the resolution of the original images. The identification accuracy is shown in Table 4, where the SURF method is clearly the most robust method in this case. Closer investigation showed that the FAST corner detector had trouble finding features in these images. Different settings were tested but did not produce more feature points. Hence the somewhat undeserved poor result for the Block matching method.

Table 4: Comparison of the identification accuracy between the feature point methods for the digital photos.

<table>
<thead>
<tr>
<th>Method</th>
<th>Accuracy (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block matching</td>
<td>9.1</td>
</tr>
<tr>
<td>SURF</td>
<td>34.1</td>
</tr>
</tbody>
</table>

Figure 5: The number of matching feature points for the best matching floorboards compared to the second best match when: \(-5^\circ \leq \theta, \varphi \leq +5^\circ\) and \(0.9 \leq \lambda_1, \lambda_2 \leq 1.1\). (a) Block matching method. (b) SURF method.

Figure 6: Example of a re-scanned floorboard image identified in the database. (a) Block matching method. (b) SURF method.
5 CONCLUSIONS

There are many parameters that can be varied and tweaked to get different amounts of feature points from the algorithms. If thresholds for accepting corners are lowered, more feature points will be returned. However, these will be less robust.

A drawback of having a lot of features is that the extraction and matching procedure will take longer, making a real-time application less feasible.

A problem with the FAST corner detector is that it can in some cases misinterpret jagged lines and edges as corners. The SURF method’s corner detector is in this respect more reliable and better at finding features in an image.

The method of finding interest points using corners, i.e., a high second derivative response in two directions, is a very general method. It is widely used within machine vision in various different scenarios. A wooden surface with knots is a special case that probably should be dealt with using a more specialized knot-finding algorithm.

The Block and SURF features are likewise common general purpose feature points, but not very specialized towards recognizing wood. The advantage of these methods is that they are quite robust. It would be easy to re-tune the parameters and apply the methods on a slightly different recognition problem.

The SURF method’s problem of incorrectly matching several scattered features in one image to a tight cluster of features in the other image must be addressed. This is not a huge problem and can definitely be solved.

The Block matching method performed surprisingly well in all experiments, especially considering that it is scale and rotation dependent. The method would, however, greatly benefit from using a more robust knot-finding algorithm.

The rotationally invariant SURF method generally handled rotations worse than the Block matching method, which was unexpected. Rotational invariance is possibly not as crucial due to knot features being mostly rounded.

Both the Block matching method and the SURF method are valid options for recognizing wood products covered with distinct features. The Block matching method outperformed the SURF method for small geometric distortions and moderate radiometric distortions. The SURF method, in its turn, performed better compared to the other method when faced with low resolution digital images. However, the FAST corner detector often failed to acquire enough valid interest points to the Block matching method in this case.

A combination of Block- and SURF features would most certainly provide a big improvement in identification accuracy.

Future work will include the incorporation of multi-sensor data and defect classification used in existing systems for wood inspection.

ACKNOWLEDGEMENTS

The authors would especially like to thank Olof Broman for giving us access to the floorboards used in this work. Also, thanks to Simon Dahlquist at SP Trä for assisting us with the re-scanning of the boards.

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[21] Broman, N. O., Nyström, J., and Oja, J., Modelling the connection between industrially measured raw
ABSTRACT: This paper reports on the new COST Action FP1101 “Assessment, Reinforcement and Monitoring of Timber Structures”. The objective of the Action is to increase confidence of designers, authorities and end-users in use of timber in the design of new and in the repair of old structures by developing and disseminating assessment, reinforcement and monitoring methods and guidelines. COST Action FP1101 will build on previous findings, target the existing shortcomings, benefit from multidisciplinary views and innovative solutions by the involved stakeholders, enable synergies between them, and provide an effective way of discussing and disseminating the results from on-going projects. The paper summarizes the relevant results from COST Action E55 and RILEM committee AST-215, provides the rational for the new Action, and gives an overview of the proposed scientific program.

KEYWORDS: timber structures, assessment, reinforcement, monitoring, COST Action

1 INTRODUCTION

Timber has been used as structural material for centuries and numerous examples demonstrate its durability if properly designed and built and when adequately assessed and monitored. In recent years, the use of timber in structures has gained new importance, considering that it is the only truly renewable building material. The timber sector in Europe, however, is characterized by relatively small and locally working companies. While the level of technologies and experience in design and construction is relatively good, the current practice of assessing and monitoring of existing timber structures, based on very heterogeneous traditions, is not sufficient to ensure confident decisions about their reliability and can lead to unnecessary costly rehabilitation measures or premature dismantling.

1.1 ASSESSMENT OF TIMBER STRUCTURES

The last decades were marked by a widening in the range of application of timber in structures and consequently a growing importance of the assessment of these structures. The time and cost of structural assessment are justified by ensuring safety, and protecting investments and cultural heritage.

A wide variety of methods exist to evaluate timber structures, however, their frequency and scope, the decision making approach concerning safety and the necessary interventions are far from being agreed upon. Most assessment methods used today can give qualitative information about the state of in-situ timber, but only few give reliable quantitative information [1,2]. Methods can be non-destructive (NDT); which are useful for the screening for potential problem areas for the qualitative assessment of structures. But a drawback of NDT is the relatively poor correlation between the measured quantity and material strength. Semi-destructive techniques (SDT) bridge the gap between NDT and fully destructive methods; they often require the extraction of samples for subsequent testing to determine elastic and strength parameters while preserving the member’s integrity. One problem of SDT is the high variability in test observations.
1.2 RETROFITTING OF TIMBER STRUCTURES

The need for structural reinforcement of timber buildings results from various requirements such as change of use, deterioration, exceptional damaging incidents, new regulatory requirements, or interventions to increase structural resistance. About 50% of all construction in Europe is related to existing buildings, this leads to a growing need for the maintenance and upgrading of existing buildings, not only for economical but also environmental, historical and social concerns. Over 80% of European buildings are over 50 years old; they need to be upgraded to reflect the requirements of energy- and use-efficiency [3,4].

Recent developments related to structural reinforcement [5-10] can be grouped into three categories: (i) addition of new structural systems to support the existing structure, (ii) configuration of a composite system (timber-concrete, timber-steel, timber-FRP, and timber-timber), and (iii) incorporation of reinforcing elements to increase strength and stiffness. Rational guidelines are needed for these technologies for in-situ use and special considerations are necessary when the structure belongs to cultural heritage.

1.3 MONITORING OF TIMBER STRUCTURES

The monitoring of timber structures received special attention after the collapse of the ice rink in Bad Reichenhall, Germany, in 2006 [11], which only stands as one example of a series of structural failures [12]: e.g. those of over 50 timber structures in Sweden [13].

Structures are being monitored: i) during structural renovations where the acquired data is used to provide the basis for further action; ii) to acquire information when progressive phenomena are suspected; iii) to prevent or reduce the cost of interventions during building maintenance; and iv) to evaluate the long-term effectiveness of interventions.

Although recent developments focus on simple, robust and redundant systems [14,15], presently, the monitoring of timber structures mostly consists of regular on-site visits [16,17] that only give qualitative answers to whether a structure conforms to regulations or not.

2 SUMMARY OF RECENT WORK

2.1 COST ACTION E55

The main objective of COST Action E55 was to provide the basic framework for the efficient and sustainable use of timber as a structural and building material. The Action was structured into three working groups: 1) Assessment of failures and malfunction, 2) Vulnerability of timber structures, and 3) Robustness of timber structures. Within the scope of Working Group 1, focus was laid on studying structural failures of timber structures around Europe, resulting in a scheme to classify failures in the future.

Another objective was to discuss and develop guidelines on implementing inspections on structures to improve their reliability over the anticipated lifetime. To expand this domain, it seemed desirable to append a collection and evaluation of existing assessment and monitoring methods. A Task Group within E55 targeted this objective and published its findings [2]. The report contains the assessment methods which have been evaluated by a group of experts against keywords like applicability, expenditure of time/cost, validity of results and possible constraints. In addition, common approaches towards the assessment of timber structures and the subsequent documentation are given. Consent was found that most assessment methods utilized today can give qualitative information but only few NDT methods can deliver quantifiable information.

Methods to determine strength parameters of built-in timber elements are very scarce; more methods exist to derive timber stiffness parameters. The correlation, however, between stiffness and strength parameters is usually not very high, especially for strength parameters featuring brittle failure modes. From this latter finding and the fact that any single method only allows assessing certain types of material properties, damages or degradation processes, follows the necessity to combine assessment methods to derive a clearer picture of the performance of the structure.

All data require careful evaluation by an experienced engineer and/or wood scientist who considers the context of the structure under investigation. The judgement on the results, which is in the responsibility of the expert carrying out the assessment, will therefore oftentimes, be set up from a standpoint which could be summarized as “the best knowledge available”.

The above-mentioned facts show that more focus should be given to the optimization of assessment methods. This includes the improvement of frequently applied methods such as mapping of cracks and measurement of timber moisture content in terms of expenditure of time/cost. In addition, emphasis should be laid on the further development of methods for the in-situ determination of timber strength parameters. The objective of such developments should be to be able to incorporate results from structural assessments into updated models of structural systems (“system updating”).

