Finite-Element Simulations of the Influence of Cracks on the Strength of Glulam Beams

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Abstract:
The importance of the number, size and location of cracks and how these conditions influence the mechanical properties of glulam beams have been investigated in this thesis. This study in fracture analyses and crack simulation of wood will compliment previous studies, which makes this investigation important. The investigation was made theoretically with finite-element simulations using the commercial software ABAQUS. The simulations were then compared with experimental results in order to verify the results. The shear strengths and failures of the glulam beams were studied. In addition, attention was given to the initiation of the crack propagation and the failure of the structure. Experimental results were obtained from the experiments conducted by SP Trätek, Skellefteå, Sweden. In conclusion, the finite-element model indicated some consistency to the experimental results especially with deeper cracks on one side.

Keywords:
Crack, FEM, Glulam beams, Shear strength
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A Guide to Engineered Wood Products, Form No. C800B, pages 4-9
Glulam Product Guide, Form No. EWS X440D, figure 5 - page 21
Glulam laminated Beam Design Tables, Form No. EWS S475H, Appendix A, page 28
Owner’s Guide to Understanding Checks in Glued Laminated Timber, Form No. EWS F450, page 4 & 6
Evaluation of Check Size in Glued Laminated Timber Beams, Form No. EWS R475E, pages 2, 4-5
Finally, I wish to express my boundless appreciation to my family for their encouragement and invaluable support.

Skellefteå, September 2011

Erhan Saracoglu
# Contents

1 Notation 6

2 Introduction 8

3 Literature Study on Glulam beams 10
   3.1 General Structure of Wood 10
   3.2 Engineered Wood Products 11
      3.2.1 Plywood 11
      3.2.2 Oriented Strand Board (OSB) 12
      3.2.3 I-Joists 12
      3.2.4 Structural Composite Lumber (SCL) 13
      3.2.5 Rim Boards 14
   3.3 Glued Laminated Timber (Glulam) 14
      3.3.1 History of Glued Laminated Timber 14
      3.3.2 Life Cycle of Glulam 15
      3.3.3 Manufacturing Process 16
      3.3.4 Glulam Beams Design Regulations 18
      3.3.5 Beam Systems 20
      3.3.6 Design Specifications and Methods According to Eurocode 5 21
      3.3.7 Design Diagrams 25
   3.4 Cracking and Checking in Glulam Beams 28
      3.4.1 Identifying Cracks 29
      3.4.2 Significance of Cracks in Glulam Structures According to APA 30

4 Experimental Work 37
   4.1 Background 37
   4.2 Experimental Model and Setup 38
   4.3 Test Specimens 40
   4.4 Test Results 44
      4.4.1 Beam Type 1 44
      4.4.2 Beam Type 2 46
      4.4.3 Beam Type 3 49
      4.4.4 Beam Type 4 50

5 Simulation 53
   5.1 Assumptions 54
5.2 The method of modelling and simulation of 3D timber structures with cracks
5.3 Simulation Results
  5.3.1 Beam Type 2
  5.3.2 Beam Type 3
  5.3.3 Beam Type 4
6  Conclusions and Future Work
7  References
1 Notation

\begin{align*}
A & \quad \text{Area} \\
E & \quad \text{Young’s modulus} \\
F & \quad \text{Force} \\
G & \quad \text{Shear modulus} \\
M & \quad \text{Moment} \\
P & \quad \text{Pressure} \\
R & \quad \text{Reaction force} \\
V & \quad \text{Shear force} \\
W & \quad \text{Section modulus} \\
b & \quad \text{Width} \\
c & \quad \text{Characteristic coefficient} \\
h & \quad \text{Height} \\
l & \quad \text{Length} \\
k & \quad \text{Coefficient} \\
s & \quad \text{Standard deviation} \\
q & \quad \text{Distributed load} \\
t & \quad \text{Time} \\
w & \quad \text{Deflection} \\
\tau & \quad \text{Shear Stress} \\
\sigma & \quad \text{Stress} \\
\sigma_{c,90,d} & \quad \text{Design bearing strength perpendicular to grain} \\
\sigma_m & \quad \text{Bending stress} \\
f_{c,90,d} & \quad \text{Design compression strength perpendicular to grain} \\
f_k & \quad \text{Characteristic value} \\
f_m & \quad \text{Mean value}
\end{align*}
\( f_{m,d} \) Design bending strength
\( f_{v,d} \) Design shear strength
\( k_{cr} \) Crack factor for shear resistance
\( k_{crit} \) Modification factor used for lateral buckling
\( l_{ef} \) Effective length of the support

**Abbreviations**

EC 5 Eurocode 5
OSB Oriented Strand Board
SCL Structural Composite Lumber
ANSI American National Standard Institute
SCD Side Crack Depth
ECD End Crack Depth
FEA Finite Element Analyses
FEM Finite Element Method
2 Introduction

This thesis work is a part of a research project performed by co-operation between SP Trätek and the Luleå University of Technology in Skellefteå. The main objective of this project is to understand the mechanical behaviour of glulam beams with certain types of cracks and under static load conditions. In addition, the objective is to determine the critical load conditions and create guidelines and instructions for inspectors.

In this work, the simulation of glulam beams with certain types of crack is studied. The main aim of this work is to create a simulation model of glulam beams with particular types and locations of cracks using the finite-element method. Bending strength, shear strength and the initiation of crack propagation will also be investigated. The correlation and validation between the finite-element model and the experimental results will be evaluated. The shear strength of glulam beams is still under discussion in Europe. The shear capacity of glulam beams can be found in the standard regulations of Eurocode 5 [1].

One of the important published researches in this field is Gustafsson P.J., Emilsson E., Crocetti R. And Ormarsson S.: “Provningar av limträs skjuvhållfasthet hösten 2009”. In their report, they investigated the shear strength of glulam beams with rectangular cross section and I-cross section. In this study, modified EN408 test setup was used [2].

Another published investigation is Pousette A.: “Träbalkar med sprickor – förstudie om bärförmåga och hållbarhet 2006”. The aim of the research was to investigate the bearing capacity and durability of timber structures that were affected by crack formation and influencing factors. The report also covered the FEM simulation of crack formation [3].

The master dissertation of Sundström T.: “Shear resistance of gluelam beams under varying humidity conditions 2010” provide comprehensive information about influence of humidity variation in the large size of glulam structures. However the moisture content variation is out of scope for this study. Nevertheless the FEM simulation part of his report provided detailed explanations about modeling the timber structures in ABAQUS [4].

In the literature study of this report, the general material properties of wood, a brief history of glulam beams, the mechanical properties of glulam
beams, the design regulations for glulam beams in Europe, especially in Sweden, and abroad will be given. A description of published researches in this field will also be given.

In the simulation part, finite-element modelling of the glulam beams based on published design parameters with specific cracks of different sizes located in different areas will be studied, and the deflections, shear stresses, bending stresses and the initiation of crack propagation will be interpreted.

The experimental part includes 4 types of experimental group of glulam beams, which have different characteristics. Each group contains 5 glulam beams of the same size and with the same crack conditions. Shear strength and bending strength was tested. The experimental setup used was based on Anderson and Oden (2009) Master Thesis.

The comparison between experimental data and the simulations is given. Modification of the simulation based on the experimental feedback was studied.

Finally, a conclusion on the significance of the number, size and the location of cracks with respect to beam deformation and strength is provided.
3 Literature Study on Glulam beams

3.1 General Structure of Wood

The inner structure of wood is composed of wood cells which consist of cellulose, hemicelluloses, lignin and relatively lesser amounts of extractives. The amounts of all these materials can vary, thus giving certain characteristic properties to the wood [5].

A tree has two growth directions and these are in diameter and height. The growth occurs in annual cycles and causes growth rings in the tree. In every growth season a new growth ring is added to the previous one, and thus the tree can grow in diameter. One exceptional case is the shoot, which allows the tree to grow vertically (i.e. in height).

In one growth season, when a single growth ring occurs, two different parts of the structure can be observed. The first part is called earlywood which grows in the springtime and early summer and has a lower density, and the second part is called latewood which grows in the late summer and it is denser than the earlywood. In the winter and autumn seasons there is no growth. The growth rings create three main parts of the inner structure of the wood, and these are called juvenile wood heart-wood and sapwood. The juvenile wood consists of 10 to 20 growth rings close to the pith. These growth rings can shrink more than the rest of the growth rings, however they have a lower strength. The sapwood can be seen on outer part of the stem, and surrounds the heartwood. It has the function of conduction and storage of the tree’s needs. When the inner part of the stem gets older, it loses these functions and becomes heartwood [6].

Wood is an anisotropic material because wood cells exhibit different strength properties in different directions and it includes many defects (i.e. knots) which cause difficulties in its simulation. The most reasonable way to model a timber structure is to assume that it is an orthotropic material, which means it has three orthogonal directions to define symmetry planes. This topic will be emphasized in later chapters.
3.2 Engineered Wood Products

For construction purposes, structural timber with rectangular cross-sections are produced at sawmills.

