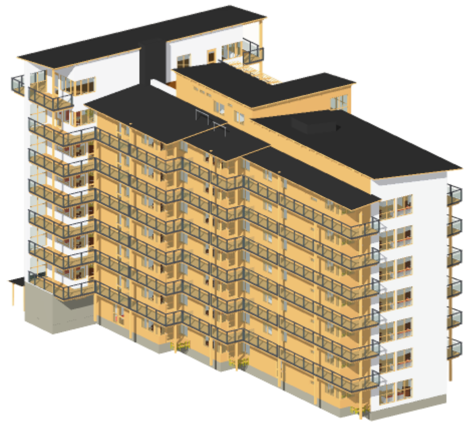
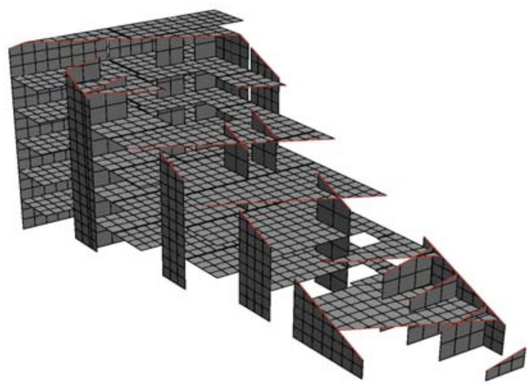
THESIS FOR THE DEGREE OF LICENTIATE OF ENGINEERING





SHEAR WALLS FOR MULTI-STOREY TIMBER BUILDINGS

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The figure on the previous page shows one of the timber buildings located in the Limnologen building complex in the city of Växjö. At the top is a drawing of it that the firm Arkitektbolaget Kronoberg AB, which designed the building, made of it using the software ArchiCAD, and at the bottom is a finite element model of the same building.

Abstract

Wind loads acting on wooden building structures need to be dealt with adequately in order to ensure that neither the serviceability limit state nor the ultimate limit state is exceeded. For the structural designer of tall buildings, avoiding the possibly serious consequences of heavy wind loading while taking account at the same time of the effects of gravitation can be a real challenge. Wind loads are usually no major problem for low buildings, such as one- to two-storey timber structures involving ordinary walls made by nailing or screwing sheets of various types to the frame, but when taller structures are designed and built, serious problems may arise.

Since wind speed and thus wind pressure increases with height above the ground and the shear forces transmitted by the walls increase accordingly, storey by storey, considerable efforts can be needed to handle the strong horizontal shear forces that are exerted on the bottom floor in particular. The strong uplift forces that can develop on the wind side of a structure are yet another matter that can be critical. Accordingly, a structure needs to be anchored to the substrate or to the ground by connections that are properly designed. Since the calculated uplift forces depend very much upon the models employed, the choice of models and simplifications in the analysis that are undertaken also need to be considered carefully.

The present licentiate thesis addresses questions of how wind loads acting on multi-storey timber buildings can be best dealt with and calculated for in the structural design of such buildings. The conventional use of sheathing either nailed or screwed to a timber framework is considered, together with other methods of stabilizing timber structures. Alternative ways of using solid timber elements for stabilization are also of special interest.

The finite element method was employed in simulating the structural behaviour of stabilizing units. A study was carried out of walls in which sheathing was nailed onto a timber frame. Different structural levels were involved, extending from modelling the performance of a single fastener and of the connection of the sheathing to frame, to the use of models of this sort for studying the overall structural behaviour of wall elements that possess a stabilizing function. The results of models used for simulating different load cases for walls agreed reasonably well with experimental test results. The structural properties of the fasteners binding the sheathing to the frame, as well as of the connections between the members of the frame were shown to have a strong effect on the simulated behaviour of shear wall units.

Regarding solid wall panels, it was concluded that walls with a high level of both stiffness and strength can be produced by use of such panels, and also that the connections between the solid wall panels can be designed in such a way that the shear forces involved are effectively transmitted from one panel to the next.

Keywords: multi-storey structures, timber engineering, wind stabilization, shear walls, cross-laminated timber wall panels, fasteners, connections, sheathing

Acknowledgements

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Finally, I would like to thank the great family of mine, my parents in particular, for listening to me and discussing things with me throughout.

Växjö, May 2008

Johan Vessby

Outline of the thesis

The present work is concerned above all with the stabilization of tall timber structures. After the introduction to the work as a whole in chapter 1, an overview of the horizontal loads of interest is provided in chapter 2. Because of the marked concern for wind load in Sweden such loading is emphasized in particular. In chapter 3 both structural systems for stabilizing timber buildings and the properties of the structural elements involved in the transfer of horizontal loads to the foundations of a structure are discussed. Conventional stabilization approaches in which sheathing is nailed or screwed to a timber frame as well as use of solid timber panels are considered here. Chapters 1-3 are not to be viewed as summaries of the papers or as further discussion of the topics taken up there, but rather as a background and introduction to the topic of the stabilization of tall timber structures generally. The discussion there is followed in chapter 4 by a brief summary of the matters that the appended papers deal with. Proposals for further research are considered then in chapter 5.

Appended papers

Paper I **Stabilizing strategies for multi-story timber frame structures**

Johan Vessby and Anders Olsson

Presented at *World Conference on Timber Engineering*, August 6-10, 2006, Portland, Oregon, USA.

Paper II **Modelling aspects of wooden shear walls with nailed sheathing**

Johan Vessby, Anders Olsson, Bo Källsner and Ulf Arne Girhammar

Paper in preparation for publication.

Paper III **Experimental study of cross-laminated timber wall panels**

Johan Vessby, Bertil Enquist, Hans Petersson and Tomas Alsmarker

Submitted to *Forest Products Journal.*

Paper IV **Contact-free strain measurement of bi-axially loaded sheathing-to-framing connection**

Johan Vessby, Anders Olsson and Bertil Enquist

Accepted for *World Conference on Timber Engineering*, June 2-5, 2008, Miyazaki, Japan.

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1 Introduction

1.1 General remarks

In Sweden the construction of single-family houses in which the structural support system involves use of timber has a long tradition. According to the Ministry of Industry, Employment and Communications [45], timber is contained in the bearing parts of some 90 % of all single-family houses built in Sweden today. There are many reasons for the choice of wood here, possibly the two major being the long tradition that building with timber has had and the ample supply of timber as a raw material. Today Sweden has about 22,6 Mha of forest land [51], much more than it had several decades ago.