2.2 RILEM TECHNICAL COMMITTEE AST-215

The RILEM (The International Union of Laboratories and Experts in Construction Materials, Systems and Structures) Technical Committee AST-215 was established in 2005 and targeted in-situ timber structural members and evaluation of their material (physical and mechanical) characteristics. The physical properties included the species, age, moisture contents, density and level of deterioration; the mechanical properties entailed strength and stiffness. The phenomena studied included life expectancy, failure probability and durability of the structure under investigation.
NDT and SDT methods were experimental validated on case studies; theoretical studies focused on reliability and robustness of the NDT, SDT and their combination. Technical workshops (07/2006, Prague, Czech Republic; 06/2010, Biel, Switzerland) were organized and allowed training technicians and students from North America and Europe.

A comprehensive literature review of the state-of-the art was published [1]. This report reflected the knowledge in in-situ assessment of the physical and mechanical properties of timber at the time. NDT and SDT methods are described in a systematic manner discussing the technology, equipment and limitations. Some of the discussed methods are used in other materials such as masonry and concrete; most of the methods, however, are specific to wood and special qualifications are required to understand and apply these effectively.

One of the main conclusions of RILEM TC 215 was that more research is required to both estimate individual member strengths and obtain accurate quantification of deterioration to improve the assessment of in situ timber members. As an example, after extensive research of stress wave techniques, the conclusion was drawn that the relationship between stress wave parameters and timber mechanical properties is not developed enough to accurately predict in situ member strength. Published research and correlation values are inconsistent and at times conflicting depending on technique, species and parameters employed.

Amongst several others, research opportunities were identified in the area of applying stress wave techniques in-situ to arriving at reliable strength estimates. Accurate quantification of deterioration also needs to be addressed, while X-ray and resistance drilling techniques have proven that they can accurately detect areas of deterioration, these techniques can be improved by adding the ability to quantify and identify different stages of deterioration.

3 OBJECTIVE

Research on the assessment of timber structures has been coordinated by the COST Action E55 “Modelling the Performance of Timber Structures” and the RILEM technical committee AST-215 “In-situ assessment of structural timber”. The European Commission mandates the development of rules for the assessment of existing structures and their reinforcement (M/466 EN) [18]. COST represents an ideal platform to support this assignment. The objective of the COST Action FP1101 “Assessment, Retrofitting and Monitoring of Timber Structures” is to help increase confidence of designers, authorities and end-users in safe, durable and efficient use of timber [19].

4 SCIENTIFIC PROGRAMME

According to the objectives, the Action’s scientific programme will be divided into three scientific areas expressed as work groups (assessment, reinforcement, monitoring). Common to these areas are following tasks:

- Analysing different approaches and evaluating methods in terms of applicability, expenditure of time/cost, validity of results and constraints;
- Adapting techniques and methods from other materials and systems;
- Validating new technologies with laboratory and on-site experiments;
- Adopting principles for the preservation of historic structures;
- Creating practical tools with an immediate applicability;
- Establishing common practices and methodologies;
- Disseminating knowledge of approaches, new methods and technologies.

4.1 PLANNED ACTIVITIES “ASSESSMENT”

- Increasing knowledge in NDT and SDT and combinations thereof to improve applicability of results to assess the remaining structural capacity;
- Compiling methods which deliver reliable and robust results that can subsequently be incorporated into analytical and probabilistic models;
- Promoting the cross-validation of data obtained during inspection using similar methods in different projects;
- Combining visual grading, vibrational methods and mechanical tests, for the decay characterisation and the mechanical characterisation of the material;
- Developing specific in-situ grading standards to both estimate individual member strengths as well as obtain accurate quantification of deterioration.

Research on the following methods will be coordinated:

- Simplified methods for the reliability assessment of structures based on calculations, analytical and visual inspection procedures;
- NDT (e.g. stress wave based techniques, X-ray radiography, Ground Penetrating Radar, vibrational methods, scanning, tomography);
- SDT (e.g. drilling techniques, hardness testing);
- Proof loading actual structures.
4.2 PLANNED ACTIVITIES “REINFORCEMENT”

- Identifying and categorizing types of deterioration, damage and failure of timber structures, weak zones and their relevance for safety;
- Facilitating the decision-making process for choosing an appropriate reinforcement method with consideration of cultural heritage aspects;
- Creating a handbook of solutions for the main categories of problems;
- Analysing the relationship between reinforcement techniques and protection technologies, as i.e. coatings to prevent decay and/or fire resistance;
- Evaluating solutions regarding their function as seismic reinforcement;
- Developing computational concepts that allow for safe and reliable design.

Research on the following systems will be coordinated:

- Timber and timber based products (e.g. densified, cross-laminated timber);
- Adhesive systems (considering on-site application and durability issues);
- Mechanical fasteners (e.g. glued-in rods or self-tapping screws);
- Fibre reinforced polymers and natural fibres also applied with adhesives;
- Nanotechnology (e.g. carbon nanotubes with polymeric resins).

4.3 PLANNED ACTIVITIES “MONITORING”

- Identifying relevant properties that should be monitored, including environment;
- Clustering long term experiments according to the involved risks as basis for the development of advanced diagnostic tools and technologies;
- Digital image processing, remote data acquisition and early warning systems;
- Defining criteria for the efficiency control of applied monitoring approach by means of numerical simulations and/or field and laboratory testing;
- Developing practical-operative guidelines and monitor schemes, e.g. survey, documentation, on-site inspections decision making guidelines.

Research on the following systems will be coordinated:

- Simple, robust, redundant and reliable long term moisture measuring devices;
- Deflection and crack propagation measuring devices and methods including contact sensors (fibre-optics) and non-contact approaches (photogrammetry);
- Acoustic emission monitoring;
- Remote data transmission systems;
- Wireless sensors and sensor networks.

5 CONCLUSIONS

The COST Action FP1101 “Assessment, Reinforcement and Monitoring of Timber Structures” will form a platform to coordinate on-going research, summarise recently obtained results, and increase and disseminate knowledge. The stakeholders are architects, structural engineers, researchers, lecturers, manufacturers, builders, students, policy makers and standardization bodies. The Action will benefit from multidisciplinary approaches, enable useful synergies and provide the most effective way of avoiding duplication and disseminating the results from a large number of on-going projects.

The Action will create scientific, economic and social benefits. Improving the assessment, retrofitting and monitoring of timber structures are research domains with large-scale activities which require the exchange of information and identification of new research ideas. Increased knowledge of retrofitting techniques will help architects and engineers to make timber a viable option for more applications and new opportunities in design and construction. Reliably assessing timber structures, avoids failures and unnecessary decommissioning, and leads to safer structures and better use of resources. An increased and more innovative use of timber and timber-composites as building materials based on stronger confidence will bring sustainable benefits.

ACKNOWLEDGEMENT

This publication represents a fraction of the findings of the COST E55 task group “Assessment of Timber Structures” and the RILEM committee AST 215 "In-situ assessment of structural timber". The commitment and contributions of all members of both committees are greatly appreciated. Gratitude is addressed to the COST Office for funding the task group meeting and the publication of the guideline and to RILEM for the support in publishing the State-of-the-art-report.
REFERENCES


NUMERICAL ANALYSIS OF COMPOSITE STRUCTURE WOOD CONCRETE IN FIRE

Julio Cesar Molina¹, Carlito Calil Junior²

ABSTRACT: In this paper, the temperature fields of wood-concrete composite beams, of section T, with connectors formed by steel rods, in vertical position to the grain, were evaluated using the software ANSYS. The evolution of temperature was made according to ISO 834:1999 standard and the temperature in both elements concrete and wood was measured after 30 minutes of exposition in fire with the fire in three sides exposed of the model. The values of temperature were compared with the results from other researchers showing a good agreement. The numerical model presented in this paper may be used to determine the depth of carbonization of wood and also the temperature into the concrete with reasonable approximation.

KEYWORDS: Composite structures timber-concrete, numerical modelling, temperature field, fire

1 INTRODUCTION

Fire resistance is defined as the ability of a material or structural element to remain during a certain time, performing the functions for which it was designed, under the action of fire. In Brazil, most studies have a character essentially numeric because there is still no horizontal furnaces in the country for tests of structural elements with real dimensions under load. The first horizontal furnace of Brazil of this nature will be in operation in the second half of 2012.

For the other hand, the advanced calculation models are based on the numerical methods such as Finite Difference Method (FDM) and Finite Element Method (FEM) and these methods allow obtaining the temperature during the heating process.

For use of these methods it is necessary the use of computer programs with settings and tools compatible to numerical effort to solve the problem. Some programs used in this case are: ADAPTIC, SUPRTEMPCALC, SAFIR and VULCAN. However, the numerical analysis of heat transfer can also be performed by commercial packages developed based on the FEM such as ANSYS, ABAQUS, ADINA and DIANA, all known worldwide.