Engineered wood products are glued building products. They are made by gluing together lumber, veneers, wood strands or other forms of wood fibres to larger structural units. Some engineered wood products are presented in Chapters 3.2.1 - 3.2.5. Glulam is described in more detailed in Chapter 3.3.

3.2.1 Plywood

Plywood is made from laminated veneer sheets and bonded under high heat and pressure with moisture-resistance adhesives. Plywood is one of the most common engineered wood products and it is used for outside floor, roof sheathing, furniture, industrial containers, pallets, etc. [7].
3.2.2 **Oriented Strand Board (OSB)**

OSB is made of rectangular shaped strands, flakes or wafers sliced from wood that are layered in specific orientations and bonded together with moisture-resistant adhesives. Common usage of OSB is in flooring applications, furniture, industrial containers, etc. [7].

![Figures 3.2 and 3.3.](image_url)

3.2.3 **I-Joists**

I-joists are engineered wood products which are made from sawn or composite lumber flanges and OSB webs and bonded together with adhesives. I-joists have resistance to bending stresses while the web provides good shear stress performance. Common usage of I-Joists is in floor applications, roof framing, residential construction, etc. [7].
3.2.4 Structural Composite Lumber (SCL)

SCL is made from layering dried and graded wood veneers, strands or flakes with moisture-resistant adhesives into blocks of material, which are called billets that are improved in a heated press. Structural composite lumbers are commonly used in headers, beams, rafters, studs, columns, etc. [7]. SCL includes laminated veneer lumber (LVL) that has greater strength and stiffness than the corresponding dimensions of structural timber.
3.2.5 Rim Boards

Rim board is a special design engineered wood product used in order to complete the I-Joists applications. Rim board can be manufactured by plywood, OSB, glulam or SCL. The application of the rim board is in flooring systems [7].

![Figure 3.6. Rim board [7].](image)

Detailed explanation of glued laminated timber will be given in the following sections.

3.3 Glued Laminated Timber (Glulam)

In this section the brief history, life cycle, characteristic and material properties, manufacturing process, types of glulam beam products, design regulations of glulam beams abroad and in Europe (especially in Sweden) will be given.

3.3.1 History of Glued Laminated Timber

Glued laminated timber products are considered one of the most effective and renewable approaches to wood building products. It is an engineered product manufactured to fulfil the most requested needs for structural demands. Glued laminated timber products have been used for around one hundred years. Based on information from the APA - The Engineered
Wood Association, the first usage of glulam was in the early 1890s, and the patent for the product was taken out in Switzerland. Alternatively, based on the Glulam Handbook, the first glulam patents were taken out in Germany around 1900. Hetzer Binder, with his patent of 1906, was the starting point of modern glulam technology. The USDA Forest Product Laboratory in Madison, Wisconsin, USA is one of the first glulam structures, it was erected in 1934 and is still in service today. Also in Europe some of the oldest glulam structures can be found in the central railway stations in Stockholm, Gothenburg and Malmö, which were built in the 1920s. The glulam industry showed signs of significant improvement when fully water-resistant phenol-resorcinol adhesives started to be used in 1942. With this type of adhesive the glulam products could be used in exterior environments without concerns for deformation along the glue lines for a long time [8, 9].

3.3.2 Life Cycle of Glulam

The raw material of glulam wood products is spruce or pine and synthetic glue. The only non-renewable material in the glulam is the glue, which damages the environment. Fortunately, if we compare the amount of glue and per unit volume of glulam, it can be seen that the amount of glue is fortunately very small (less than 1% weight) which means that the negative effect on nature can be considered negligible.

Glulam products need to be supplied at 12% moisture content, so there is a need for a drying process. The large part of the drying process is fuelled with sawdust and other bi-products. This type of fuelling also reduces the use of electricity.

If the strength class and loading history of the glulam product is known it can be re-used. Also the environmental effect of glulam products during their life cycle can be controlled by the producer [9].
Figure 3.7. Timber life cycle (Re-drawn from Glulam Handbook) [9].

3.3.3 Manufacturing Process

The raw material of glulam is specifically selected timber that is chosen based on its strength characteristics. In Nordic countries spruce or pine are usually selected.

The moisture content of the final product should be 8-15%. Maintaining a 5% moisture content difference between lamellas will provide balance to the structure and prevent splitting at gluelines in the long term [9].

The manufacturing process can be seen in the Figure 3.8;

Figure 3.8. Manufacturing process of Glulam [9].

The lay-up of the glulam can be of two types; homogeneous (balanced) and combined (unbalanced). Homogeneous glulam can be formed with the
same (approximately) strength class laminates. In order to improve its strength characteristics, in the compression and tension lamella where the highest stresses can be observed, a higher quality of timber can be used in combined type of glulam beams.

![Standard Beam Layup](image)

Figure 3.9. American standard glulam beam lay-up (left), Swedish Combined (Unbalanced) glulam beam lay-up (right) [9, 10].

One of the most important processes is finger jointing. In the finger jointing process the laminates are cut to a certain length and placed on the top of each other. Finger jointing allows the production of glulams for long span applications. Hardening of the finger joint glue must be waited before the flat sides of the laminates are bonded together with adhesive.

In the next step, specific pressure is applied to the laminates in the gluing benches. This process is highly dependent on the glue type and room temperature. The product is then planed by planing machine to meet the required surface finish. For the final work, the glulam is drilled for the connectors.

All glulam products produced in European countries should be based on the EN 386 standard regulation. Certified glulam products are marked with an “L-mark” in Nordic countries, the “trident mark” is compulsory for glulam in Sweden, and also “Gütezeichen” is required in Germany [9]. In addition, CE-marking of glulam is based on the standard EN 14080.
3.3.4 Glulam Beams Design Regulations

The design properties of glulam beams are variable across Europe and abroad. In U.S, members of glulam beams are selected based on their bending stress characteristics. For instance, a 24F notation on the glulam member shows that 2400 psi is the allowable bending stress for this member. It is also possible to design a glulam member with higher bending stress resistance by varying the species, percentages and grade of better quality lumber in the beam layup. Furthermore, the second parameter of the notation indicates that the lumber has been graded mechanically or visually. For instance, a 24F-V4 notation shows that the layup of the glulam beam uses visually graded lumber (“E” notation is used for mechanically graded lumber) [8].

In Nordic countries, glulam members are selected based on their strength and stiffness characteristics. In order to determine these strength classes, a large number of test specimens were tested in laboratories and statistical interpretation was carried out to establish the strength classes of glulam and timber structures. The characteristic strength values of glulam were determined from a frequency diagram (normal distribution diagram)

![Figure 3.10. Example of normal distribution diagram [9].](image)

In this procedure, the breaking strength is assumed to be normally distributed, and as a result the characteristic value $f_k$ can be determined from the Equation (3).
\[ f_k = f_m - (c \times s) \]  \hspace{1cm} (3.0)

Where, \( f_m \) is the mean value, \( c \) is the characteristic coefficient and \( s \) is the standard deviation. In this case the characteristic coefficient is taken as 1.75.

Characteristic stiffness (elasticity module, shear module) can be calculated using the same procedure.

In Nordic countries glulam structures are manufactured according to European Standards and the most common class is called CE L40C.

In accordance with Swedish standards, straight glulam components with a rectangular cross-section are generally made from 45 mm thick laminates. The Table 3.1 shows the widths of glulam structures based on the older Swedish standard SS 23 27 21 [9].

**Table 3.1. Glulam cross-sections (mm) in accordance with older Swedish standards [9].**

<table>
<thead>
<tr>
<th>Class</th>
<th>Width (mm)</th>
<th>Structural Timber</th>
<th>L-marksed glulam</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-0</td>
<td>42-90</td>
<td>56-90</td>
<td>66-90</td>
</tr>
<tr>
<td>L-1</td>
<td>42-135</td>
<td>56-135</td>
<td>66-135</td>
</tr>
<tr>
<td>L-2</td>
<td>42-180</td>
<td>56-180</td>
<td>66-180</td>
</tr>
<tr>
<td>L-3</td>
<td>42-225</td>
<td>56-225</td>
<td>66-225</td>
</tr>
<tr>
<td>L-4</td>
<td>42-270</td>
<td>56-270</td>
<td>66-270</td>
</tr>
<tr>
<td>L-5</td>
<td>42-315</td>
<td>56-315</td>
<td>66-315</td>
</tr>
<tr>
<td>L-6</td>
<td>42-360</td>
<td>56-360</td>
<td>66-360</td>
</tr>
<tr>
<td>L-7</td>
<td>42-405</td>
<td>56-405</td>
<td>66-405</td>
</tr>
<tr>
<td>L-8</td>
<td>42-450</td>
<td>56-450</td>
<td>66-450</td>
</tr>
<tr>
<td>L-9</td>
<td>42-495</td>
<td>56-495</td>
<td>66-495</td>
</tr>
<tr>
<td>L-10</td>
<td>42-540</td>
<td>56-540</td>
<td>66-540</td>
</tr>
<tr>
<td>L-11</td>
<td>42-585</td>
<td>56-585</td>
<td>66-585</td>
</tr>
<tr>
<td>L-12</td>
<td>42-630</td>
<td>56-630</td>
<td>66-630</td>
</tr>
<tr>
<td>L-13</td>
<td>42-675</td>
<td>56-675</td>
<td>66-675</td>
</tr>
</tbody>
</table>

The Table 3.2 contains some of section properties and load carrying requirements of glulam members in accordance with the American National Standards Institute (ANSI) standard A190.1.
3.3.5 Beam Systems

The most common usage of glulam structures is in beam systems which are freely supported at each end by columns. This type of glulam structure is suitable for use in both residential and non-residential applications. However, making holes and notches in the glulam beam is a common issue. Normally a beam carries the lateral forces with its whole cross-section, and these forces are higher at the supports than in the middle of the beam. Therefore, holes and notches should be placed in the middle of the cross-section where the shear forces are at a minimum.