In the nineteenth century the extensive use of timber for building purposes led to numerous and severe fires in cities [8]. During that century, at least one major fire occurred in the central part of most Swedish cities. As a result, regulations were introduced allowing only one- or two-storey structures to be of timber. This led to other materials replacing timber, as well as to many broad avenues dividing cities into different parts being built, and to some buildings being constructed of stone in the bottom storey or storeys and of timber in the two upper storeys. An important consequence of the regulations was that tall houses of timber were no longer built. This led to much detailed knowledge concerning the building of tall timber structures gradually being lost and to timber construction only being used for single-family houses that were a maximum of two storeys high. When the regulations were changed once again in 1994 [7], so that the building of tall timber structures was again permitted, engineers and architects had to deal with many challenging new design issues without having much knowledge or experience from the past to guide them. One of the last tall timber houses constructed before the more restrictive building laws came into effect early in the 20th century is shown in Figure 1, together with the first five-storey timber structure built in Växjö after the change in regulations in the mid-1990s.

Figure 1. Two timber-based multi-storey buildings in southern Sweden. A multi-storey wooden structure, erected in Kisa 1902 representing one of the last ones built in Sweden prior to the passage of restrictive laws, is shown at the left, and Wälludden, the first modern multi-storey wooden building in Sweden, built in Växjö in 1995, is shown at the right.

For tall buildings in general, achieving adequate stabilization so as to counteract the horizontal action produced by the wind load on the building is a challenging task, see Olsson [46]. This wind loading and the gravitational forces accompanying it represent the basic load cases that need to be borne in mind as a building is being designed. These two types of load actions are closely related to each other and need to be taken into account in analysing the structural performance of tall building structures generally [11]. How such loads can be dealt with in timber structures is of special interest in the present study.

A number of different structural systems for the stabilization of tall multi-storey buildings of timber are available. The choice of a stabilizing system often affects both the architectural appearance of such a building and its structural performance. It is thus often of considerable importance that an early discussion is held between the architects and the structural engineers involved regarding what system is to be selected.

1.2 Aim and scope

The aim of the thesis is to investigate the different ways in which tall timber structures can be stabilized against the effects of strong horizontal wind loading. The wall systems more closely investigated are composed either of timber frames to which sheets are nailed or screwed or of cross-laminated solid timber elements. Detailed analyses based on the structural properties of individual connectors as well as discussions of the overall structural performance of these wall systems under different loading conditions are included.

The major emphasis is on methods utilizing the interior or the exterior timber walls of buildings as stabilizing elements. The alternative approach of using other parts of the structure, such as elevator shafts of concrete, for stabilizing purposes is not considered here. Also, for the most part Nordic European design conditions are considered.

2 Horizontal loading of timber structures

2.1 General remarks

Building structures are subjected to both horizontal and vertical loads. The latter are caused mainly by dead weights, working loads and snow loads. Horizontal loading of structures, in turn, may be caused by extreme events such as accidents or terrorist attacks, the attack on the World Trade Center in 2002, for example, although such events occur very seldom. Far more common is horizontal loading in the form of either seismic loading or wind loading. On a worldwide basis loadings of these two types have produced roughly equal amounts of damage over a very long period of time, see Holmes [31]. Sweden and the other Nordic countries, fortunately, are very seldom exposed to seismic actions, wind-related disasters being far more costly in terms of property damage and casualties, cf. Davenport [12]. Since in the Nordic countries wind loads are the most common type of horizontal loads measures to counteract them by means of appropriate design are of immediate importance to engineers, see e.g. Solli and Bovim [52]. In the present chapter wind loads acting on timber structures are discussed and various comments concerning common engineering praxis in connection with such loads are provided. Extensive background material concerning earthquake engineering in connection with timber constructions can be found in the journal literature, see e.g. Filiatrault et al. [21] and White and Dolan [57]. Boughton [5] presents the results of full-scale testing of timber structures subjected to horizontal loading. Tests results of this sort provide valuable information on the behaviour of timber structures loaded to failure.

2.2 Horizontal loading acting on timber structures

Various aspects of the general behaviour of a loaded timber structure and certain nomenclature used for the structural parts of a one-storey timber building are presented in Figure 2. The structural parts shown are a floor diaphragm and two walls, one of these a lateral wall subjected to wind loading and the other a shear wall subjected to shear forces. Facade walls distribute wind loads to the substrate and to the floor diaphragm, which acts as a deep beam through its tension and compression edge zones or “chords”. Shear forces caused by wind loads are transmitted from the floor diaphragm to the gable walls, these walls being called shear walls for this reason. Reaction forces on the bottom rail can also act either as horizontal shear forces, as vertical compression forces or, close to the so-called leading studs, as tension forces.

The structural behaviour of the one-storey timber structure shown in Figure 2 also partly holds for multi-storey structures, the main difference being that in the latter case the shear forces at the bottom rail of each storey are transmitted to the storey below instead of to the substrate.

The structural response that timber shear walls show to loading has been subjected to substantial research. An overview of the literature can be found, for example, in Dinehart, Shenton and Harry [14], Dolan and Johnson [15], Filiatrault et al. [20] and Itani and Cheung [33] and in Lam et al. [41].