 Thermal analysis usually has a transient character considering of the fact that the temperature in an environment in fire has variation with the temperature. This work originated from the necessity of obtaining information about the modeling of composite beams of wood and concrete in fire.

The main aim of this work was the simulation, through numerical modeling, of the temperature field in a wood-concrete composite beam, of section T, with connectors formed by steel rods, in vertical position to the grain. From the numerical model proposed was possible the presentation of the main finite elements used in the numerical model which consider the variation of temperature and also determinate the depth of carbonization of wood besides the temperature into the concrete with reasonable approximation. The thermal properties for the materials wood and concrete used in the numerical model were also presented.

2 NUMERICAL MODELLING

The numerical modeling was performed from a bi-dimensional model using the software ANSYS, version 10.0, that has with base the FEM. The temperature in the composite cross section was measured after 30 minutes of exposition in fire. Figure 1 shows the composite cross section considered in the numerical model that was defined with base in the structural elements evaluated in the works [1,2].

In the composite cross section the element of timber present dimensions of 16 cm (width) and 26 cm (height) and the concrete element present dimensions of 100 cm (width) and 10 cm (height). To facilitate the construction of the numerical model the materials wood and concrete, that form the cross section, are modelled for provide

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common nodes at the interface between the referred elements and this detail could be observed in Figure 4. The steel connection system was modelled for a common node coupled between both materials wood and concrete.

![Figure 4: Mesh of finite elements](image)

### 2.2 DISCRETIZATION OF THE MESH

The mesh of finite elements related to the composite cross section was discretized in plan elements (PLANE177) having each element dimensions of 1 cm x 1 cm totalling 1416 finite elements in the cross section as shown in Figure 4. Of this total, 1000 finite elements composes the part of the concrete and 416 the part of the wood.

![Figure 2: Finite element PLANE77. Source: [3]](image)

![Figure 3: Finite element SURF151. Source: [3]](image)

In the lower boundary of cross section were applied 304 elements type surface (SURF151) in two different layers, being 152 elements for the radiation effects and the others for the convection effects as shown in Figure 5.

Analysis considered was of the “thermal transient” and the numerical results were evaluated for time 30 minutes for the depth with about of 10 mm. The heating standard curve used in the numerical model to verify of the evolution of temperature in the composite cross section was the [4] In the numerical model the composite cross section was exposed to the fire only on three sides, i.e., the top surface of the concrete slab has not been exposed, as shown in Figure 6.
2.3 THERMAL PROPERTIES OF WOOD

The values used for the wood in the numerical model are according to [5] that considered wood of the type *Eucalyptus citriodora*. Figure 7 shows the thermal conductivity values adopted for the wood.

![Figure 7: Thermal conductivity](image)

The range of values of specific heat as a function of temperature for the wood is shown in Figure 8.

![Figure 8: Specific heat](image)

The values of density versus temperature for the wood as showed in Figure 9. In this case, to obtain the density resorted to thermogravimetric analysis as presented in [1]. The results shown in Figure 9 have been calibrated for insertion into the numerical model, adopting the residual value of 10% for the relative density.

![Figure 9: Density of the wood.](image)

The coefficient of convection adopted for the finite element (SURF151-convection) that bypassed the timber element surface of the composite cross section was equal to 13.5 W/m²K.

The emissivity value adopted for the finite element (SURF151-radiation) that bypassed the timber element surface of the composite cross section was equal to 0.6.

For temperatures of until approximately 100 °C is relatively wide the amount of available information about the thermal properties of some wood species. However for higher temperatures that doesn't happen and studies correlating properties thermal versus temperature are relatively scarce, mainly when it is species hard wood.

2.4 THERMAL PROPERTIES OF CONCRETE

The values of properties used for the concrete in the numerical model are according to [2,6] for comparison of the results. Thus, the thermal conductivity of the concrete, \(\lambda\), varies with the temperature, \(\theta\), according Equation (1) below:

\[
\lambda(\theta) = \begin{cases} 
1.748 & \text{if } \theta \in [20;199] ^\circ \text{C} \\
1.748 - 1.246 \times 10^{-3} & \text{if } \theta \in [199;899] ^\circ \text{C} \\
0.846 & \text{if } \theta > 899 ^\circ \text{C}
\end{cases}
\]

The value of specific heat used for the concrete was constant and equal to 1139 J/kg/°C and the density was 2403 kg/m³.

The coefficient of convection adopted for the finite element (SURF151-convection) that bypassed the concrete element surface of the composite cross section was equal to 25 W/m²K.

The emissivity value adopted for the finite element (SURF151-radiation) that bypassed the concrete element surface of the composite cross section was equal to 0.522.

In the application of fire safety it is assumed that the gases of the combustion and the structural elements are in direct contact, so that this phenomenon can be treated as the case of two infinite surfaces (plates). In this case the configuration factor (\(\Phi\)), for definition, is equal to 1. The configuration factor represents the fraction of the radiant thermal energy that leaves a surface (issuing) and that indeed intercepts another surface (receiving), being that there is loss of energy along the road among the surfaces.

The configuration factor depends on the special configuration between the issuing and receiving surfaces.

The constant of *Stefan Boltzmann* used in the numerical model to the radiation effects was 5.67 x 10⁻⁸ W/m².K⁴.

This value was used in the radiation surfaces of wood and concrete.

In the interface of the materials concrete-wood the degrees of the freedom “temperature” of overlapping nodes were coupled. This tactics was used to provide the continuity of the temperature field between the jointed materials.
3 CALIBRATION OF THE MODEL

The calibration of the numerical model was made of the form independent for each material. Thus, for the element of wood was considered the experimental results obtained by [1].

The medium values of the experimental results in this case were obtained for specimens of wood for the depth of 10 mm and 20 mm. Figure 10 shows the curves of temperature versus time for the comparison of the experimental and numerical results.

The curve [4] presented in Figure 10 corresponds to the control of heating of the *extra node*.

The calibration of the numerical results for the concrete element was made with base in the work developed by [2] through the comparison with other numerical models presented by another researchers.

4 RESULTS

Figure 11 shows the temperature field for the composite cross section considering a time of 30 minutes (1800 seconds) of exposition to the fire standard according curve [4].

![Figure 10: Comparison among the numerical and experimental results obtained by [1] for the evolution of the temperature in function of the time for the wood element.](image)

![Figure 11: Temperature field obtained in the numeric analysis by the software ANSYS for time of 30 minutes (1800 seconds)](image)

ACKNOWLEDGEMENT

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Selected physical and mechanical properties of Norway spruce (Picea abies) grown in the European Alps on different sea level and hillside orientation

Wilfried Beikircher¹, Manuel Mössl², Anton Kraler³

ABSTRACT: The goal of this study was to determine the mechanical properties on defect free samples of Picea abies in dependence of the growing area. The material was selected regarding different sea level (1060 m to 1760 m) and hillside orientation (North side and South side) in the Austrian Alps. Seven to nine trees of each growing area were harvested and adapted for the investigation. Following mechanical and physical properties will be presented in the paper: equilibrium moisture content at 20 °C/65 % RH (%), density at 20 °C/65 % RH (kg m⁻³), longitudinal compression strength (N mm⁻²), Brinell hardness (500 N) (N mm⁻²), modulus of elasticity (bending test) (N mm⁻²), bending strength (N mm⁻²) and the maximal swelling (%).

The results suggest that Brinell hardness, compression strength and Young’s moduli are clearly correlated with the density and can marginally be related to the growing area. The investigations give information for the influence on the growing area regarding mechanical properties and support the development on optimisation of machine grading equipments.

KEYWORDS: Picea abies, mechanical properties, modulus of elasticity, hardness, swelling coefficient

1 INTRODUCTION
For wood in general and especially for Norway spruce much information about the mechanical and physical properties is available [1-3]. For the wood properties related to different growing areas as the sea level and hillside orientation very little information is available [2, 4]. The performed investigations should give information to the correlation of the properties related to the growing area and close this gap.

2 EXPERIMENTAL INVESTIGATIONS
2.1 MATERIAL
The material was selected regarding different sea level (1060 m to 1760 m) and hillside orientation (North side and South side) in the Austrian Alps. For the North side material was taken from the sea level of 1060 m, 1190 m, 1380 m, 1620 m and 1700 m and for the South side material from the sea levels of 1160 m, 1400 m, 1600 m and 1780 m were taken as shown in Figure 1.

Figure 1: Scheme of sampling sites in Austria (NORDTIROL - Navistal)
Seven to nine trees of each growing area were randomly selected, harvested and adapted for the investigation. The material was taken from the tree at the distance of about 4.5 m from the basic to avoid growing defects such as compression wood. From each tree four defect free small samples for bending tests (20 x 20 x 360 mm³) were prepared. For the determination of further properties samples were prepared out of the material from the bending test material (see Figure 2). The batch size varied from 26 to 36 samples depending on the quality and the material availability. In total 268 samples were tested.