Beams with more supports have an advantage to beams with only two supports, one at each end. In order to achieve the most suitable distribution of moments, the continuous beams can be designed in accordance with the Gerber System [9].

Table 3.2. Glulam section properties and load carrying requirements [11].
3.3.6 Design Specifications and Methods According to Eurocode 5

The main aim of the design rules for load-bearing building structures is to avoid the failure of the structure, with concern for human safety. Moreover these design rules should ensure that the structure works satisfactorily. Glulam structures are designed and manufactured in accordance with the European Community’s Eurocode 5: Design of Timber Structures. In EC5 timber structures should be designed based on the partial coefficient method which consists of checking the structural systems in two states; the ultimate limit state and the serviceability limit state. The ultimate limit state is based on checking the construction against failure. In the serviceability limit state, the construction is checked for deformations and assessed for any risk of it losing its functions.

The partial coefficient method is based on the assumption that the parameters are not stochastic (non-deterministic) but can be given definite values. These parameters, values of loads or strength, etc., are however calculated based on probability theory. The partial coefficient method is internationally accepted and used in EC5.

The method is based on different partial coefficients which can be called safety factors that consider the uncertainty affecting the calculations. For
instance; loading assumptions \((g_f)\), calculated loading capacity \((g_m)\), and consequences of failure \((g_n)\). Furthermore, the application of the partial coefficient method can vary from one country to another. For example, in Nordic countries safety classes are used in order to treat the various consequences of failure, which is not mentioned in EC5.

In EC 5 there are 3 service classes to define the timber structures depending on their moisture content. The moisture content has a considerable influence on the strength and stiffness of the timber. It is known that dry timber is stronger and stiffer than moist timber. All service classes are characterised by an environment, which is based on relative moisture content. In service class 1, the moisture content of the environment exceeds 65% for a few weeks of the year and never reaches 80%. In service class 2, the moisture content of the environment exceeds 80% for a few weeks per year. In the service class 3, the moisture content of the environment is greater than service class 2.

The design values of the moments and forces for the glulam beams are calculated based on linear elasticity theory, with load combinations which are given in the Eurocode [9].

- **Internal Forces and Reaction Forces**

Support reactions and internal forces can be easily calculated with aid of a computer. For manual calculations, certain tables are used to calculate these forces;

\[
M = k \cdot q \cdot l^2 \quad (3.1)
\]

\[
R = k \cdot q \cdot l \quad (3.2)
\]

\[
w = \frac{k \cdot q \cdot l^4}{100E_l} \quad \text{where} \ x = k \cdot l \quad (3.3)
\]

Where;

\(x\) = is distance from the support.

\(k\) = Coefficient

\(q\) = Distributed load

\(l\) = Length
Table 3.3. Coefficient $k$ for moment, reaction at support and deflection. Applies to unjointed beam of a constant depth [9].

<table>
<thead>
<tr>
<th>Moment</th>
<th>Reaction at support</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{AB}$</td>
<td>$M_{BC}$</td>
<td>$M_{CD}$</td>
</tr>
<tr>
<td>0.070</td>
<td>-0.125</td>
<td>0.070</td>
</tr>
<tr>
<td>0.096</td>
<td>-0.063</td>
<td>-0.025</td>
</tr>
</tbody>
</table>

Table 3.4 Coefficient $k$ for moment, reaction at support and deflection. Applies to Gerber System with a constant depth [9].

<table>
<thead>
<tr>
<th>Moment</th>
<th>Reaction at support</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{AB}$</td>
<td>$M_{BC}$</td>
<td>$M_{CD}$</td>
</tr>
<tr>
<td>0.080</td>
<td>-0.100</td>
<td>0.025</td>
</tr>
<tr>
<td>0.101</td>
<td>-0.050</td>
<td>-0.050</td>
</tr>
<tr>
<td>-0.025</td>
<td>-0.050</td>
<td>0.075</td>
</tr>
<tr>
<td>-0.072</td>
<td>-0.117</td>
<td>0.053</td>
</tr>
</tbody>
</table>
• **Shear Stress**

In order to calculate the shear stress, the design condition is applied in *Equation (3.4)*;

\[
\tau = \frac{1.5V}{bh} \leq k_{cr} \cdot f_v
\]  

(3.4)

Where;

- \(V\) = shear force
- \(f_v\) = design shear strength
- \(k_{cr}\) = reduction factor with regards to cracks or notches in the beam
- \(b\) = beam width.

• **Bearing Strength**

The *Equation (3.5)* is applied;

\[
\sigma_{c,90,d} = \frac{R}{b l_{ef}} \leq k_{c,90} f_{c,90,d}
\]  

(3.5)

Where;

- \(l_{ef}\) = effective length of the support in accordance with current regulation
- \(R\) = design reaction at support
- \(f_{c,90,d}\) = design compression strength perpendicular to the grain according to current regulations.
- \(k_{c,90}\) = bearing factor.
- \(b\) = beam width.
- \(\sigma_{c,90,d}\) = design compressive stress perpendicular to grain.

• **Bending and Buckling**

The *Equation (3.6)* is applied;

\[
\sigma_{m,d} = \frac{6M}{bh^2} \leq k_{crit} f_{m,d}
\]  

(3.6)
Where;
M = design moment
\( f_{md} \) = design bending strength
\( k_{crit} \) = modification factor with regards to buckling.
b = beam width
h = beam height

**Deflection**

This design condition is expressed as the Equation (3.7);

\[
W_s = k_1 \frac{Ml^2}{Ebh^3} + k_2 \frac{M}{Gb} \leq W_R
\]  (3.7)

Where;
\( k_1 \) and \( k_2 \) = factors whose values depend on the support conditions and on loading
M = design moment
\( W_s \) and \( W_R \) = calculated and maximum permitted deflections
E, G = design values for material properties.
b = beam width
h = beam height

**3.3.7 Design Diagrams**

It is possible to design straight beams which have two supports at the end of the beam using design diagrams. The starting point of the design is to calculate the load per metre of the beam (kN/m) and this value is divided by the width of the beam (m), and the calculated design value (N/mm²) of the bending strength or elasticity modulus in bending. The design value is calculated considering the service classes, duration of load and safety classes for specific cases.
Figure 3.12. Design diagram for beams supported at each end and considering carrying capacity. Curve $A =$ straight beam, curve $B =$ double tapered or single tapered beam 1:20, curve $C =$ double tapered or single tapered beam 1:16$^\text{[9]}$. 
Figure 3.13. Design diagram for beams supported at each end considering deflection in the serviceability limit state. Maximum deflection $w = l/200$. For other conditions $w = l/n$, multiply load $q$ by $n/200$. Curve $A$ = straight beam, curve $B$ = double tapered beam 1:20, curve $C$ = double tapered beam 1:16, curve $D$ = single tapered beam 1:20, curve $E$ = single tapered beam 1:16[9].
Following is guidance on how to use these diagrams for designing glulam beams;

1. Determine the design value of the load \( q_d \) and load combination.
2. Determine the span.
3. Choose the beam type (straight, double tapered, single tapered).
4. Choose reasonable value for the beam depth and width as a starter value.
5. Determine the design bending strength \( f_{md} \) considering chosen depth, safety class, service class and duration of the shortest-time loading in the loading combination.
6. Choose the starting value of \( q_d/b/f_{md} \) paying attention to units (kN/m/m/N/mm²) and find the value on the horizontal axis in Figure 3.12.
7. Draw a vertical line through this value and mark the line that crosses the curve corresponding to the chosen type of beam (A, B or C).
8. Draw a horizontal line through this marked point and read that value of the quotient \( h/l \) where the line crosses the vertical axis.
9. Calculate the beam depth and compare with the starting value.
10. If necessary, correct the start value, and then repeat points 6 to 9 until the calculated beam depth agrees with the assumed one.
11. Determine the deflection in same way as the determination of the beam depth. In Figure 3.13 the mid-span deflection should not exceed 1/200 of the span [9].