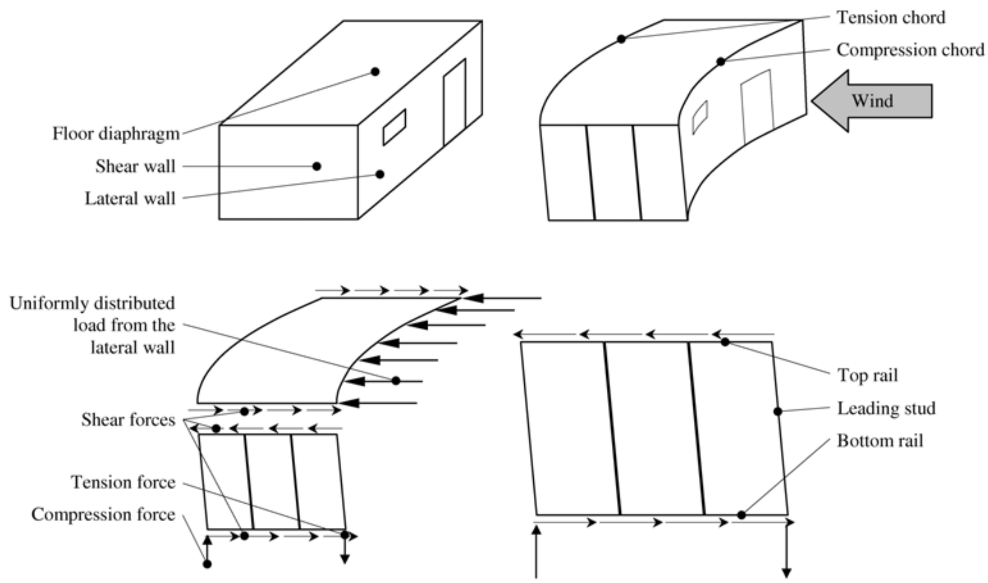

Figure 2. Nomenclature used for various structural parts of a one-storey timber building and different structural responses to wind loading.

2.3 Engineering praxis

At around the middle of the 20th century, wind gust speeds were often used as a basis for estimates of the wind loading of structures, see Hertig [30]. Since knowledge of wind action has increased very much since then, this approach is normally not used in connection with modern standards, such as Eurocode [19]. Instead, the most common approach to assessing wind load is to base calculations on the average wind speed for a 10-minute period. For buildings generally, this serves as a reference value, one that varies between different countries and between different regions within a country.

In Sweden an equivalent static wind load, q_k, is based on the reference wind speed, q_{ref}, 10 meters above the ground averaged over a 10-minute period in an open landscape [6]. This equivalent static wind load corresponding to the reference wind is expressed as

$$q_k = C_{dyn} \cdot C_{exp} \cdot q_{ref}$$

where the factor C_{dyn} takes account of the increase in wind load due to the dynamic nature of wind and C_{exp} is an exposure factor taking account of variation due to the height above the ground and the influence of the surrounding landscape.

In order to account for the load distribution over the building the characteristic static wind load, w_k, is determined by multiplying q_k by the shape factor μ, i.e.

$$w_k = \mu \cdot q_k$$

The shape factor, which depends upon the geometry of the building, is used for calculating internal or external pressure or suction.

The factor C_{exp} has been a subject of discussion for a number of years. In Sweden a logarithmic expression is used for heights above a certain given level, the value taken being dependent upon the surrounding geography. The total characteristic wind load, including contributions of pressure, suction and friction, affecting a building as a whole, is reduced by multiplying the total wind load on the structure by the constant 0.85 which reflects the fact that the wind does not strike the entire building in full strength at any given time.

q_{ref} is a statistically-based reference value representing the pressure against the building during a return period of 50 years. In Sweden such a reference value is based on measurements made during the period of 1970/71 to 1992/93. The value q_{ref} is obtained as $0.5 \cdot \rho \cdot v^2_{ref}$, where ρ is the air density and v_{ref} is the reference wind velocity. Codes differ in the return period used for the reference wind. A return period of R years means that the wind speed has a probability of $1/R$ of exceeding the given level in any particular year. For a given expected lifetime of a structure, L, the risk r, see [31], of a wind with a return period of R years exceeding this speed, can be obtained as

$$r = 1 - \left(1 - \left(\frac{1}{R}\right)\right)^L$$

In Table 1 the risk of wind speeds with differing return periods being exceeded are shown for different expected lifetimes. For example, for a building with an expected lifetime L of 50 years and a return period for the wind of 50 years, the risk of the wind speed in question being exceeded is 0.636.

Table 1. Risk of a wind speed with a return period of R years being exceeded during different expected lifetimes.

Return period, R	**Expected lifetime, L**			
	30	50	70	90
30	*0,638*	*0,816*	*0,907*	*0,953*
50	*0,455*	*0,636*	*0,757*	*0,838*
100	*0,260*	*0,395*	*0,505*	*0,595*

2.4 Gudrun – possibly the worst storm in Sweden in modern time

One of the worst storms in Sweden in modern times (one referred to as Gudrun) occurred January 8, 2005. It was a storm that hit southern Sweden, in particular, specifically the regions of Skåne, Blekinge, Halland and Småland. It was clearly a disaster. Trees with an estimated value of 18 billion SEK were knocked down. During the storm the Swedish Meteorological and Hydrological Institute (SMHI) measured the wind at a number of weather reporting stations. One of these stations is located south of the city of Växjö, at a wind-exposed site close to the university campus area there, see Figure 3. Four different series of values based on wind measurements that where made are shown in Figure 4. These four series represent, respectively, the average speed of a 10-minute wind, the average wind speed during a one-hour period, the maximum 10-minute wind speed during a 3-hour period, and the maximum wind gust speed (as measured for 2 seconds) during a one-hour period. It is of interest to note the large difference between the maximum wind gust speed and the average wind speed during a 10-minute period. This implies that, in efforts to design wooden building structures in such a way that they can withstand a serious storm, information concerning the maximum wind gust speed should be interpreted quite differently than information concerning the average wind speed during a 10-minute period.

Another observation to be made is that there is only a small difference in the average wind-speed measurements obtained for a 10-minute and for a 1-hour period. According to the Swedish code, the reference value for the 10-minute average wind speed should be 24 m/s when designing structures in Växjö for a 50-year return period. This value is considerably higher than the maximum 10-minute value during a 3-hour period, as reported for the Gudrun storm. On the western and the southern coast (in the communities of Trubaduren and Hanö, respectively) the maximum wind gust reported during this storm reached up to 40.1 and 42.2 m/s, respectively. It is very seldom that such winds are reported in southern Sweden. According to the Beaufort scaling system [3] they are classified as representing hurricane values.

Figure 3. SMHI's weather report site south of Växjö close to the university campus.