![Figure 2: Sample preparation out of the bending samples: a – compression strength, b – Brinell hardness, c – swelling, density and Moisture content, d -swelling longitudinal](image)

**2.2 METHODS**

The determination of the material properties were performed according to the Standards for defect free small wood samples. The material properties to be determined were: the density (DIN 52182), moisture content (DIN 52183), modulus of elasticity and bending strength (DIN 52186), compressive strength (DIN 52185), Brinell hardness (EN 1534) and the maximum swelling (DIN 52184). The material tests were performed at the TVFA (Technische Versuchs- und Forschungsanstalt) of the University of Innsbruck. All tests were performed after storage the material in a climate chamber at 20 °C and 65 % RH until the equilibrium moisture content (EMC) was reached.

**3 RESULTS AND CONCLUSION**

Examining the results in Table 1, the values determined on the southern slope side are higher than those from the northern side. Furthermore the values of the material properties are increasing from the lowest investigated altitude up to 1190 m on the northern slope side and 1400 m on the southern slope side. From this level the values are decreasing continuously up to the highest level (see Figure 3-5). By comparing the testing results with the mean values according to ÖNORM B 3012, the values for the density, the bending strength and the modulus of elasticity could not be reached. The mean values of the compression strength for the tested material show a similar value as given by the standard. The determined maximum swelling and Brinell hardness are higher, except for the growing area at 1700 m_N, than the values according to the referred standard.

![Figure 3: Bending strength of Norvay spruce from different sea levels in Austria](image)

![Figure 4: Modulus of elasticity of Norvay spruce from different sea levels in Austria](image)

![Figure 5: Compression strength parallel to grain of Norvay spruce from different sea levels in Austria](image)

With this study a dependency of the wood properties related to the growing area could be presented. The
gained information should support the development on optimisation of machine grading equipments.

Due to the variation of wood properties related to the growing area more information regarding growing effects should be considered in future investigations which include also aspects from the factors influencing the wood growth e.g. nutrient and water availability and other.

ACKNOWLEDGEMENT
The authors gratefully acknowledge the Holzcluster Tirol for providing the material out of the founded project “Gebirgsholz-Wald ohne Grenzen” under the program line Interreg IV.

Table 1: Material parameters of Norvay spruce from different sea levels and hillside orientations in Austria

<table>
<thead>
<tr>
<th>Sea level</th>
<th>n</th>
<th>Moisture content 20°C/95% rh [%]</th>
<th>Density ρ₀ [kg/m³]</th>
<th>Bending strength [N/mm²]</th>
<th>Modulus of elasticity [N/mm²]</th>
<th>Compression strength parallel to grain [N/mm²]</th>
<th>Maximum swelling [%]</th>
<th>Brinell hardness HB10/500 [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1600m_N</td>
<td>28</td>
<td>10.5</td>
<td>402.9</td>
<td>76.4</td>
<td>8.965</td>
<td>43.6</td>
<td>4.4</td>
<td>4.9</td>
</tr>
<tr>
<td>1160m_S</td>
<td>35</td>
<td>10.7</td>
<td>413.1</td>
<td>78.8</td>
<td>9.425</td>
<td>44.1</td>
<td>4.3</td>
<td>9.2</td>
</tr>
<tr>
<td>1190m_N</td>
<td>24</td>
<td>0.47</td>
<td>347</td>
<td>10.9</td>
<td>1.228</td>
<td>5.3</td>
<td>0.7</td>
<td>1.2</td>
</tr>
<tr>
<td>1380m_N</td>
<td>28</td>
<td>10.9</td>
<td>409.1</td>
<td>78.3</td>
<td>8.899</td>
<td>42.6</td>
<td>4.2</td>
<td>9.1</td>
</tr>
<tr>
<td>1400m_S</td>
<td>34</td>
<td>11.2</td>
<td>463.7</td>
<td>95.6</td>
<td>11.118</td>
<td>50.0</td>
<td>5.1</td>
<td>10.2</td>
</tr>
<tr>
<td>1600m_N</td>
<td>29</td>
<td>11.1</td>
<td>441.7</td>
<td>87.9</td>
<td>9.962</td>
<td>46.2</td>
<td>4.8</td>
<td>9.3</td>
</tr>
<tr>
<td>1620m_N</td>
<td>35</td>
<td>0.59</td>
<td>35.6</td>
<td>11.2</td>
<td>1.279</td>
<td>4.4</td>
<td>0.7</td>
<td>0.9</td>
</tr>
<tr>
<td>1700m_N</td>
<td>26</td>
<td>10.8</td>
<td>403.8</td>
<td>75.4</td>
<td>8.893</td>
<td>42.3</td>
<td>4.2</td>
<td>9.1</td>
</tr>
<tr>
<td>1760m_S</td>
<td>28</td>
<td>10.6</td>
<td>571.7</td>
<td>68.5</td>
<td>7.864</td>
<td>37.4</td>
<td>3.6</td>
<td>8.4</td>
</tr>
<tr>
<td>total</td>
<td>266</td>
<td>10.8</td>
<td>423.0</td>
<td>80.2</td>
<td>9.363</td>
<td>43.9</td>
<td>4.4</td>
<td>9.4</td>
</tr>
<tr>
<td>ONB 3012</td>
<td></td>
<td>-</td>
<td>441</td>
<td>95</td>
<td>12.500</td>
<td>44</td>
<td>3.7</td>
<td>8.5</td>
</tr>
</tbody>
</table>

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THE USE OF TORSION TEST METHOD TO EVALUATE THE SHEAR PROPERTIES OF TIMBER JOISTS

Aamir Khokhar¹, Hexin Zhang²

ABSTRACT: This paper presents details of the experimental method and test results from a series of torsion tests undertaken to evaluate the shear modulus and shear strength of solid timber joist. The variation of shear modulus within and between test samples, failure modes and the correlation of shear strength and shear modulus were studied. Test results obtained indicate that there was considerable (approximately 33%) variation in shear modulus between pieces of timber tested. It was also found that the shear strength of tested joists was higher than the published values in EN 338. The test joists fractured mostly at the middle with cracks propagated towards either supports or edges. A good correlation was found between shear strength and the shear modulus. The recent revision of the testing standard EN408 includes the torsion testing approach to obtain the shear modulus of timber. It is proposed that a torsion test also be adopted as a method for evaluating the shear strength of timber.

KEYWORDS: Torsion test, Shear strength, Shear modulus, Design values, Failure mechanism

1 INTRODUCTION

The shear modulus and shear strength are fundamental mechanical properties of wood that is used in general timber design. Compared with other engineering materials, timber has a relatively low shear stiffness and strength in comparison to its modulus of elasticity, and so shear deformation contributes a more significant portion of flexural deflection. In design, the shear properties are important factors for lateral-torsional stability of joists, particularly those with a long span and no lateral supports [e.g., 1]. The shear modulus is also needed when designing serviceability of wood-joist floors [e.g., 2], and is an input into analytical [e.g., 3] and finite element [e.g. 4] models to predict the vibrational serviceability.

The shear modulus and shear strength can be typically obtained from shear block [e.g., 5] or bending tests [e.g., 6, 7]. However, there are no known studies that have evaluated whether shear block and bending tests are suitable for determining shear properties in structural sized timber. Previous studies [e.g. 8, 9] have shown that the shear block test is inappropriate for estimating the actual shear strength of structural-sized timber because it includes stress concentrations and does not account for the influence of defects and orthotropy.

Likewise, the combination of flexural and shear stresses encountered in a bending test leads to difficulties in obtaining the true value of shear strength [e.g 10, 11].

In contrast, testing a structural member in torsion creates a state of pure shear. Therefore, this approach could be better suited to obtaining the shear modulus and shear strength of wood. In this regard, Gupta et. al. [12, 13] used both experimental and finite element approaches to examine the torsion test method and concluded that it is a better approach to obtain the shear strength than other methods. Considering the limitations of shear block and bending tests for determining shear properties, it is not unreasonable to assume that they might not be appropriate for obtaining information on the shear modulus of structural timber. This was shown by Hindman et al. [14] who found that the torsional rigidity (GJ) of solid sawn timber and structural composite timber joists tested in torsion was 15 to 40% lower than values based on current methods [15]. Studies also show that torsional vibration provides a better measure of shear modulus than static bending [e.g. 16].

In this paper, an experimental study is described which was conducted to determine the shear modulus and shear strength of structural timber using torsion tests. The main objective was to investigate the variation of shear modulus within the length of the joists and to compare the shear strength test values with the published values from EN338 [17] and in the Wood Handbook [18]. The secondary objectives were to examine the failure mechanism of wood under torsion, the correlation of torsional shear strength with shear modulus.
2 MATERIAL AND METHODS
Sitka spruce (Picea sitchensis) and Norway spruce (Picea abies) joists of nominal cross section of 45 × 100 mm were tested. Sitka spruce timber of C16 strength class was cut into four different lengths of 1.0 m, 2.0 m, 2.8 m and 3.6 m with 15, 10, 12 and 25 samples, respectively selected for each length (denoted here SP). Norway spruce (NS) wood of strength class C16 and C24 was cut into 2.4 m lengths with 14 and 12 specimens respectively. Before testing, all samples were conditioned in a controlled-environment room (21°C and 65% relative humidity) until they attained constant mass (approximately 12% moisture content).