### 3.4 Cracking and Checking in Glulam Beams

Glulam beams are formed by individual laminations of wood [9]. Among all these laminations the bottom and top laminates are usually the strongest due to the fact that this is where the maximum tensile and compressive stresses occur. Especially in outdoor applications where glulam structures are exposed to environmental conditions that over time can cause cracks and checks in the glulam structures. These cracks reduce the strength of the glulam beam and can cause the failure of the structure. A detailed explanation of a crack is; the outer fibres of the wood loose moisture and tend to shrink due to the surrounding atmosphere, however the inner part of the wood has a lower moisture content loss in contrast with outer part of the wood structure. Thus rapid drying processes increase the difference in moisture content between the inner part and the outer fibres and causes
separation of fibres. At this point the moisture content of the wood gains importance. However the moisture content will be assumed to be constant in this report, and thus this subject is out of the scope of this report.

Based on the manufacturing process of glulam structures, cracking is fortunately less common and smaller than normal sawn timber due to the controlled drying procedure. The controlled drying procedure provides comprehensive monitoring of the structure. In normal conditions, glulam structures are manufactured at 12 percent moisture content, which is lower than outdoor conditions and stabilises the moisture content in the glulam structure and prevents possible serious cracking [12].

3.4.1 Identifying Cracks

It can be said that the cracking depends on two factors; the first one is seasonal cracking which occurs naturally with the seasons and due to exposure to environmental conditions (rain, snow, sunlight, etc.), most of them occur near the glue lines. In some cases delamination may cause confusion about cracking. It should be known that delamination occurs when the glue bonds are not sufficient for the moisture cycle and thus the glulam members separate at the glue lines. In this case it can also be noticed that the separation line colour is dark because of the phenol-resorcinol adhesives. The second factor is rapid drying cracks that occur when the glulam laminates are prepared for the manufacturing process in the drying chambers. In this procedure if the lamella is dried too rapidly and in an uncontrolled way thus the outer surface of the timber tends to shrink faster than the inside part. In order to prevent this type of cracking the drying procedure should be controlled [12].
3.4.2 Significance of Cracks in Glulam Structures According to APA

As mentioned previously, cracks may occur in the glulam structure because of natural reasons. Fortunately the cracks are not a structural concern if they have not developed full length of the structure. Studies have shown that partial cracks in the glulam structure do not have significant effect on the load carrying capacity [13]. On the other hand, it is possible to find different regulations regarding cracks in glulam beams from country to country because the influence of cracks on glulam beam strength is still under discussion and investigation. Another important point is crack depth. The crack depth can vary during propagation and has the potential to influence the strength of the beam.

Cracks are commonly observed in bending members, for instance beams and rafters, on girders on the face of the bottom laminate, on the sides of the laminates and also at the end of the members. Figure 3.15 illustrates an example of side and end cracks.
According to APA - The Engineered Wood Association, the side cracks can be considered negligible if they are no greater than one-third of the beam width and one-third of the beam length as they have no significant influence on structural performance. However if the crack develops along the full length of the structure then it has an influence on the shear capacity and bending stress [13]. In this report, the influence of full-length side cracks on the glulam beams will be studied with experiments and simulations and the results will be discussed. Particularly in the simulation part, crack evaluation and propagation will be carried out.

The shear-critical zone location is at the end of the glulam beam. Based on APA – The Engineered Wood Association, if the cracks occur outside the shear-critical zone then they are not significant, however if they occurred inside the zone then it affects the horizontal shear capacity. Commonly, the shear-critical zone is defined as the locations at the both ends of the beam within a distance from each end and equal to three times of the height and within the middle and half height of the beam. The question regarding how the shear stress will change due to the cracks is still being studied. Based on the ASTM standards, the horizontal shear stress capacity of glulam beams is calculated using the principles of ASTM D3737 and ASTM D245, and using data given in ASTM D2555. But tests show that the horizontal shear stress varies with cracks [13].
Based on practical experiences and laboratory tests and studies, it has been shown that if the cracks are no more than 15 percent of the beam depth, regardless of the location, the shear capacity will not be reduced. According to APA - The Engineered Wood Association, the glulam industry developed certain tables and equations for allowable crack sizes in beams. The Table 3.5 and Table 3.6 illustrate the allowable crack sizes in the beam including the shear-critical zone and outside the shear-critical zone.

**Table 3.5. Allowable crack size in the shear-critical zone [13].**
Table 3.6. Allowable crack size outside the shear-critical zone [13].

<table>
<thead>
<tr>
<th>Beam Width in Inches</th>
<th>y/h</th>
<th>Allowable Side Checks (Depth) in Inches</th>
<th>Allowable End Checks (Length) in Inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1/2</td>
<td>3/4</td>
<td>1/4</td>
<td>2/3/8</td>
</tr>
<tr>
<td>3-3/1/8</td>
<td>1</td>
<td>1/2</td>
<td>2/3/8</td>
</tr>
<tr>
<td>3-1/2</td>
<td>1-1/8</td>
<td>1-3/4</td>
<td>2-1/4</td>
</tr>
<tr>
<td>5-5-1/8</td>
<td>1-5/8</td>
<td>2-1/2</td>
<td>3-1/4</td>
</tr>
<tr>
<td>5-1/2</td>
<td>1-3/4</td>
<td>2-3/4</td>
<td>3-5/8</td>
</tr>
<tr>
<td>6-3/4</td>
<td>2-1/8</td>
<td>3-1/4</td>
<td>4-1/2</td>
</tr>
<tr>
<td>8-1/2, 8-3/4</td>
<td>2-3/4</td>
<td>4-1/8</td>
<td>5-3/8</td>
</tr>
<tr>
<td>10-1/2, 10-3/4</td>
<td>3-3/8</td>
<td>5-1/8</td>
<td>6-7/8</td>
</tr>
</tbody>
</table>

- Equation 1: Allowable side crack depth inside the critical-shear zone

\[ SCD_{\text{allow,inside}} = 0.15 \times b \]  

Where:

SCD = side checks (cracks) depth
b = beam width

- Equation 2: Allowable end crack length inside the critical-shear zone

\[ ECL_{\text{allow,inside}} = 0.15 \times 3 \times b \]  

Where:

ECL = end crack length

In order to calculate the allowable crack size outside the critical-shear zone theoretical and idealised shear-stress distribution diagrams of glulam beams are used. It is assumed that the allowable crack size outside the critical-shear zone can be increased proportionally.
Example:

For a beam 115x315 mm cross-section:

\[ SCD_{\text{Allow,inside}} = 0.15 \times 115 = 17.25 \, mm \]

\[ ECL_{\text{Allow,inside}} = 0.15 \times 3 \times 115 = 51.75 \, mm \]

*Figure 3.17. Theoretical and idealised shear-stress distribution in glulam beams [13].*
Equation 3: Allowable side-crack depth outside the shear critical zone:

\[ SC_{D,allow, outside} = C \times b \]  \hspace{1cm} (3.10)

Equation 4: Allowable end-crack length outside the shear-critical zone:

\[ EC_{D,allow, outside} = C \times 3 \times b \]  \hspace{1cm} (3.11)

Where:

\[ 0.15 \leq C = 3.4 \left( \frac{y}{h} \right) - 0.7 \leq 0.8 \]  \hspace{1cm} (3.12)

h= beam height.
y= distance from mid-depth of the beam.

If the measured crack size exceeds the allowable crack size the Equation (3.13) and Equation (3.14) will be applied in order to calculate the factor of reduced shear stress:
• Equation 5: For side cracks:

\[ R_{crack} = \frac{1 - \frac{a}{b}}{1 - C} \leq 1.0 \]  \hspace{1cm} (3.13)

• Equation 6: For end cracks:

\[ R_{crack} = \frac{1 - \frac{a}{3b}}{1 - C} \leq 1.0 \]  \hspace{1cm} (3.14)

Where;

a = measured crack length.
b = glulam beam width.

C = factor calculated from equations 3 or 4 when the crack is outside critical zone.

C = 0.15 when the crack is within the critical zone [13].

Example:

Cross section of the beam is 115x315 mm, crack depth is 34.5 mm

For side crack:

\[ R_{crack} = \frac{1 - \frac{34.5}{115}}{1 - 0.15} = 0.79 \leq 1.0 \]

For end crack:

\[ R_{crack} = \frac{1 - \frac{34.5}{3 \cdot 115}}{1 - 0.15} = 1.06 \geq 1.0 \]

It exceeded normal value for end crack so it needs to re-calculate of allowable end-crack.
4 Experimental Work

4.1 Background

The experiment was performed at Sp Trätek in Skellefteå in order to test and understand shear strength capacity of glulam beams with certain cracks. The significance of the cracks in the glulam beams was studied. The species of the test specimens are spruce.

Many studies have been previously conducted to estimate the shear strength of glulam beams, and some are still going on. However, there are few studies done on glulam beams with cracks. The main reason for that is undoubtedly the lack of test specimens with natural cracks. Many years must pass in order to obtain natural cracks in glulam beams.