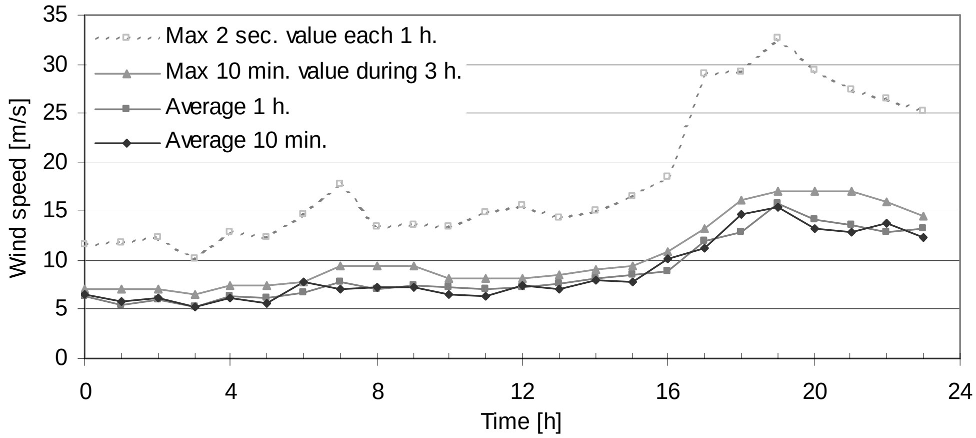

Figure 4. Wind speed values in Växjö during the Gudrun storm 8:th Januari 2005 according to SMHI.

After the Gudrun storm, various building failures were reported. One of these is shown in Figure 5. In timber buildings, large deformations normally appear before any final failure occurs. It can be argued that the risk of such large deformations should be reduced to an acceptably low level by performing an analysis of extreme wind loadings. Other important issues concern what velocities and accelerations should be regarded as acceptable for those living in wooden buildings. If these are established, a serviceability limit state analysis can be performed in order to determine whether a given building fulfils these requirements.

Figure 5. An old timber structure that failed in the Gudrun storm. Photo S. Palmblad.

2.5 Remarks concerning dynamic aspects of wind

Wind pressure is not constant but varies over time. The variation corresponds to differing frequency values for the pressure coefficient involved. The distribution of the wind frequencies indicates whether treating the wind load as a static or as a dynamic load is more adequate. Typical frequency distributions, both for wind loading and for earthquakes, are shown in Figure 6. The normalised spectral density is given there as a function of the frequencies as represented on a logarithmic scale. Wind has a broader frequency domain than an earthquake does, although its mean level is much lower.

As shown in Figure 6, some winds achieve frequencies well above 1 Hz. Traditionally, building structures in which the first natural frequency is less than some particular value are treated as being dynamically loaded structures. This can be a value of 1 Hz, see e.g. Holmes [31] or 3 Hz, see BSV 97 [6]. Natural frequencies below 1 Hz normally only occur in tall buildings. Dynamic effects thus need to be taken into account there, whereas in low-rise buildings it is usually sufficient to consider wind loads as being static loads. According to a Swedish design recommendation [6], the wind load can be regarded as static if the first natural frequency of the building structure in question is higher than 3 Hz and the damping is greater than 0.1. Since in timber structures the damping is usually high displacements are often damped out rather quickly.

Jeary [34] made a rough estimate of the first natural frequency for high office buildings, finding it to be

$$N_0 = 46/h$$

where h is the height of the building (in meters) and N_0 is the first natural frequency. Note that such an estimate is not made for timber buildings.

Ellis and Bougard [18] have shown that for a six-storey timber-frame structure of specified type the first natural frequency is above 2 Hz, although this depends somewhat upon the finishing level of the building. A comprehensive background to the field of structural dynamics can be found in Géradin and Rixen [24], whereas a practical approach for direct use in structural design can be found in Åkerlund [58].

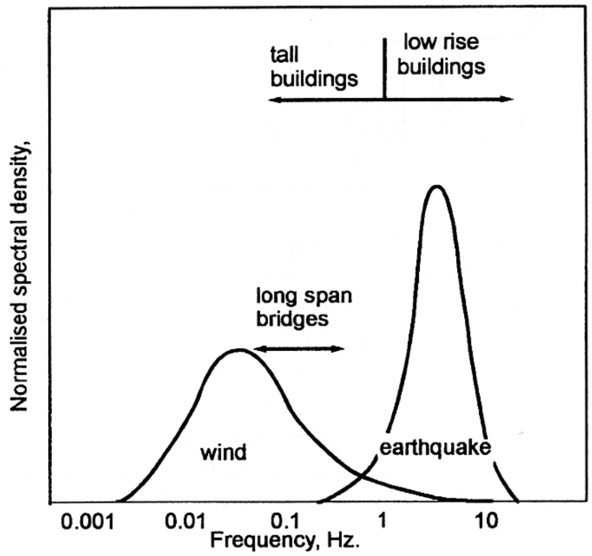

Figure 6. Frequency content of winds and of earthquakes according to Holmes [31].

2.6 Some remarks regarding the design criteria for serviceability limit state analysis

The current praxis in Sweden today in the designing of shear walls in timber buildings able to resist lateral loads is generally to perform only an ultimate limit state analysis. If no dynamic effects are considered, the wind load is treated as an equivalent static load. The Swedish building code [6] prescribes a reference wind, the designer needing to verify that the building can resist this static loading. In a discussion recently of this manner of designing high-rise buildings, it was noted that in slender timber structures in particular, large deformations can occur before the ultimate load is reached. Such deformations may disturb the occupants of the building through giving rise to structural noise that creates anxiety on their part. Undesired events of this sort indicate that the limiting state for serviceability can be of considerable importance in the designing of tall timber buildings.

Estimation of the top displacement of a timber frame structure subjected to static wind loads can easily be carried out by use of established calculation methods, see Carling [10]. Little is said in Swedish design recommendations, however, regarding what variations in the top displacement of high timber buildings are allowable and what levels of vibrations can be accepted. Perceptions of vibration, however, are subjective and may therefore be difficult to handle in objective terms. In codes and in design recommendations in which the performance of high buildings is discussed, acceleration is a measure often used as a basis for determining a comfort criterion. A horizontal acceleration limit has usually been employed as a design measure. The allowable peak accelerations for different return periods are specified as function of the first natural frequency of the structure, see Figure 7. According to Melbourne [44], the equation for

the maximum allowable horizontal acceleration $\hat{x}$ for the different return periods, should be set to

$$\hat{x} = \sqrt{2\ln(n_0 T)}(0.68 + \frac{\ln(R)}{5})\exp(-3.65 - 0.41\ln(n_0))$$

where T is the duration considered in seconds (10 min = 600 sec.), n_0 is the first natural frequency ($0.06<n_0\leq 1.0$) and R is the return period in years ($0.5<R<10$). In Figure 7 the suggested maximum allowable horizontal acceleration $\hat{x}$ is shown, for four different return periods, as a function of the first natural frequency.