A 1 kN-m torsion testing machine (Tinius Olsen, Pennsylvania USA) was used to test the timber joists under torsion. To measure the twisting displacement of the timber, inclinometers with a range of ± 30° were attached to the upper edge (45 mm dimension) of each sample. Multiple inclinometers were attached to each sample to allow the longitudinal variation in shear modulus to be investigated. The mounting positions for the inclinometers depended on the length of sample being tested, but in all cases inclinometers were mounted at least 200 mm from the clamps to avoid possible end effects. For 1.0 m long samples, two inclinometers, each located 200 mm from the end clamps allowed displacement to be measured on a 600 mm central segment. The 2.0 m and 2.8 m samples were partitioned into four segments of 400 mm and 600 mm, respectively (Figure 1), while the 3.6 m samples were divided into five segments of 600 mm with the end inclinometers mounted 300 mm from the clamps.

![Diagram for test setup of 2.8m specimen](image)

All test specimens were tested at 4°/min [6] until they fractured. The shear strength and shear modulus (G) of each test specimen was calculated on the basis of Saint-Venant torsion theory for rectangular sections as follows:

\[
\text{Shear Strength} = \frac{\text{Maximum Torque}}{(d \cdot t^2 \cdot k_1)} \quad (1)
\]

\[
G = \frac{\text{Stiffness}}{(d \cdot t^2 \cdot k_2)} \quad (2)
\]

In Equations (1) and (2), d is the depth (major cross-section dimension) and t is the thickness (minor cross-section dimension) of the test specimen and \(k_1\) and \(k_2\) are constants that depend on the depth thickness ratio (see e.g. [19]). Shear strengths were calculated on the basis of the maximum applied torque, as shown in Figure 2. The stiffness was obtained by conducting linear regression analysis of the applied torque and the relative twist per length within the elastic region as shown in Figure 2. For most of the tested specimens the elastic region was found between 3% and 30% of maximum applied torque, and therefore, linear regression analysis was conducted between 5% and 25% of the maximum applied torque to obtain the stiffness.

![A typical applied torque and relative twist of 2.8m test sample](image)

3 RESULTS AND DISCUSSIONS
3.1 VARIATION IN SHEAR MODULUS WITHIN AND BETWEEN SAMPLES
The variation of shear modulus was conducted for Sitka spruce joists. It was seen that shear modulus varied substantially for Sitka spruce (SP) joists. Across all the segments from each of the specimens tested, shear modulus ranged from 298 MPa to 762 MPa (Figure 3). From the relatively limited number of samples tested to date, G does not appear to be normally distributed, but is positively skewed (Shapiro-Wilk test p-value = 0.013). It should be noted that segments are not all independent of each other, with each test specimen containing between 1 and 5 segments.

![Variation of shear modulus of the segments of tested samples](image)
components analysis was undertaken in order to determine the relative magnitude of between-specimen and within-specimen variation. This analysis revealed that 66 percent of the variation in G was due to differences between specimens, while the remaining 33 percent was due to differences between the segments within a specimen. It is hypothesised that the source of the variation between segments of a specimen, and the apparent trend of increasing shear modulus with length, may be due presence of knots and other defects. Longer specimen have a greater probability of having large knots than shorter specimen but each large knot takes up a smaller proportion of the total length. The trend of increasing G with specimen length should be interpreted with some caution as different length samples were obtained from different sources and no attempt was made to randomly select material for the different lengths.

**Table 1: Comparison of shear modulus (G) between different sample lengths**

<table>
<thead>
<tr>
<th>Group</th>
<th>Number of specimens</th>
<th>Number of segments</th>
<th>Mean G (MPa)</th>
<th>Max G (MPa)</th>
<th>Min G (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP 1.0m</td>
<td>15</td>
<td>15</td>
<td>465±94</td>
<td>715</td>
<td>298</td>
</tr>
<tr>
<td>SP 2.0m</td>
<td>10</td>
<td>10</td>
<td>497±58</td>
<td>604</td>
<td>339</td>
</tr>
<tr>
<td>SP 2.8m</td>
<td>12</td>
<td>12</td>
<td>513±79</td>
<td>695</td>
<td>375</td>
</tr>
<tr>
<td>SP 3.6m</td>
<td>16</td>
<td>8</td>
<td>542±91</td>
<td>762</td>
<td>382</td>
</tr>
<tr>
<td>Overall</td>
<td>53</td>
<td>183</td>
<td>565±87</td>
<td>775</td>
<td>421</td>
</tr>
</tbody>
</table>

### 3.2 DESIGN STANDARDS AND TORSIONAL SHEAR STRENGTH VALUES

Table 2 provides the mean shear strength values of both Sitka Spruce and Norway spruce beams. For C16 Sitka Spruce, the mean shear strength of 7.2 MPa was attained and that was about 15% lower than Norway spruce of the same grade, and 22% less than the C24 Norway Spruce. C24 class timber has the highest shear strength (9.3 MPa), which agreed with expectations that the higher strength class would have higher shear strength values. For the C16 of Norway Spruce the shear strength was about 9% lower than C24 of the same species. It was found that different species has different shear strength values. This is because the different species have different ratios of shear and bending properties.

In the current EN338 [20], the characteristic shear strength values for C16 and C24 are 1.8 MPa and 2.5 MPa respectively. These values are calculated on the basis of bending strength of full size structural timber beams tested under four point bending test in accordance with EN408 [21]. Much higher characteristic shear strength values of 4.8 MPa (166% higher) of C16 (combined SP and NS) and 7.5 MPa (200% higher) of C24 were achieved when joists were tested under torque. The revision of EN338 [20] has raised values of characteristic shear strength for these grades (3.2 MPa and 4.0 MPa) but these are still substantially less than those observed experimentally in this study.

**Table 2: The mean shear strength values of tested joists**

<table>
<thead>
<tr>
<th>Group</th>
<th>Strength Grade</th>
<th>Length (m)</th>
<th>Number of specimen</th>
<th>Max. Applied Torque (N.m)</th>
<th>Mean Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C16</td>
<td>1.0</td>
<td>15</td>
<td>485</td>
<td>7.8</td>
<td></td>
</tr>
<tr>
<td>C16</td>
<td>2.0</td>
<td>10</td>
<td>460</td>
<td>6.7</td>
<td></td>
</tr>
<tr>
<td>C16</td>
<td>2.8</td>
<td>12</td>
<td>550</td>
<td>7.7</td>
<td></td>
</tr>
<tr>
<td>SP</td>
<td>3.6</td>
<td>25</td>
<td>475</td>
<td>6.7</td>
<td></td>
</tr>
<tr>
<td>C16</td>
<td>Overall</td>
<td>490</td>
<td>7.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS</td>
<td>C24</td>
<td>2.4</td>
<td>390</td>
<td>8.5</td>
<td></td>
</tr>
</tbody>
</table>

Based on shear block tests of clear samples, the Wood Handbook [18] provides the mean shear strength values for Sitka spruce and Norway spruce of 6.7 MPa and 7.4 MPa respectively. From this research, mean shear strength of SP was 7.2 MPa (8% higher) and for NS was 8.5 MPa (13% higher), even though the material tested was of structural size and not clear wood. Similarly, Riyanto and Gupta [11] have shown that in comparison to the shear block tests, the torsional shear strength values of Douglas-fir were about 18% higher than the shear strength values of tested shear blocks and about 20% higher than the published values in the Wood Handbook [18].

This comparison shows that relatively higher shear strength values were achieved when the torsion test approach was used, even in comparison to tests on clear timber. Although it should be noted that only two species were tested in this research, a marked difference in shear strength was found compared with values given in EN338 [20].

This suggests that the assignment of shear strength values according to the results of bending tests may be over-conservative, at least for lower grades where knots affect bending strength, but not so much the shear strength (which may even be improved by knots). It may be appropriate that the torsion test procedure be adopted as a standard method to obtain the shear strength values, especially in light of its inclusion as a method to obtain shear modulus.

### 3.3 MECHANISM UNDER TORSIONAL LOADING

All test specimens were fractured when tested under torsion. Samples of shorter length (1 to 2.4 m) fractured within the range of 30° per meter twist, while longer samples (2.8 m and 3.6 m) fractured within the range of 20 to 30° per meter. This amounts to a high value of total twist for long specimens. Throughout the tests, small cracking noises were heard and it was noticed that
small horizontal hair-type cracks appeared in the test samples while torque was still applied on specimens. During tests, most of the joists fractured with large bang sound and a puff of wood dust in air around the location of failure was seen. It was found that failure cracks, in many cases, were initiated within the clear wood even though a number of large knots were present in test joists.

Four different types of failure modes (viz crushing (40% of tested specimens), shear (25%), combined tension shear failure (12%) and horizontal shear failure (23%)) were observed as described below.