According to Eurocode 5, the crack factor is $k_{cr} = 0.67$ [1]. The value can be chosen in the National Annex and be different in different countries. One purpose of this test was also to investigate the validity of this factor. Currently the shear strength of the glulam beam is 2.7 MPa for CEL40C class in Sweden. It is planned to change this to 3.5 MPa in future. Then, the shear strength with the crack factor will be; $0.67 \times 3.5 = 2.34 \text{ MPa}$ unless it is shown to be higher value of crack factor in Sweden. In the experimental work, the glulam beams were tested with the three-point bending test method, because this method is the closest method to real life cases. However, while testing there is a risk of bending failure instead of shear failure. It can be seen that various tests were carried out in previous studies (block tests with small specimens, three-point bending, five-point bending of full-sized beams, testing of beams with rectangular cross-section, I-section beams, etc.). In addition, the methodology of the test will affect the test results, and the shear stress can be influenced by bending stresses or normal stresses during test procedure.
4.2 Experimental Model and Setup

The experimental model was chosen based on Anderson and Oden [14] as in Figure 4.1.

![Experimental model](image)

*Figure 4.1. Experimental model redrawn from Anderson and Oden Master thesis [13].*

In the experiment, roller supports were used. The power of the bending cylinder was 210 Bar. The cylinder of the bending machine was adjusted to 6 mm/min. Under the support plate the rubbers were attached in order to keep them rigid. The displacement of the beams was measured from the head of the cylinder.
Figure 4.2. Experimental setup (front view).

Figure 4.3. Experimental setup (side view).
The necessary force for shear and bending can be estimated with the aid of the following formulae;

The shear capacity was taken as \( f_{vk} = 4 \text{ MPa} \)

\[
V = \frac{A \cdot f_{vk}}{1.5} \tag{4.1}
\]

\( V = 96.6 \text{ kN} \tag{4.2} \)

\[
F = \frac{V \cdot L}{b} = 147.7 \text{ kN} \tag{4.3}
\]

For the bending capacity;

\[
M = f_{m,k} \cdot W \tag{4.4}
\]

\[
F = \frac{f_{m,k} \cdot W \cdot L}{a \cdot b} = \frac{33 \cdot 10^6 \cdot 0.0019 \cdot 2.275}{0.788 \cdot 1.488} = 121.8 \text{ kN} \tag{4.5}
\]

### 4.3 Test Specimens

In the experimental procedure, 4 groups of test specimens were studied. Each group has 5 test specimens. Special cracks with certain depths and lengths were created in 3 groups of the test specimens, and 1 group was left as the reference group. In the following tables the properties of the test specimens are presented.

**Beam Type 1:** The test group without cracks (reference group).

<table>
<thead>
<tr>
<th>Nr</th>
<th>Quality</th>
<th>Dimensions (mm)</th>
<th>Crack Location</th>
<th>Crack Depth</th>
<th>Crack Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1-5</td>
<td>spruce, L40c</td>
<td>115x315x2600</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 4.4. General view of beam type 1.

**Beam Type 2:** The test group with 30% crack depth with respect to the width on one side.

*Table 4.2. Specifications of beam type 2.*

<table>
<thead>
<tr>
<th>Nr</th>
<th>Quality</th>
<th>Dimensions (mm)</th>
<th>Crack Location</th>
<th>Crack Depth (mm)</th>
<th>Crack Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1-5</td>
<td>spruce, L40c</td>
<td>115x315x2600</td>
<td>Mid height</td>
<td>34.5</td>
<td>2600</td>
</tr>
</tbody>
</table>

Figure 4.5. General view of beam type 2.
**Beam Type 3:** The test group with 15% crack depth with respect to width on opposite sides.

*Table 4.3. Specifications of beam type 3.*

<table>
<thead>
<tr>
<th>Nr</th>
<th>Quality</th>
<th>Dimensions (mm)</th>
<th>Crack Location</th>
<th>Crack Depth (mm)</th>
<th>Crack Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1-5</td>
<td>spruce, L40c</td>
<td>115x315x2600</td>
<td>Mid height</td>
<td>17.2</td>
<td>2600</td>
</tr>
</tbody>
</table>

*Figure 4.6. General view of beam type 3.*

**Beam Type 4:** The test group with two cracks on one side, crack depth 30% of the width

*Table 4.4. Specifications of beam type 4.*

<table>
<thead>
<tr>
<th>Nr</th>
<th>Quality</th>
<th>Dimensions (mm)</th>
<th>Crack Location</th>
<th>Crack Depth (mm)</th>
<th>Crack Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4:1-5</td>
<td>spruce, L40c</td>
<td>115x315x2600</td>
<td>Middle of the third lamella from top and bottom(113.5 mm)</td>
<td>34.5</td>
<td>2600</td>
</tr>
</tbody>
</table>
All the cracks on the glulam beams were created with a log saw machine with 2.5 mm wide blade as can be seen in the Figure 4.8.

Figure 4.8. Procedure for creating artificial cracks in the glulam beams.
### 4.4 Test Results

#### 4.4.1 Beam Type 1

*Table 4.5. Beam type 1 geometrical data and test results.*

<table>
<thead>
<tr>
<th>Nr</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>Width (mm)</th>
<th>Weight (kg)</th>
<th>Density (kg/m³)</th>
<th>Number of Annual Rings per cm *</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>2600</td>
<td>313</td>
<td>112</td>
<td>42.44</td>
<td>465.63</td>
<td>5.6</td>
</tr>
<tr>
<td>1:2</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>42.08</td>
<td>457.59</td>
<td>6</td>
</tr>
<tr>
<td>1:3</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>41.56</td>
<td>451.94</td>
<td>5.8 (3rd Lam. 26)</td>
</tr>
<tr>
<td>1:4</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>42.24</td>
<td>459.33</td>
<td>6.1 (4th Lam 24)</td>
</tr>
<tr>
<td>1:5</td>
<td>2600</td>
<td>313</td>
<td>112</td>
<td>42.42</td>
<td>465.41</td>
<td>6.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Finger Joints</th>
<th>Crack Length After Test</th>
<th>Number of Knots</th>
<th>Defects</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>6</td>
<td>1380</td>
<td>49</td>
<td>4 small(sm.) Cracks on side</td>
</tr>
<tr>
<td>1:2</td>
<td>7</td>
<td>990</td>
<td>58</td>
<td>7 sm. Crack on side and top</td>
</tr>
<tr>
<td>1:3</td>
<td>6</td>
<td>2000</td>
<td>63</td>
<td>2 sm. Crack on side</td>
</tr>
<tr>
<td>1:4</td>
<td>4</td>
<td>1050</td>
<td>47</td>
<td>5 sm. Crack on side</td>
</tr>
<tr>
<td>1:5</td>
<td>5</td>
<td>1010</td>
<td>62</td>
<td>5 sm. Crack on top</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Moisture Content (%) **</th>
<th>Failure Type</th>
<th>Critical Load (kN)</th>
<th>Max. Def.(mm)</th>
<th>Time until Failure (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>12.23–13.96</td>
<td>Bending</td>
<td>163.66</td>
<td>28.72</td>
<td>5.7</td>
</tr>
<tr>
<td>1:2</td>
<td>12.26–13.23</td>
<td>Bending</td>
<td>159.74</td>
<td>18.01</td>
<td>3.92</td>
</tr>
<tr>
<td>1:3</td>
<td>12.1–12.6</td>
<td>Mixed</td>
<td>166.55</td>
<td>27.77</td>
<td>5.58</td>
</tr>
<tr>
<td>1:4</td>
<td>12.26–13</td>
<td>Bending</td>
<td>168.76</td>
<td>33.94</td>
<td>6.67</td>
</tr>
<tr>
<td>1:5</td>
<td>12.4–13.13</td>
<td>Shear</td>
<td>176.80</td>
<td>25.19</td>
<td>4.99</td>
</tr>
</tbody>
</table>
*The parameters of the annual rings are mean values for each beam. The number of annual rings/cm varies from 2.6 to 6.4 in whole beam group.

**Moisture contents were measured with a portable moisture content device at three points along each beam and the mean value was taken. First value shows the surface moisture content, and the second shows the internal moisture content at depth 30 mm.