For materials such as steel and concrete, the dynamic response of tall buildings to wind loading is generally taken into account in building above a certain construction height, a design practice for such buildings having been established [11]. Although this height can be thought to often be substantially lower for timber structures, since timber structures are more flexible, little work regarding this has been reported in the literature. Some work has been done regarding the serviceability limit state for tall timber structures that are wind loaded, see e.g. Ellis and Bougard [18], but further investigation is clearly needed to obtain better understanding of the dynamic behaviour of timber structures with respect to their serviceability state. The effects that use of stucco and various other finishing materials has on stiffness, strength and deformation as discussed by Uang and Gatto [55], for example, indicate their use to lead to a considerable increase in stiffness. Finite element modelling of wooden buildings subjected to hysteric dynamic loading has been carried out by Tarabia and Itani [54], for example.

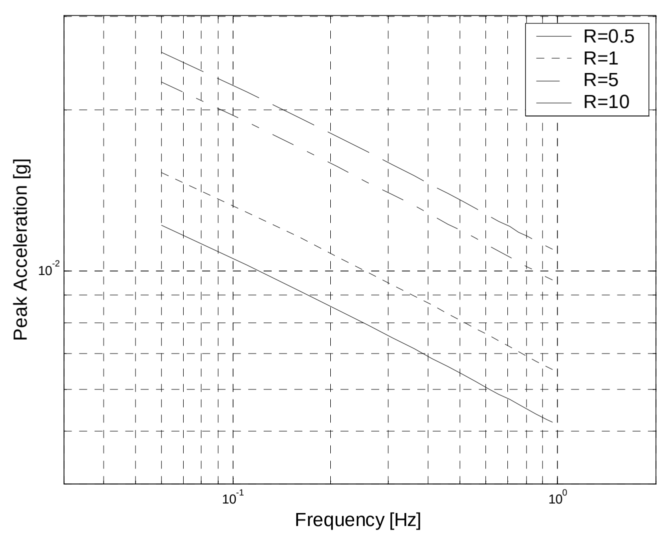

Figure 7. Maximum allowable horizontal acceleration $\hat{x}$ as a function of the first natural frequency for four different return periods, according to Melbourne [44].

3 Stabilizing systems for tall timber buildings

A number of different systems for the stabilization of tall multi-storey timber buildings are available. The choice of a stabilizing system can affect the final appearance of a structure, since systems differ in the type of buildings for which they are most appropriate. In Figure 8, different systems and philosophies that can be used are presented. In the top row, Figure 8(a-b), two different ways of treating conventional shear walls are shown. At the left (a) a system traditionally used today in one- and in two-storey buildings is shown. The wall consists of sheets nailed or screwed onto a timber frame, although the influence of the sheathing above and below the openings is not accounted for and the contribution of narrow sheets is not included in the calculations. This exclusion of the sheathing in the walls results in the calculations regarding the horizontal load-bearing capacity of the structure being conservative and very high anchorage and compression forces being obtained at specific points in the shear walls. One of the main advantages of using this rough design philosophy is that hand calculations can be easily performed. In wall (b) the entire wall, with all of the sheets that are assumed to be involved in stabilization are included. This results in the calculated performance for wall (b) being quite different than that for wall (a). This latter manner of considering the various contributions to the global stiffness of the structure is more correct, although the analysis called for is more demanding. Normally, it requires the use of more advanced calculation methods, such as use of the finite element method. The reader is referred to Ottosen and Petersson [48], for example, for more detailed insight into the method. Software is available and the topic is under intense research. There are also other, though rougher methods available for dealing with the wall in its entirety. One of these, which can be found in the international building code (IBC) [32], is based on the reduction in capacity due to use of the opening area as compared with the total wall area.

In Figure 8(c-d), two different stabilizing systems using solid timber elements are shown. The stiffness of the solid timber elements is much greater than the stiffness of conventional timber frame walls with nailed or screwed sheathing. In the wall shown in Figure 8(c) only a part of the wall (the marked region) consists of solid timber, whereas the whole wall shown in Figure 8(d) is built of solid timber elements. One of the advantages of using solid timber elements in a part of the wall as compared with the conventional technique based on sheathing is that the stabilizing part of the wall is well defined. The solid timber element can be designed in the ultimate limit state and in the serviceability limit state in order to handle horizontal loads. Since with use of this principle the rest of the wall is not part of the stabilizing system, it can be designed accordingly. This can create possibilities for use of larger openings and more sparsely placed fasteners or use of lower quality (less expensive) sheathing material in other parts of the wall. Another consequence is that high compression or anchorage forces can be dealt with entirely by the solid timber elements that have a high capacity for handling such forces.

In any of the systems chosen, the walls are normally viewed as plane elements without any interaction with the laterally located walls. If such interaction were considered in the analysis, this would make it easier to handle the anchoring issue, since the calculated