3.3.1 Crushing Failure
The crushing failure is defined here as a failure that occurs at the supports triggered mainly by clamps crushing the wood material. It was noticed that 40% of SP and NS specimens were fractured either at loading or reaction clamps with crushing failure mode. The main reason behind crushing of wood was because, in addition of shear stresses, the test clamps induced compressive stresses on the cross sectional area and the combined shear and compressive stresses caused small cracks in growth rings which, in turn, caused crushing failure. The cracks began in the earlywood zone in radial tangential (RT) plane and propagated along longitudinal radial (LR) plane (Figure 4).

3.3.2 Combined Shear Tension Failure
Another type of failure mode observed was the combined shear-tension failure and this occurred mostly in Sitka spruce joists. Nine out of 46 SP joists fractured with shear failure mode. The applied torque produces shear stresses and these stresses were dominant in causing this type of fracture. In the case of clear wood, the shear crack initiated from the middle of the LT plane and due to tension propagated towards, and was ended, in the LR plane. This may be because the grain angle might not be parallel to the longitudinal axis and, therefore, grains were fractured locally in tension and the failure travelled diagonally along the grain direction. It was also observed that when a crack approaches a knot it travelled around the knot rather than pass through it. Thus, this indicates that knots may provide some resistance to the shear failure. Figure 5 shows a combined shear tension failure.

3.3.3 Shear Failure
Another type of failure that occurred was the shear failure, which was also mainly seen in the Sitka spruce joists. About 18 out 46 SP joists fractured with shear failure mode. It was observed that shear stresses were the main cause of initiating the cracks for this failure mode, and that these cracks were usually started at either the top or bottom side, due to a knot, and then propagated as a diagonal crack along the long side to rupture the specimen in shear due to the knot at the other edge. This failure takes place because edge knots are usually surrounded by cross grain and this cross grain breaks locally in shear to initiate the failure (Figure 6).

3.3.4 Horizontal Shear Failure
This type of failure was only observed in Norway spruce specimens. In this type of failure, the shear cracks were usually initiated from clear wood within the LR plane and travelled parallel to the longitudinal direction towards end supports, as shown in Figure 10. 16 out of 26 Norway spruce battens fractured with horizontal shear failure mode. The term horizontal shear failure is
given here because the shear cracks ran horizontally along the length of the joists.

3.4 RELATIONSHIP BETWEEN SHEAR STRENGTH AND SHEAR MODULUS

3.4.1 Shear Strength and Shear Modulus Correlation

A linear correlation between the shear strength and $G$ of Sitka spruce and Norway spruce joists was developed, as shown in Figure 7. The R-squared values were calculated without including the outlying higher shear strength values of the Norway spruce test specimens. This is because only two higher shear strength values were obtained and this their inclusion would unduly bias the correlation of shear strength and $G$. It is thought that the slightly higher correlation for Norway spruce was obtained because most of these specimens were free of wood defects and joists failed within clear wood. The Sitka spruce specimens, on the other hand, contained more knots resulting in some specimens failing prematurely in a brittle manner. However, it was also noted in this study that knots have very little influence on $G$ and on shear strength overall. Rather, in some Sitka spruce specimens it was found that knots initiated the failure and caused a low shear strength values but had no major affect on $G$, which may weaken the correlation.

Figure 7: Linear relationship between G and the shear strength of SP and NS joists

In the test method, it was found that samples fractured within the long side where shear stresses are presumed to be maximum under applied torque (except in the cases where fracture was initiated at the support).

It was noticed that the cracks were commonly initiated within clear wood and caused shear failure, but that, in some specimens, cracks started due to shear and then propagated as a tensile failure. Support conditions were found to be important. It was noticed that testing clamps induced additional compressive stresses which lead to crushing of the wood at the supports and premature failure for some specimens. Therefore, this is important to design such testing clamps so that they minimise localised compressive stresses.

The recent draft revision of the testing standard EN408 [21] includes the torsion testing approach to obtain the shear modulus of timber. In this study it was found that both shear strength and shear modulus can be obtained from torsion tests and it is proposed that EN408 allow the torsion test to be used to determine shear strength as well. However it is also noted that the torsion testing approach requires careful application to avoid premature failure due to crushing at the specimen grips.

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CASE STUDY – ASSESSMENT AND RECONSTRUCTION PROJECT OF TECHNICAL MUSEUM’S TIMBER STRUCTURE IN ZAGREB - CROATIA

Vlatka Rajčić¹, Mislav Stepinac², Dean Čizmar³,

ABSTRACT: Case study – timber structure of Technical museum in Zagreb is presented, from the inspection and assessment phase to the reconstruction works. This timber structure represents an existing example of European engineering concept of expo-hall timber structures with large span (85m x 40 m) from the early 20 century.

KEYWORDS: case study, early 20th century, engineering concept, large span expo hall

1  INTRODUCTION

During end of 2010, 2011 and 2012, the Chair for timber structure, Faculty of Civil Engineering, University of Zagreb (Prof. Rajcic and assistants Cizmar and Stepinac) is leading the assessment and reconstruction project of timber structure of the Technical Museum in Zagreb, financed and run by City of Zagreb. Technical Museum is one of the most visited Museums in Zagreb (capital of Croatia). The museum presents a powerful and distinctive scientific and educational center in the field of Technical sciences. However, relatively few people know that the entire structure was designed and constructed as a timber structure and as such represents a rare existing example of European engineering concept of expo-halls timber structures with large span (85m x 40 m) from the early 20 century. As such, in technical terms, it is example of European architectural heritage of building in timber. Technical Museum Zagreb was founded on the model of existing large scientific and technical museums in the world, the common type; it is a complex museum of science and technology. The facility is under the protection of the Conservation Department in Zagreb, Protection of Cultural Heritage.

2  HISTORY AND GENERAL FACTS

Three building units received by the Museum were built in the 1948. (Designer: Marijan Haberle, B. Sc. architect) for the Zagreb International Fair as a timber structure and had a temporary purpose. But after the Zagreb Fair moved to its new location, these facilities were used for various social and sports activities, and later donated to the Technical Museum, which is still located there. Thus, the wooden buildings of the original temporary purpose, here in a series of 64 years, remained in constant, intensive use, with a very large number of annual visits by visitors from abroad and domestic people. Total area of museum is 44 000 m² (14 000 m² covered space).

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3 VISUAL AND NDT INSPECTION, GEOMETRY MEASUREMENT

When starting preparing for the replacement of old facades made from timber grids with glasses infill in the main exhibition hall by new one with much bigger self weight and the renovation of roof structure (end of year 2009), perceived damage and excessive strain on the respective revision spots on the timber structure are found. Also, it was concluded that only few original architectural drawings of the building are present and there was no original static proof of structural elements bearing capacity and stability as well as deformations control. Therefore it was decided to access to detailed survey of the timber structure (visual inspection, geometry and condition of structural elements survey and NDT determination of residual strength and stiffness elements of timber structures which is 64 years old, and was subjected during exploitation by high loads of permanent character.

3.1 STRUCTURAL CONCEPT

Layout of the main exhibition hall of Technical Museum is designed as a circular segment of a ring. (Figure 2). The highest elevation of the hall from the front-eastern side of the building is 19.74m. The timber structure of the highest central part of exhibition hall consists of the 13 inner truss frames. The frame columns consist of 4 single pieces forming the complex lattice column with N infill. The main frames of structure are extending in the east – west direction.

Vertical load-bearing structure - columns are designed as four or six assembled vertical beams with "N" lattice infill. Static analysis was taken to check bearing capacity and residual stiffness of structure.

Figure 4: Plan on the ground floor level with the marked sites that were planned for visual inspection and NDT in 1st phase. Afterwards whole structure was inspected

East and west facade of the building is partially glazed and partially covered with wooden paneling, while the north and south facade entirely covered with wooden paneling. Roof of the building is covered by synthetic roofing membrane based on PVC SIKAPLAN-15G. The interior space of the building consists of ground floor area with a height from floor to ceiling is 15.00m, which was partially covered gallery that runs the perimeter of external walls of the building height from the floor to the upper elevation flooring warehouse is 4.05m, and height from floor to ceiling is 10.5m.

Figure 5: Photo of the original drawing of the cross section of the main hall - roof truss as part of truss frame, roof trusses of the left and right side spaces and floor structure of the gallery

Figure 6: Drawing of the assessed structure of the central hall truss frame
Spatial stabilization of the building was done with three horizontal transverse, four longitudinal horizontal wind stabilizations and two vertical wind coupling. The horizontal cross-wind couple is designed as "K" form and set the height of the lower chord of the main roof truss, set in the external fields and in the middle field.

Figure 7: Roof structure of the central hall needs only minimal reparation

Figure 8: Decayd facade timber columns at foundations And its reparation detail

Figure 7: Roof transversal and longitudinal bracing elements

Except central hall, on the Figure 3 two side spaces could be noticed. It is side exhibition space that is circling around the central part. Structural elements in these parts are small columns from basement to gallery that support gallery floor structure which consists of the horizontal beams 36/40 cm at distance of 1, 25 m. Since all the museums depose are situated on the gallery, it has dead load which caused cracks especially in the columns but also in the beams.