![Beam Type 1](image)

**Figure 4.9. Load-deflection charts for beam type 1.**

Three of the beam type 1 specimens tested failed due to bending stress. On the other hand, the bending failure critical loads show that the critical loads for shear failure for these beams are higher than the bending failure load, or they are at least equal to the bending failure load. One of the remarkable details for this group is beam number 3. It failed due to both shear stress and bending stress (mixed). The shear failure occurred in the middle of the beam and almost along the entire beam length and the bending failure occurred at the tension lamella by cracking in the finger joints. The load/deflection curve of the beam 2 started around 27 mm due to non-calibrated deflectometer. The mean value for the failures is 167.10 kN. In the **Figure 4.10** the mixed failure of beam number 3 can be seen;
4.4.2 Beam Type 2

Table 4.6. Beam type 2 geometrical data and test results

<table>
<thead>
<tr>
<th>Nr</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>Width (mm)</th>
<th>Weight (kg)</th>
<th>Density (kg/m³)</th>
<th>Number of Annual Rings per cm</th>
<th>Annual Lam.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>40.52</td>
<td>440.63</td>
<td>6.3 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 32)</td>
<td></td>
</tr>
<tr>
<td>2:2</td>
<td>2598</td>
<td>314</td>
<td>113</td>
<td>42.74</td>
<td>463.65</td>
<td>6.2 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 49)</td>
<td></td>
</tr>
<tr>
<td>2:3</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>40.98</td>
<td>445.63</td>
<td>5.8 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 17)</td>
<td></td>
</tr>
<tr>
<td>2:4</td>
<td>2601</td>
<td>312</td>
<td>112</td>
<td>40.58</td>
<td>446.48</td>
<td>5.6 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 59)</td>
<td></td>
</tr>
<tr>
<td>2:5</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>41.84</td>
<td>454.99</td>
<td>5.6 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 42)</td>
<td></td>
</tr>
<tr>
<td>Nr</td>
<td>Finger Joints</td>
<td>Crack Length After Test</td>
<td>Number of Knots</td>
<td>Defects</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----</td>
<td>---------------</td>
<td>-------------------------</td>
<td>----------------</td>
<td>--------------------------------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:1</td>
<td>4</td>
<td>1090</td>
<td>43</td>
<td>Small break at end</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:2</td>
<td>2</td>
<td>1360</td>
<td>49</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:3</td>
<td>4</td>
<td>730</td>
<td>43</td>
<td>2 small defects(def.) crack at end</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:4</td>
<td>2</td>
<td>1200</td>
<td>49</td>
<td>1 small def. cracks at bottom</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:5</td>
<td>3</td>
<td>990</td>
<td>73</td>
<td>1 big 2 sm. cracks at end</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Moisture Content (%)</th>
<th>Failure Type</th>
<th>Critical Load (kN)</th>
<th>Max. Def.(mm)</th>
<th>Time until Failure (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1</td>
<td>11.86-12.63</td>
<td>Shear</td>
<td>125.26</td>
<td>16.84</td>
<td>2.59</td>
</tr>
<tr>
<td>2:2</td>
<td>11.9-12.83</td>
<td>Shear</td>
<td>133.99</td>
<td>19.17</td>
<td>2.82</td>
</tr>
<tr>
<td>2:3</td>
<td>11.7-12.2</td>
<td>Shear</td>
<td>92.67</td>
<td>10.42</td>
<td>2.38</td>
</tr>
<tr>
<td>2:4</td>
<td>11.86-12.6</td>
<td>Shear</td>
<td>141.8</td>
<td>23.07</td>
<td>4.67</td>
</tr>
<tr>
<td>2:5</td>
<td>12.1-12.46</td>
<td>Shear</td>
<td>120.2</td>
<td>20.30</td>
<td>3.32</td>
</tr>
</tbody>
</table>
In the beam type 2, all the test specimens failed due to shear stress. The mean value for the critical load is 122.78 kN. One important detail is that test specimens number 3 failed at 92.67 kN. The most reliable explanation for this is that the number of the annual rings at the crack lamella is very low (17 annual rings) compared to the other lamellas, and therefore the distance between the annual rings is large. This reduces the shear capacity of this lamella and causes shear failure very early.

Figure 4.11. Load-deflection charts for beam type 2 beams.
### 4.4.3 Beam Type 3

*Table 4.7. Beam type 3 geometrical data and test results.*

<table>
<thead>
<tr>
<th>Nr</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>Width (mm)</th>
<th>Weight (kg)</th>
<th>Density (kg/m³)</th>
<th>Number of Annual Rings per cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1</td>
<td>2599</td>
<td>313</td>
<td>113</td>
<td>41.02</td>
<td>446.24</td>
<td>5.9 (5&lt;sup&gt;th&lt;/sup&gt; Lam. 37, 4&lt;sup&gt;th&lt;/sup&gt; Lam. 28)</td>
</tr>
<tr>
<td>3:2</td>
<td>2599</td>
<td>313</td>
<td>113</td>
<td>41.46</td>
<td>451.02</td>
<td>5.7 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 20)</td>
</tr>
<tr>
<td>3:3</td>
<td>2599</td>
<td>313</td>
<td>113</td>
<td>40.44</td>
<td>439.93</td>
<td>5.9 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 32)</td>
</tr>
<tr>
<td>3:4</td>
<td>2599</td>
<td>313</td>
<td>113</td>
<td>42.00</td>
<td>456.90</td>
<td>5.3 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 35)</td>
</tr>
<tr>
<td>3:5</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>41.76</td>
<td>454.11</td>
<td>5.8 (4&lt;sup&gt;th&lt;/sup&gt; Lam. 28)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Finger Joints</th>
<th>Crack Length After Test</th>
<th>Number of Knots</th>
<th>Defects</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1</td>
<td>2</td>
<td>746</td>
<td>51</td>
<td>Sm. crack at end and surface</td>
</tr>
<tr>
<td>3:2</td>
<td>4</td>
<td>560</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>3:3</td>
<td>3</td>
<td>953</td>
<td>64</td>
<td>Sm. splitting on surface</td>
</tr>
<tr>
<td>3:4</td>
<td>4</td>
<td>800</td>
<td>52</td>
<td>1 crack at the end</td>
</tr>
<tr>
<td>3:5</td>
<td>4</td>
<td>410</td>
<td>60</td>
<td>2 sm. defects</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Moisture Content (%)</th>
<th>Failure Type</th>
<th>Critical Load (kN)</th>
<th>Max. Def. (mm)</th>
<th>Time until Failure (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1</td>
<td>12.46-13.03</td>
<td>Shear</td>
<td>154.20</td>
<td>30.67</td>
<td>5.83</td>
</tr>
<tr>
<td>3:2</td>
<td>12.7-13.8</td>
<td>Shear</td>
<td>134.14</td>
<td>15.83</td>
<td>2.9</td>
</tr>
<tr>
<td>3:3</td>
<td>12.53-12.86</td>
<td>Shear</td>
<td>141.09</td>
<td>24.17</td>
<td>4.14</td>
</tr>
<tr>
<td>3:4</td>
<td>12.56-12.93</td>
<td>Shear</td>
<td>124.91</td>
<td>11.84</td>
<td>2.95</td>
</tr>
<tr>
<td>3:5</td>
<td>12.3-12.63</td>
<td>Bending</td>
<td>140.91</td>
<td>44.97</td>
<td>6.4</td>
</tr>
</tbody>
</table>
Figure 4.12. Deflection-Time charts for beam type 3 beams.

The mean value for the critical load is 138.59 kN. One of the important points in the beam type 3 bending tests is that beam number 1 experienced shear failure, however shear failure occurred at one lamella below the cracked lamella and it was at the maximum critical load. This result will be discussed in the simulation and in the conclusions and future work chapters.

### 4.4.4 Beam Type 4

Table 4.8. Beam type 4 geometrical data and test results.

<table>
<thead>
<tr>
<th>Nr</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>Width (mm)</th>
<th>Weight (kg)</th>
<th>Density (kg/m³)</th>
<th>Number of Rings per cm</th>
<th>Annual Rings</th>
</tr>
</thead>
<tbody>
<tr>
<td>4:1</td>
<td>26001</td>
<td>314</td>
<td>112</td>
<td>41.94</td>
<td>458.50</td>
<td>6.1 (3³ Lam. 51, 5⁵ Lam. 51)</td>
<td></td>
</tr>
<tr>
<td>4:2</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>40.80</td>
<td>443.67</td>
<td>5.9 (3³ Lam. 37, 5⁵ Lam. 29)</td>
<td></td>
</tr>
<tr>
<td>4:3</td>
<td>2600</td>
<td>313</td>
<td>113</td>
<td>40.64</td>
<td>441.93</td>
<td>5.6 (3³ Lam. 37, 5⁵ Lam. 31)</td>
<td></td>
</tr>
<tr>
<td>4:4</td>
<td>2599</td>
<td>313</td>
<td>112</td>
<td>42.66</td>
<td>468.22</td>
<td>5.4 (3³ Lam. 38, 5⁵ Lam. 32)</td>
<td></td>
</tr>
<tr>
<td>4:5</td>
<td>2599</td>
<td>313</td>
<td>113</td>
<td>43.00</td>
<td>467.78</td>
<td>5.1 (3³ Lam. 32, 5⁵ Lam. 44)</td>
<td></td>
</tr>
<tr>
<td>Nr</td>
<td>Finger Joints</td>
<td>Crack Length After Test</td>
<td>Number of Knots</td>
<td>Defects</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----</td>
<td>---------------</td>
<td>-------------------------</td>
<td>----------------</td>
<td>---------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:1</td>
<td>5</td>
<td>1466</td>
<td>56</td>
<td>2 cracks on side</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:2</td>
<td>1</td>
<td>1175</td>
<td>50</td>
<td>5 sm. cracks on side, 1 crack at the end</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:3</td>
<td>6</td>
<td>920</td>
<td>74</td>
<td>2 sm. cracks on side</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:4</td>
<td>4</td>
<td>1412</td>
<td>76</td>
<td>2 sm. cracks on top, 3 sm. cracks on side</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:5</td>
<td>6</td>
<td>855</td>
<td>55</td>
<td>Big crack on side</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nr</th>
<th>Moisture Content (%)</th>
<th>Failure Type</th>
<th>Critical Load (kN)</th>
<th>Max. def.(mm)</th>
<th>Time until Failure (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4:1</td>
<td>11.5-12.86</td>
<td>Shear</td>
<td>111.11</td>
<td>13.15</td>
<td>3.07</td>
</tr>
<tr>
<td>4:2</td>
<td>11.9-13.5</td>
<td>Shear</td>
<td>130.70</td>
<td>15.36</td>
<td>3.14</td>
</tr>
<tr>
<td>4:3</td>
<td>11.36-12.5</td>
<td>Shear</td>
<td>143.11</td>
<td>17.58</td>
<td>3.32</td>
</tr>
<tr>
<td>4:4</td>
<td>11.8-13.11</td>
<td>Shear</td>
<td>121.14</td>
<td>15.68</td>
<td>3.00</td>
</tr>
<tr>
<td>4:5</td>
<td>10.66-13.1</td>
<td>Shear</td>
<td>126.78</td>
<td>15.54</td>
<td>2.78</td>
</tr>
</tbody>
</table>
Figure 4.13. Load-deflection charts for beam type 4 beams.