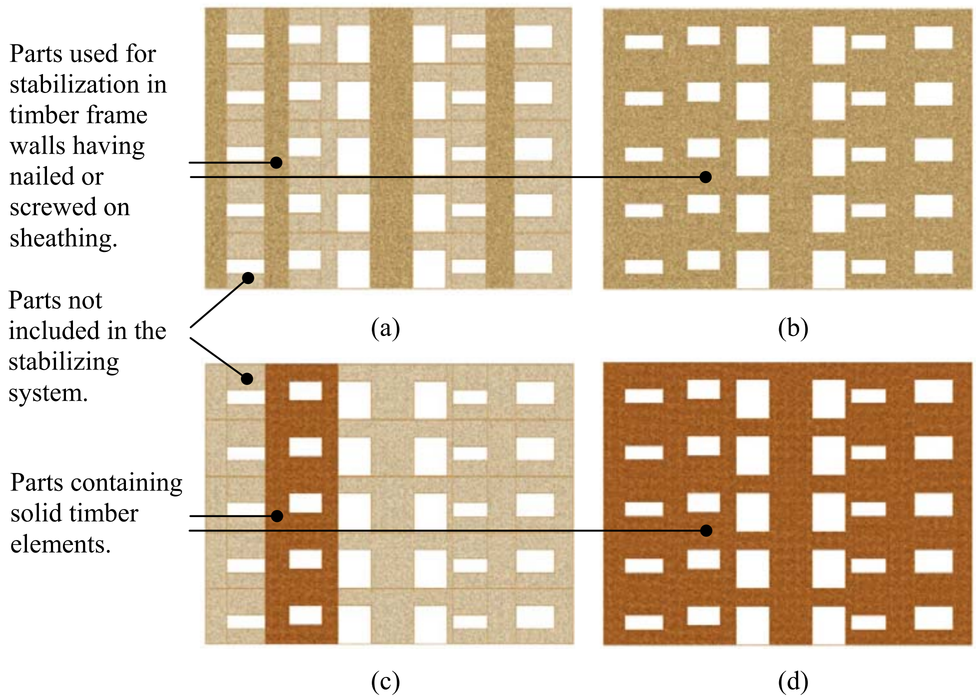

Figure 8. Four different stabilizing systems:
(a) Conventional method, in which selected parts are used for stabilization.
(b) Conventional method, in which all the sheets are included in the stabilizing system.
(c) Solid timber element located in a selected part of the wall.
(d) Solid timber elements located in the entire wall.

uplift forces would be lower. This is a topic taken up by Andreasson [2], for example. Another matter that can affect the performance of shear walls is that of moisture induced deformations of the wooden members. Ormarsson [47] provides a detailed introduction to the field of moisture-related distortions. There has also been work carried out aimed at including such effects in the overall analysis of shear walls, e.g. Bäckström [9].

3.1 Conventional design of the stabilizing system

Tall timber buildings subjected to wind loading can be designed in different ways, as was discussed above. The use of shear walls with nailed or screwed sheathing for stabilization purposes is very popular. Common sheathing materials used are gypsum, plywood and oriented strain board (OSB). Achieving sufficient lateral resistance through use of such building methods can normally be obtained for one- and two-storey buildings without difficulties even if conservative design assumptions are employed. Different hand calculation methods are available, see Carling [10], Källsner [39], Degerman [13],

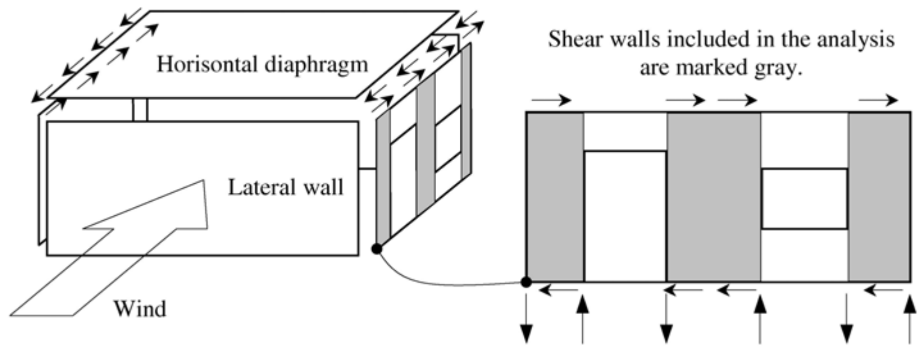

Figure 9. Schematic view for a simplified analysis of a one-storey portion of a timber frame structure.

Gyproc Handbok [28] and McCutcheon [43]. Some of the design principles used for low-rise structures can also be extrapolated so as to hold for multi-storey buildings as well.

Today timber frame structures are often simply modelled as consisting of a number of vertical sheets acting together, which are connected to stiff horizontal diaphragms and horizontal rails that distribute the forces involved. An analysis of wood diaphragms in this way can be found in Foschi [23]. Figure 9 shows the parts of a stabilizing system corresponding to a single storey of a timber frame building. The sheets used for stabilization in the simplified analysis carried out are marked in grey. In accordance with a Swedish design recommendation, the floor diaphragms can be assumed to be rigid in the ultimate limit state, see Carling [10], when the length-to-width ratio of the structure is not too large. This assumption implies that the horizontal load that acts on the structure can be distributed to the shear walls in proportion to their relative stiffness. For a further discussion of models for analysis and the choice of which shear walls are most suitable to use in the calculations, the reader is referred to Kasal et al. [37], Salenikovich and Dolan [50] and Girhammar and Källsner [25], for example.

The assumptions and simplifications described above, neglecting contributions to the overall lateral resistance of certain parts of the walls and assuming infinite stiffness of the floor diaphragm, make hand calculations relatively simple, and lead in most cases to solutions being on the safe side. Neglecting the stiffness of wall sections above and below the windows and the door openings (the white parts of the walls, as shown in Figure 9) can result in misleading conclusions. The simplified calculations obtained can provide displacements that are much too large in a serviceability state analysis, their crudely underestimating the overall global stiffness of the structural system. Another consequence of the simplifying assumptions is that the analysis leads to very high compression forces and uplift forces appearing at the bottom of the shear walls. The calculated forces obtained may be considerably larger than the forces calculated in an analysis in which all the sheathing material is included. The topic of the anchorage of timber structures is discussed by Salenicovich and Dolan [50] and by Kessel and Dettermann [38].

3.1.1 Types of connections for vertical forces

Various types of connections or joints can transmit the forces acting between the structural elements of a timber structure. The forces transferred between the storeys are of special interest and, as shown in Figure 9, both vertical and horizontal forces need to be transmitted from a given storey to the one below. The capacity of the connections between different storeys and the devices for anchoring the structure to the ground are of considerable importance for the overall performance of a structural system.

Examples of three different types of connections between storeys are shown in Figure 10(a-c), where (a) provides an example of so-called “balloon framing” and (b) and (c) represent examples of “platform framing”. In the case of balloon framing, the flooring is attached to the wall via a joist on the inside of the wall, so that the wall is continuous at the periphery of the building, whereas in the platform case the flooring is laid on top of the rails at each storey. The differing designs involved result in differing properties with respect to strength and stiffness. Few experimental tests and evaluations of such connections have been carried out, despite the properties of these connections being of marked importance for the overall structural behaviour. Figure 10(d-f) shows three examples, from a wide range of available solutions, of how anchorage to the substrate can be designed. The details shown in (e) and (f) represent designs that are frequently used, whereas (d) is less well known. In this latter case the bottom rail is cut into two parts and the stud supports a piece of metal instead of the rail, so as to avoid loading perpendicular to grain in the bottom rail.