Figure 8: Gallery columns with quite large cracks- reparation was made by epoxy resin, column was under big dead load of depose

3.2 FAULTS ERRORS AND DEFECTS

Inspection activities were observed severe damage to the large number of facade columns. The western part of the facade has been suffered systematically direct contact of the timber columns with foundation. Decay was caused by contact with rainwater.

Timber elements have many cracks due to moisture Change and overstressing. Some connections are either poorly executed or variations occur over time. A number of joints were observed with improper installation of fasteners so their load bearing capacity is questionable. Several extensions in the tensile zone have been made by just a few fasteners. Carpentry joints are generally in good condition but there are a number of which were initially wrongly executed.
6 part complex column which is 19.5 m height is coming from foundation to the roof truss structure. Big columns are stabilized with X form bracing made from timber diagonals. Longitudinal direction is very badly braced and suffers from large horizontal drift.

Some collisions are also poorly executed. On some main and facade columns, torsion effects are visible. Galleries structural floor system consists of the large beams 36/40 cm. Some of the beams has also problem due to high torsion bending. Bracing system on the roof structure is poorly designed so has to be strengthening by additional elements.

**3.3 NDT INSPECTION**

Moisture measured in the given positions varied from 7% to the maximum moisture content of 17.5%.

Tests by acoustic ultrasound were carried out to obtain modulus of elasticity of timber structural elements perpendicular to the fibers. Based on the results obtained (velocity of propagation of ultrasonic waves in the range $E_{90} = 602 \text{ N/mm}^2$ up to $E_{90} = 857 \text{ N/mm}^2$), data processing can be classified as a timber tree of class C40.
While assessing the structure, many defects are detected that they are not dangerous for the global bearing capacity, serviceability and structural stability, but as a local damage can lead to local instability. With prolonged use during construction, if it would not be repaired, could experience a rapid decline, especially because of the increased workload of the new wooden frames with izo-glass which is twice heavier than now has significant defects in design, and visual perceptual deformations (deflections) are the result of excessive deformation of eastern facade and have not enough stiffness to support structure. One of the reparations measures is to incorporate vertical bracing system in the same arrays as the roof K bracings are already built in. So, bracing system will incorporated in three arrays between main central hall big column.

CONCLUSION

According to the reconstruction project, works began in mid-2011 and still are going on. In the future period order all the elastic metal elements for X bracing vertical longitudinal stabilization of the building will be built in the ground 6 fields. In two fields V form bracing will be built in the fire exit arrays by using circular steel tubes. In the same fields of the gallery floor to the bottom glass walls at the top of the central part of the roof, bracing elements will be also mounted (using X form bracing). After structure lateral stabilization with bracings, it is necessary to detect all previously undiscovered columns in the ground and assess their condition and determine appropriate rehabilitation measures for performance details. The primary need is to capture and rehabilitate columns supporting the gallery. Also it is necessary to detect record and make a conclusion with any remediation measures for the beams that carry the gallery. After the closure of all elements of the ground floor to the gallery and the gallery soffits, it is planned to open the columns and beams of the high diaphragm and canopies that are placed on the level of the gallery to the roof of the central hall, recording their condition and to determine if remediation measures would be needed. In the event that the remaining funds, it is necessary to repair the roof truss elements of the central hall.

REFERENCES

RESEARCH ON EXPERIENCED ARCHITECTURAL EDUCATION METHOD OF USING TIMBER STRUCTURE

Shinsuke Kawai¹, Atsushi Tabuchi²

ABSTRACT: In the curriculum of the architectural education of Japan, the timber structure tends not to be the leading part. However, it can contribute very much to students supporting the sense of the flow of force. The architecture is closely connected to the structural planning dynamically proven. Through the workshop, this paper aims to clarify the problem to understand relating the space and the structural planning. Then, the following were clarified. Continuously to repeat the ‘knowledge, simulation and experience’, to let students realize a partial relation are necessary.

KEYWORDS: Timber structure, Architectural education, Space planning and structural planning

1 INTRODUCTION

The curriculum of the architectural education of Japan is composed of the reinforced concrete structure and the steel structure, which are considered as core structures. The timber structure tends not to be the leading part. However, the timber structure can contribute very much to students supporting the sense of the flow of force. Because by their hands, it is possible to cut, carry, and assemble the materials. This paper focuses on the ideal way of the architectural education through the timber structure, and reports the attempt and the result by it.

2 BACKGROUND AND OBJECT

The educational program of the architecture in Japan exists in the field of engineering and the dwelling study. The base of the dwelling study is a way of living science. Compared with engineering, the feature of this curriculum is that there are less students who are skilful in physics and mathematics, and there are not many classes related to the architecture. Considering the structural planning, the tendency is seen by students in the first half semester of the faculty includes the following.

#1 in the architectural design studio, scale feeling and weight feeling of structure are lacked.
#2 the whole of the structure is not adjusted.
#3 joint parts of structures such as the pin joints and the rigid joints are inapposite and not considered.
#4 in the class of building structural mechanics, the structural members are understood as a line but not as a volume.
#5 building structural mechanics is understood as a calculating formula but not as an object.

2.1 PROBLEMS

The educational program of the architecture in Japan exists in the field of engineering and the dwelling study. The base of the dwelling study is a way of living science. Compared with engineering, the feature of this curriculum is that there are less students who are skilful in physics and mathematics, and there are not many classes related to the architecture. Considering the structural planning, the tendency is seen by students in the first half semester of the faculty includes the following.

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#5 building structural mechanics is understood as a calculating formula but not as an object.

2.2 PAST STUDIES

The past research reported that it was important to see architecture, the space, the structure, and mechanics as a series of system, to feel actually, and to hold the interest [1, 2]. In addition, the proposal and the effect of the sight type education that used the model, the image material, and PC in the class, and experience type education that passed the workshop were reported [3–5].
2.3 OBJECT OF STUDY
This paper makes the student’s understanding level of the timber structure through the workshop an evaluation axis. And it aims to clarify the problem to understand relating the space planning and the structural planning.

3 METHODOLOGY
3.1 TARGETED STUDENTS AND CURRICULUM
In the department that authors belong, which is the field of the dwelling study; the 1st and the 2nd grader were targeted, because the class of the specialized instruction starts at latter term of 1st grader (Table 1, 2).

**Table 1: Curriculum**

<table>
<thead>
<tr>
<th>First term</th>
<th>1st grader subjects</th>
<th>2nd grader subjects</th>
</tr>
</thead>
<tbody>
<tr>
<td>Architectural design studio 1</td>
<td>Architectural design studio 1, Building Structural Mechanics 1</td>
<td></td>
</tr>
<tr>
<td>Outline of Environmental Design, Fundamental Mathematics 1, Fundamental Physics 1, Fundamental Chemistry 1, Fundamental Biology 1, Fundamental Earth Science 1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 2: Contents of subjects**

<table>
<thead>
<tr>
<th>Subjects</th>
<th>Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Systems</td>
<td>various structural forms, construction method</td>
</tr>
<tr>
<td>Introduction to Building Structures</td>
<td>hanging force each other, moment, simple beam, cantilever</td>
</tr>
<tr>
<td>Architectural design studio 3</td>
<td>design of house, atelier</td>
</tr>
<tr>
<td>Building Structural Mechanics 1</td>
<td>shearing force, Bending moment, quietness fixed ramen, truss, sectional stress</td>
</tr>
<tr>
<td>Architectural design studio 4</td>
<td>design of swimming pool for game, renovation in historic architecture, welfare institution</td>
</tr>
<tr>
<td>Building Structural Mechanics 2</td>
<td>stress and transformation in statically determinate and no statically determinate</td>
</tr>
<tr>
<td>Workshop</td>
<td>practice in field</td>
</tr>
</tbody>
</table>

**3.2 SCHEDULE**

The workshop started in June 2009, and was constructed in September through the meeting, the site investigation, the design, drafting, the estimate, and the order material. The questionnaire investigation was done immediately before construction, and the interview investigation was done in March after it ended at latter term (Table 3). Participants in the workshop were 21 persons (Table 4). Among those, the questionnaire investigation was done to 12 persons who participated in the short course held in advance.

In addition, the interview investigation was done to 5 persons of 2nd grader who played the role like a leader. There were 11 effective analysis candidates of the questionnaire investigation.

**3.3 CONTENTS OF MANUFACTURE AND INVESTIGATION**

The manufacture was decided as a pergola by the request from the region built. ‘Mengoushi’, the side lattice, that was a shear wall produces a structural, spatial, and design feature (Fig.1). By adopting the size defined by JIS (Japanese Industrial Standards), the side lattice was materialized as a shear wall. Moreover, almost was built...
Figure 2: Pergola and Joint parts of structures
by the students’ manual labor also including work of joint parts of structures such as the tenon and the halving joint (Fig.2). Basically, the meeting, the design, the structural mechanics calculation and the field operation were carried out with the student subject. Authors only did advice and the confirmation.

The contents of the questionnaire investigation were an understanding level of a structural method, a concern for the Joint parts of structures, an image of the size and the weight of the material, etc. About the interpretation of the pergola’s structure, the interview investigation was done to attempt the effect of the experience type education program at time when the architectural design studio and the class of structural mechanics ended.