In the beam type 4, all test specimens failed due to shear stress, and the mean value for critical load was 126.56 kN.

For calculation of mean value for critical load, all test specimens were included, also beams with bending failure. If we compare all test specimens, it can be seen that the beam type 1 test specimens (reference beams) with no cracks have the highest shear stress capacity, as is expected. However, for more reliable results more tests are of course needed in order to avoid bending failure of the beams. If we look at the beam type 2 it can be seen that the shear capacity decreased considerably compared to beam type 1.

Furthermore, the number of the annual rings at the crack location influences the shear critical load. The test group 3 beams have better shear capacity than beam type 2 and beam type 4 test specimens. The main reason behind this is the fact that the beam with cracks on opposite sides with equal crack depth act like a beam with no crack but a rather small cross-section. Thus the shear stresses are symmetrically distributed in the beam, and in consequence not enough stress concentration occurred that initiates propagation around the crack location, compared to the beams type 2 and 4. Beam type 4 exhibits similar mechanical properties to beam type 2, and their shear stress capacities are close.

52
5 Simulation

In the simulation part, the test specimens were simulated in the commercial finite-element analysis program ABAQUS CAE version 6.10. Only the reference beam group (beam type 1) was not simulated due to the failure model. If there is no crack in the structure there will be no failure in the simulation, but you can observe the bending of the structure and study stresses.

In the simulation, the crack propagation is based on theoretical fracture criteria which will be explained in detail in the following sections.

Wood as an engineered material is always difficult to simulate due to its structural properties. Wood is an anisotropic material and it contains many defects, knots, angle grains, etc., which cause great difficulties in modelling and also in designing. Due to this wood has been accepted as an orthotropic (transversely isotropic) material, which means it has similar structural properties in three local coordinates, which are radial-R tangential-T and longitudinal-L(z-axis), as shown in the Figure 5.1.

![Figure 5.1. Local directions of wood [15].](image)
5.1 Assumptions

- The moisture content of each lamella is assumed constant and has no influence on beam strength.
- The pith location of each lamella is assumed as shown in Figure 5.1 (middle of the bottom edge).
- The failure criterion of the wood material is based on energy considerations as derived by Griffith (1920) and linear elastic fracture mechanics.
- Each wood lamella has the same elastic properties.
- Mode III critical energy release rate is chosen as a very high value in order to avoid crack propagation in mode III.
- The temperature of each lamella is assumed constant and has no influence on beam strength.
- The density of each beam was assumed to be the same.
- Adhesives between the lamellas were not modelled, and it was assumed that the adhesive properties are similar to wood.

5.2 The method of modelling and simulation of 3D timber structures with cracks

- Enter the part field and create the glulam beam in the required dimensions. The important point of this part is to create the whole beam first and then divide it into several lamellas in order to avoid problem with discontinuities in the results.
- In the part field create the crack in the required dimensions. The modelling space should be 3D and the base feature should be shell and type extrusion.
- After creating a whole beam then create datum planes in order to create partition cells.
- Each partition cell should have its own coordinate system. To do that enter “create datum CSYS: 3 Points” option and then select the cylindrical option. Now you should define the centre of the coordinate system, then radial axis-r, tangential axis-t ABAQUS automatically defines the longitudinal-l axis (z-axis).
- Enter the section assignment field. Attribute each lamellas of the glulam to the corresponding section.
- Enter orientations field. Attribute a material orientation for each lamella using the pre-defined local cylindrical coordinate system.
- Enter property field. Define density and then elastic properties for each lamella by using “Engineering Constants” option.
- Define the damage criterion for the lamella which has crack or cracks. Choose “Damage for Traction Separation Laws” and “Maxps Damage”.
- Enter assembly field. Instance the part and select all parts of the beam and crack and then cross the option “Independent”. In this area it may be required to relocate the crack. To do that click “Translate Instance” and move the crack to the required location.
- Enter step field. Create a step or steps and define the time interval, the maximum number of increments, and the initial, minimum and maximum increment.
- Enter the “Field Output Requests”. In the “Failure/Fracture” option cross the “PHILISM, level set value phi”. Enter the “State/Field/User/Time” option, and cross the “STATUSXFEM, Status of xfem element” option.
- Enter the interaction field. Create interaction select step as initial then select elastic foundation. Define the area in which you would place the elastic foundations and enter the “Foundation stiffness per area” value.
- Enter the interaction properties field. Create a new interaction property by choosing the “Contact” option. Then define the fracture criterion by choosing the “Fracture Criterion” option and define the direction of crack growth relative to the local 1- direction and mixed mode behaviour. Enter the critical energy release rate values and exponents. Then go to top main menu bar and enter the “Special” menu then select “Crack” and “Create”. Select the “XFEM” option. Select the lamella which has cracks as a crack region. Cross the “Allow crack growth” option. Select the 3D planar crack as the crack location. Cross the “Specify contact property” and select the contact property as defined before.
- Enter the load field. Select “Pressure” and apply that pressure on the specific area that is required (the area can be created by using partition face method). Create boundary conditions based on 3 point bending test.
- Enter the jobs field. Create a job and submit the model.
In this study, the pith location of the each lamella is placed on the bottom edge and in the middle of the width as in the Figure 5.2.

*Figure 5.2. The pith locations.*

The local coordinate system of the beam was created as in Figure 5.3.
Figure 5.3. Local cylindrical coordinate system of each lamella.

The R-direction shows the radial direction, T-direction shows the tangential direction, and the Z-direction shows the longitudinal direction of the wood.

In this study, the maximum principle stress criterion was defined as a damage criterion. The maximum principle stress criterion can be explained as Equation (5.0);

$$f = \left\{ \frac{<\sigma_{max}>}{\sigma^0_{max}} \right\}$$  \hspace{1cm} (5.0)

Where the $\sigma^0_{max}$ is the maximum allowable principle stress. The symbol $<>$ represents the Macaulay bracket. The purpose of using the Macaulay bracket is to avoid damage initiation by purely compressive stress state. It is assumed that the damage initiates when the maximum principle stress ratio reaches a value of one [16].

$$< \sigma_{max} > = \left\{ \begin{array}{ll}
0 & \sigma_{max} < 0 \\
\sigma_{max} & \sigma_{max} \geq 0
\end{array} \right.$$  \hspace{1cm} (5.1)
For our model, the maximum principle stress was kept at 4 MPa as a constant material property during the simulation. Furthermore, the influence of the different values of the maximum principle stress on the various simulation models was investigated. As a result, this parameter does not have such a large influence on crack propagation.

The orthotropic material properties of each lamella of the glulam beam are given in the Table 5.1;

<table>
<thead>
<tr>
<th>Elasticity Modulus</th>
<th>Poisson’s Ratio</th>
<th>Shear Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_r = E_1 = 424.8$ MPa</td>
<td>$\nu_{rt} = \nu_{12} = 0.418$</td>
<td>$G_{rt} = G_{12} = 34.17$ MPa</td>
</tr>
<tr>
<td>$E_t = E_2 = 249.4$ MPa</td>
<td>$\nu_{rl} = \nu_{13} = 0.00707$</td>
<td>$G_{rl} = G_{13} = 729$ MPa</td>
</tr>
<tr>
<td>$E_l = E_3 = 11390$ MPa</td>
<td>$\nu_{tl} = \nu_{23} = 0.00521$</td>
<td>$G_{tl} = G_{23} = 694.8$ MPa</td>
</tr>
</tbody>
</table>

To observe the crack propagation, the extended finite element method (XFEM) was applied in the simulation.

In the simulation, to avoid the distortion of those elements located at the supports, the elastic foundations were applied. This also provides a better distribution of the reaction forces at the supports and overcomes the hourglass problem in the elements around the support area. The value of the foundations stiffness per area was chosen as $81.5 \times 10^6$ N/m$^2$.