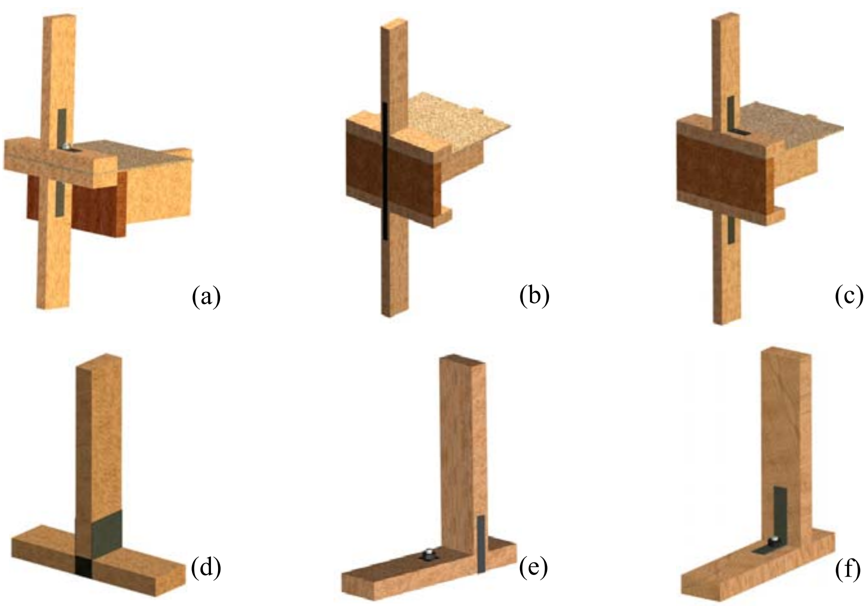

Figure 10. Examples of inter-component connections (a-c) and of anchorage connections (d-f).

Although there are many different types of joints for the handling of uplift forces available on the market, most of them are not efficient enough to handle the large forces that can be calculated for multi-storey buildings. These forces may be up to 100 kN in magnitude. None of the joints shown in Figure 10 are suitable for handling such large forces. What have become popular instead are post-tensioned tie-down-rods cast into the concrete foundation of the building. With use of such a technique, even very strong uplift forces can be handled effectively in the ultimate limit state. It is also furthermore important to consider the dead weight of the building in the calculations in order to obtain a more favourable load-case, i.e. more reasonable uplift forces. Källsner and Girhammar [40] have developed a plastic design method that has been extended to include calculation methods for incompletely anchored diaphragms in which account can be taken of vertical loading of the structure. The method is verified with tests performed on partially anchored shear walls, Girhammar and Källsner [26].

3.1.2 Properties of connections for vertical compression forces

In the traditional design of a wooden frame, the stud meets the rail at a right angle, and in case of the transmission of compression force between the members (the normal loading case) the wood in the rail becomes loaded perpendicular to the grain. Large compression forces may occur, not only from dead and live vertical loads, but also from additional forces related to wind loads on the building. In general, two different cases can occur: the case in which the rail ends on one side of the stud, and the case in which the rail is continuous on both sides of the stud. The stiffness and the load capacity of such connections can be evaluated experimentally. In Figure 11, results for three different test setups are shown and are compared. In two of the tests, a steel specimen is compressed against the rail, and in the third test, a wooden member is pressed against the rail. The dimensions, the density and the moisture content for the different tests also differ, yet the

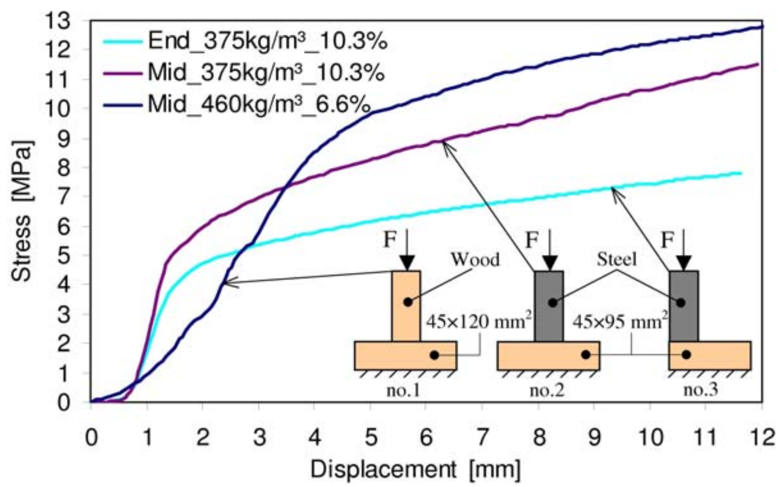

Figure 11. Experimental results for compression of a rail, i.e. compression perpendicular to the grain, see Vessby, Olsson and Källsner [56].

most important difference is that of loading in the centre versus at the end of the rail. It can be clearly seen from the results that the test in which there is wood in both the vertical and the horizontal part is initially less stiff than in the two cases in which steel members are pressed against the rail. This can be explained not only by the possibility of penetration of the vertical member by the fibres, but also on the basis of the initial gap between the two surfaces. Once plasticity in the rail has occurred, the inclinations of the stress-displacement curves are very similar for the two tests involving mid-compression. The higher density and the lower moisture-content can explain the difference in stress level between the two tests in which there is compression in the middle of the bottom rail. In the test in which the rail ends at one side of the stud, plastic behaviour occurs at a lower compression force. The tests give an indication of the capacity of connections in which wood is compressed perpendicular to the grain. For example, a stress of 10 MPa corresponds to a contact force of 54 kN in the case of a 45×120 mm^2 contact surface. This is less than what can be required for a tall building. In such a case, a design in which wood is compressed perpendicular to the grain should be avoided. For further information about the experimental test, see Vessby, Olsson and Källsner [56]. Practical design recommendations can be found in for instance Larsen and Riberholt [42].