4 CONCLUSIONS

4.1 SITUATION BEFORE WORKSHOP
Comparing the 1st and 2nd grader, as for the presence of the knowledge of a shear wall, ‘Mengoushi’ and the rigid timber frame structure, the difference of the class experience was remarkable (Fig.3). On the other hand, a structural understanding level of a shear wall, ‘Mengoushi’ and the rigid timber frame structure was low even by the 2nd grader. And few students could imagine the size and weight of a member from a drawing. After having received a structural explanation from the teacher about the manufacture, all students ultimately understood it.

4.2 ARCHITECTURAL EDUCATION PROBLEM
Each interview was held individually in 30 to 60 minutes. Questions of an interview are as follows.

Figure 3: Result of the questionnaire investigation

Q1. Please explain some structural features of the constructed pergola.
Q2. Please describe the functional role of a column and a beam. What is required in order to exhibit that function? What is a concrete method?
Q3. What is the difference between the structure of the constructed pergola and the rigid frame structure?
Q4. Was the ‘weight’ realized during construction?
Q5. While seeing or designing architecture, can you imagine the ‘weight’? Or can you imagine the flow of force?
Q6. It is assumed that many persons rode on this pergola. How does the pergola change?
Q7. How is a frame changed when its span doubles?
Q8. To which level did you study mathematics in your high school? Did you learn physics?
Q9. The results of the structural mechanics 2/Self-assessment
Q10. After the workshop, did you get interested in the structural planning?

As a result of the interview investigation, the character was clarified for every subject.

(1) Student A
Explanation of the constructed one had stopped at form expression called the shape of a lattice, and structural explanation was not given. The fundamental function of a column and a beam was understood. However, special terms were deficient in explanation. Since this student did not understand the point of the rigid frame structure correctly, she could not clarify the difference from the structure of the pergola, and judged the difference with her feeling. During construction, the ‘weight’ was realized, and after the workshop came to be imagined the ‘weight’ of the structure of architecture, and the flow of force. Deformation when vertical uniformly distributed load is applied on the upper side was understood as type A (Fig.4). When its span became larger, it was considered that the member become thick, but the marginal size of member to the whole size was not considered. Although this student had the interest to

Figure 4: The image of deformation
the structural planning and the class of structural mechanics was taken, could not understand it further and did not pass an examination.

(2) Student B

Explanation of the constructed one, a column and a beam was deficient in the concept of load, and its attention was paid only to the form which was in sight as a picture. This student was aware of her understanding being ambiguous about the rigid frame structure, also could not clarify the difference from the structure of this pergola, judged the difference with her feeling. During construction, the ‘weight’ was realized. But after the workshop she could not and would not imagine the ‘weight’ of the structure of architecture, the flow of force. Deformation when vertical uniformly distributed load was applied on the upper side was understood as type A (Fig.4). When its span became larger, it was considered that the member become thick, but the marginal size of member to the whole size was not considered. Although this student had the interest to the structural planning and the class of structural mechanics was taken, could not understand it further and did not pass an examination.

(3) Student C

This student grasped almost correctly about the structure of the constructed one, and the function of a column and a beam. This student had described the own interpretation with confidence, and judged the difference of the structure. However, the point of a judgment basis was not appropriate. During construction, the ‘weight’ was realized. After the workshop, he came to be imagined the flow of force, but could not imagine the ‘weight’ of the structure of architecture. That deformation was understood as type A (Fig.4). In the case of larger span, the whole scale was taken into consideration and a roof truss was proposed. This student was interested in structural planning or structural mechanics from before the workshop, his interest increased after the workshop. The class of structural mechanics was taken, and the examination was passed with excellent results.

(4) Student D

This student grasped almost correctly about the structure of the constructed, and tried to explain the function of a column and a beam from a viewpoint of the rigid joints. The point which was going to make the difference of structure also clear from the rigid joints could be evaluated. But the idea of moment was lacking. The ‘weight’ was realized, and after workshop came to be imagined the ‘weight’ of the structure of architecture, the flow of force. Deformation when vertical uniformly distributed load was applied on the upper side was answered that it was type B (Fig.4). In the case of larger span, it was considered that the member become thick, but the marginal size of member to the whole size was not considered. Although this student had the interest to the structural planning, but the class of structural mechanics was not taken. Structural mechanics was not recognized as a practical method.

(5) Student E

This student grasped almost correctly about the structure of the constructed, and tried to explain the function of a column and a beam from a viewpoint of the rigid joints. However, the rigid joints were mixed with metal connector. Since this student had the ambiguous basis of the difference of structure, she judged with her feeling. During construction, the ‘weight’ was realized. But after the workshop she could not imagine the ‘weight’ of the structure of architecture, the flow of force. That deformation was understood as type C (Fig.4). This could be said to be being unable to understand the

**Table 5: Result of the interview investigation**

<table>
<thead>
<tr>
<th>The interview contents</th>
<th>Student A</th>
<th>Student B</th>
<th>Student C</th>
<th>Student D</th>
<th>Student E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q1. Please explain the structure of the constructed pergola?</td>
<td>Impossible</td>
<td>Impossible</td>
<td>Possible/exact</td>
<td>Possible/almost</td>
<td>Possible/almost</td>
</tr>
<tr>
<td>Q2. Please describe the functional role of a column and a beam. What is required in order to exhibit that function? What is a concrete method?</td>
<td>Possible/basic</td>
<td>Impossible</td>
<td>Possible/basic</td>
<td>Possible/basic</td>
<td>Possible/basic</td>
</tr>
<tr>
<td>Q3. What is the difference between the structure of the constructed pergola and the rigid frame structure?</td>
<td>Feeling judgment Mistaken reason</td>
<td>Feeling judgment No reason</td>
<td>Confidently judgment Mistaken reason</td>
<td>Confidently judgment Nearly reason</td>
<td>Feeling judgment Reason</td>
</tr>
<tr>
<td>Q4. Was ‘weight’ realized during construction?</td>
<td>Did</td>
<td>Did</td>
<td>Did</td>
<td>Did</td>
<td>Did</td>
</tr>
<tr>
<td>Q5. While seeing or designing architecture, can you imagine the ‘weight’? Or can you imagine the flow of force?</td>
<td>Possible</td>
<td>Impossible</td>
<td>Impossible</td>
<td>Possible</td>
<td>Impossible</td>
</tr>
<tr>
<td>Q6. It is assumed that many persons rode on the created pergola. How does the pergola change?</td>
<td>Understanding</td>
<td>Understanding</td>
<td>Understanding</td>
<td>Not understanding</td>
<td>Not understanding</td>
</tr>
<tr>
<td>Q7. How is a frame changed when span doubles?</td>
<td>Easy idea</td>
<td>Easy idea</td>
<td>Suitable idea</td>
<td>Easy idea</td>
<td>Easy idea</td>
</tr>
<tr>
<td>Q8. To which level did you study mathematics in your high school? Did you learn physics?</td>
<td>Arts</td>
<td>No study</td>
<td>Sciences</td>
<td>Study</td>
<td>Sciences</td>
</tr>
<tr>
<td>Q9. The results of the structural mechanics 2 /Self-assessment</td>
<td>Study</td>
<td>Fail</td>
<td>Study</td>
<td>Fail</td>
<td>Study Pass/Excellent</td>
</tr>
<tr>
<td>Q10. After a workshop, did you get interested in the structural planning?</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes From before</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>
bending moment. In the case of larger span, she had only an idea to place a column between its larger span. The practical skills into which the present condition was developed are scarce.

This student had the interest to the structural planning and the class of structural mechanics was taken, and the examination was passed with minimum results. Although she came to have the interest to structure, it did not lead to the interest to structural mechanics.

4.3 ARCHITECTURAL EDUCATION PROBLEM

The result by an experienced architectural education method was described below.

(1) The structure was able to be actually felt by the object with the weight feeling and the volume feeling.

(2) Students came to be interested in the structural planning.

On the other hand, the following points became clear from the process of verification of these educational methods.

(3) The interest of the structure planning was not directly connected to the interest and skill level to structural mechanics.

(4) It was another to realize weight etc. as an object and to understand a phenomenon logically.

(5) Ambiguous structural understanding did not lead to intellectual improvement. The points of structural understanding were the rigid joints and the pin joints. Especially the bending moment is hard to be understood.

(6) About regarding architecture, the space planning, the structural planning, and structural mechanics as a series of systems, it could be said that the consciousness of relation is very shallow.

(7) The cause that there were students who could not connect the space planning and the structural planning immediately was that knowledge and the experience would tend to be forgotten in six months from the workshop.

From these results, for the architectural education that connects to the space planning to the structural planning, the followings are necessary.

(a) Continuously and gradual to repeat the ‘knowledge’ of structural mechanics, the ‘simulation’ of the architectural design studio, and the ‘experience’ of the experience type education program.

(b) At the stage of the 2nd grader, to let students realize a partial connection with their own interests.

REFERENCES