The pressure was estimated from equation 4.3 with Figure 4.1 and chosen to be 4.28 MPa.

In the simulation, for the fracture criterion the XFEM-based LEFM (using VCCT) method was applied. Power law was chosen as the mixed mode behaviour. The power law criterion is based on relationship between the energy release rates in I,II,III modes and the critical energy release rate parameters as in Equation (5.2);

$$
\left( \frac{g_I}{g_{IIC}} \right)^a + \left( \frac{g_{II}}{g_{IIC}} \right)^b + \left( \frac{g_{III}}{g_{IIC}} \right)^c \geq 1
$$

Also the three fracture modes can be seen in the Figure 5.4.
During the simulation, ABAQUS/CAE calculates all three modes’ energy release rates, based on the criterion that if the fracture parameter is greater than unity then crack growth is expected.

Haller and Putzger claim that based on the experiments, the mode I critical energy release rate for spruce is 179 N/m, and the mode II critical energy release rate is 737 N/m [18]. However, the mode III critical energy release rate was not mentioned in that paper.

In addition, the mode II critical energy release rate is influenced by mode I and mode III as a consequence of the mixed mode behaviour (power law criterion). The certain parameters was used and optimised in the simulation in order to observe the normal stress (S23) around the crack location and minimize the influence of the other modes on mode II.

In our simulation a fracture criterion for the mode III critical energy release rate was chosen as 18000 N/m to avoid mode III crack propagation. Furthermore, the mode I critical energy release rate was chosen to be 179 N/m, and mode II critical energy release rate was adjusted and calibrated as 60 N/m, after it was tested as 35 N/m to 70 N/m, in order to get good match with the experiments. The mode II critical energy release rate is lower than the other modes because of the pre-existing crack.

Figure 5.4. Fracture modes [17].
5.3 Simulation Results

5.3.1 Beam Type 2

In this type, the crack depth is 30% of the beam width. The crack location is in the middle of the 4th lamella of the beam. In the Figure 5.5, the bending shear stress distribution (S23) and the shear stress critical zone can be seen;

*Figure 5.5. Deformed shape of the beam type 2 and contours of the shear stress at critical load.*
Figure 5.6. Normal stress distribution in the cracked lamella at critical load.

The direction of the crack propagation is similar to the experiment such that the crack started to propagate from the crack tip and followed in the annual ring direction. In order to predict when the structure will fail we have to look at when the crack starts to propagate, because if the crack starts to propagate then structure will fail shortly afterwards. For observing the starting point of the crack propagation we can examine the strain energy-time graphic for whole model.
As it can be seen in Figure 5.7 the strain energy increases very rapidly after 0.7 seconds. It is the point at which the crack starts to propagate. Also, we can see the damage dissipation energy graphic for whole model for observing when the damage initiates. Thus we can calculate the critical load from one of these graphs.

Figure 5.8. Damage dissipation energy-time graph for the whole model.
The exact time for damage initiation (crack propagation) is 0.745 sec. Thus, the critical load can be determined as in Equation (5.2);

\[
F_{crit} = \frac{P \cdot t_{crit} \cdot A_p}{t_{all}}
\]

(5.2)

Where;

- \( F_{crit} \) is the critical load for crack propagation.
- \( P \) is the pressure.
- \( t_{crit} \) is the critical time for crack propagation.
- \( A_p \) is the area at which the pressure is applied.
- \( t_{all} \) is the time of the whole procedure.

From the Equation (5.2);

\[
F_{crit} = \frac{4.28 \cdot 10^6 \cdot 0.745278 \cdot 0.3 \cdot 0.115}{1} = 110047.75 N = 110.05 kN
\]

(5.3)

In the experiment, the mean value of the critical load \( P = 122.78 \) kN. The difference between the experiment and the simulation might be in the material properties, moisture content, defects in the beam, loading conditions, and material orientation of each lamella, discontinuities in the beam, support, or might be in other reasons. On the other hand, all the simulations showed similarities between the experiments and models and reasonable result for the critical load were obtained.

### 5.3.2 Beam Type 3

In this type the crack depth is 15% of the width and at both side of the middle lamella. In Figure 5.9 the bending shear stress distribution (S23) and the shear stress critical zone (yellow and green zone) can be seen.
Figure 5.9. Deformed shape of the beam type 3 and contours of shear stress at maximum load.

Figure 5.10. Normal stress distributions in beam type 3 at maximum load.
In the simulation of the beam type 3 under the same loading condition as the experiment, crack propagation was not observed. Even though in the critical shear zone there was a high shear stress, it was however not enough to initiate damage.

According to EC 5 the maximum theoretical load for the shear stress was calculated as 147.7 kN (Eq. 4.3). The result of the FE simulation showed that this type of beam can resist more than 147.7 kN for initiate the crack propagation. In the simulation, for this type of beam the threshold value for crack propagation was 253.38 kN. However, in the experiments the mean critical load was 138.58 kN, and as a consequence further experiments are needed in order to understand and verify the failure, whether it is due to crack propagation or other reasons.

On the other hand, in the experiments (see Figure 5.12) one beam failed in the fifth lamella instead of the lamella which had cracks. This lamella had also different pith location. In addition, one beam has failed by bending stress. These results also show that the validity of the FE modelling results for this type of beam are questionable. The fracture criterion does not fit the simulation of beam type 3.

Figure 5.11. Strain energy-time graph.
Figure 5.12. Beam type 3 beam number 1.
5.3.3 Beam Type 4

In this type the crack depth is 30% of the beam width on one side of the beam at the 3rd and 5th lamellas.

Figure 5.13. Deformed shape of beam type 4 and contours of shear stress at critical load.

Figure 5.14. Normal stress distribution of 3rd lamella.
As it can be seen in Figure 5.16, crack propagation has not started until 0.91 seconds. Thus the critical load for crack propagation is;
\[ P = 4.28 \cdot 10^6 \cdot 0.91 \cdot 0.3 \cdot 0.115 = 134370.6 \quad N = 134.38 \quad kN \]

In the experiments the mean critical load for the crack propagation or it can be said that, the critical load for structure failure was 126.56 kN.
6 Conclusions and Future Work

In this study, 4 types of glulam beam with different cracks were studied. The influence of these cracks on the beam strength was investigated. 3D FEM analyses of each beam was modelled and simulated for three of the beam types with different located pre-existing cracks. The simulation model was modified and improved based on the experimental feedback. The reference group of beams (beam type 1) could not be modelled because it did not include any cracks and therefore it was not possible to determine when the structure failed in the simulation. The comparison of the critical loads between the experiments and simulations can be seen in the Table 5.2.

Table 5.2. Comparison between experimental and simulation results.

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Experimental Results*</th>
<th>Simulation Results</th>
<th>%Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Type 1 (ref. beam)</td>
<td>167.10 kN</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Beam Type 2</td>
<td>122.78 kN</td>
<td>110.05 kN</td>
<td>10.36%</td>
</tr>
<tr>
<td>Beam Type 3</td>
<td>138.59 kN</td>
<td>No crack propagation</td>
<td>-</td>
</tr>
<tr>
<td>Beam Type 4</td>
<td>126.56 kN</td>
<td>134.38 kN</td>
<td>5.81%</td>
</tr>
</tbody>
</table>

*All test specimens were included also the test specimens with bending failure.

One interesting fact to note is that beam type 4 exhibited better mechanical properties compared to beam type 2 in both the simulation and experiment. This illustrates the influence of the crack location on the shear stress capacity of glulam beams. One of the main reasons for this condition is that the shear stress diagram of the glulam beam along the beam height shows the shear stress distribution as illustrates in Figure 5.17.
The other noticeable fact is that in the simulation of beam type 3 (15% crack depth on opposite sides), no crack propagation was obtained under same pressure conditions as 4.28 MPa. This also shows that the beam exhibited similar mechanical properties as a beam with no cracks but with a smaller cross-section, as in Figure 5.18.

Figure 5.17. Shear stress distribution along the beam height.

Figure 5.18. The beam type 2 and the loading condition.
The major weakness of this study is the lack of the material data, and the crack model (fracture criterion parameters) was unknown for the timber structure. In addition, there are very few studies that exist in the literature about the simulation of cracks in timber structures, and the number of experiments and samples that have been conducted on this topic is not sufficient for deducing accurate results. As a consequence, differences occurred between the experimental results and simulation results. Furthermore, the simulation can be improved in the future by adding more parameters such as the moisture contents or elastic properties of each lamella.

More realistic fracture parameters could also be created and applied for timber structures, and the various fracture criterions could be applicable in the simulation (BK Reeder model, bilinear model, etc.).

Furthermore, a beam with natural cracks in different locations, different directions, with different crack lengths and depths under various loading conditions could be simulated in the future. However, despite many parameters being added in the simulation there will eventually be differences between the real life and simulated conditions due to the anisotropic structure of wood.
7 References

4. Sundström T., (2010), Shear Resistance of Gluelam Beams Under Varying Humidity Conditions, Master’s Dissertation, Aalto University, School of Science and Technology, Faculty of Engineering and Architecture, Espoo, Finland.