3.1.3 Modelling of shear walls

Shear walls consisting of a timber frame with a sheathing mechanically attached to the frame by nailing or screwing has been modelled by many researchers threw use of the finite element method, see e.g. Gupta and Kuo [27], He, Lam and Foschi [29], and Itani and Cheung [33]. In the analysis of such shear walls, several decisions have to be made regarding the assumptions, the simplifications and the modelling to be employed. Below is a brief account of various important choices to be made regarding the modelling when simulating the behaviour of walls.

In shear walls there are three different components that can be identified: the timber frame, the sheathing and the fastener. Both a physical and a finite element representation of these three components are shown in Figure 12. The framing in the shear wall is normally modelled by use of beam elements, the elements being each assigned a cross-section and being placed in the symmetry line of the physical member. This approach is normally effective with respect to computational costs yet it introduces a geometric approximation into the model since the precise location of the interface between the members is not specified. In addition, the effects of compression perpendicular to the fibres need to be modelled with use of certain simplifying assumptions, e.g. spring elements with a given load-displacement characterisation instead of a contact condition. The interface between the members and the properties these have when compressed can be modelled in detail, although at the expense of simplicity and of computational efficiency, by use of 2- or 3-dimensional elements. This also provides the possibility of including further material parameters.

The geometry of the sheathing is ideal for modelling by use of shell elements. It may be important to include the effects of sidewise interaction between the sheets, especially in the early stages of the loading history.

The third and most crucial element in the obtaining of sufficiently accurate simulation results is modelling of the connections. There are two different types of connections, those used in the framing, which connect the different members to each other, and those connecting the sheathing to the framing. The connections can be modelled by use either of a single-spring model or of a spring pair model. For reflecting the very different strength and stiffness properties in the two main directions in the connections between the timber members, the spring-pair model is normally suitable. In the connection between sheathing and framing, the choice of a model is more questionable. Using the single spring model is the simplest alternative, but there no account is taken of the differences in characteristics between the parallel and the perpendicular loading direction with respect to the wood fibres. Account of these differences can readily be taken by use of the spring pair model, although with the drawback that this model can overestimate the strength and the stiffness of the connection. In both models there is also a choice to make between use of nonlinear elasticity and use of plasticity. Further information regarding models for elasticity and plasticity can be found in Ottosen and Ristinmaa [49], for example. Considerable research has been directed at the question of how the connection between sheathing and framing should best be modelled. Some of this work is found in Foschi [22], Judd and Fonseca [35] and Girhammar and Källsner [25]. A further specialized topic is that of the cyclic loading of connections. This topic is dealt with in part by Dolan and Madsen [16] and by Tarabia and Kamiya [53].

Models of shear walls and entire buildings often tend to become large with respect to number of degrees of freedom if the aim is to simulate the behaviour of the structure in detail. This problem can be dealt with by use of sub-structuring, which is a common technique for reducing the size of a system, or by the representing of nails, for example, as an elastic medium in the model rather then as individual connections, see e.g. Kasal, Leichti and Itani [36] and Alsmarker [1].

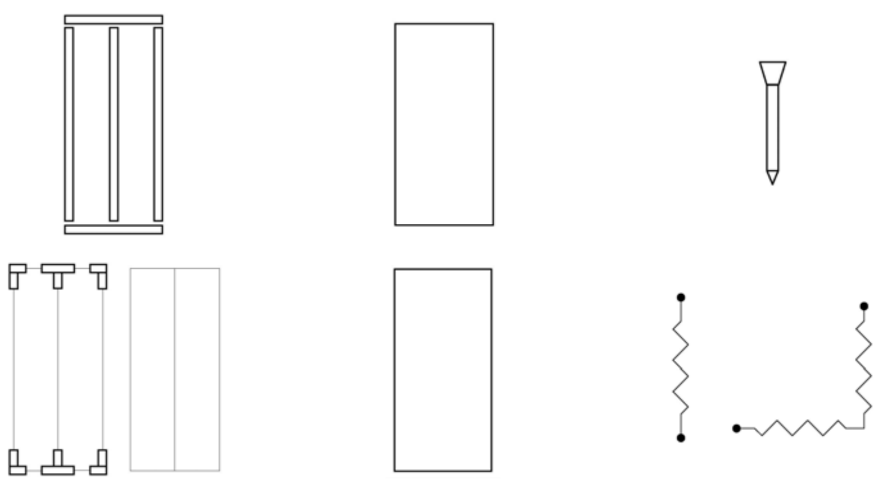

Figure 12. Representation of the three different parts of which a shear wall consists, in the top row for the physical model and in the bottom row for the finite element model.

3.2 Use of solid timber panels in the stabilizing system

An alternative way of stabilizing timber buildings is to use solid timber elements. Either a part of the walls or the walls in their entirety may be built of solid timber. Elements with differing numbers of layers and differing thicknesses of the layers are available on the market. Figure 13 shows two 96 mm thick, five-layer timber elements built up in two alternative ways. Layers 1, 3 and 5 are oriented in the longitudinal direction of the element, whereas layers 2 and 4 are oriented perpendicular to this. Similarities can be found between these solid timber elements and the well-established sheathing material plywood, which also has two main directions and is constructed of layers glued together. Laminated veneer lumber (LVL) is another product that can have two main material directions. Solid timber elements are normally produced with a width of less than 1.8 meters and a length of up to 12 meters or more. It is possible to assemble elements alongside each other and in this way to obtain broader elements. This can be very useful in structures in which units more than 1.8 meters in breadth are needed in order to provide for sufficient strength when the structures are loaded laterally. Due to the perpendicular orientation of the layers, the sections close to openings are high in stiffness and in strength. Details regarding techniques for calculating the deformations of solid wood panels can be found in Blass and Fellmoser [4], for example.

In a stabilizing system, the solid timber element may be included in one, two or still more storeys. The horizontal forces are transmitted to the elements by the floor and the roof diaphragms. Three different modes of deformation and displacement can be observed: bending, shear and anchorage slip or tilting, see Figure 14(a-c). The first two modes, bending and shear, relate to internal forces in the element, whereas the third one, anchorage slip, is a rigid-body mode caused by deformations in the anchorage. This last mode is often the critical one, see e.g. Dujic, Pucelj and Zarnic [17].

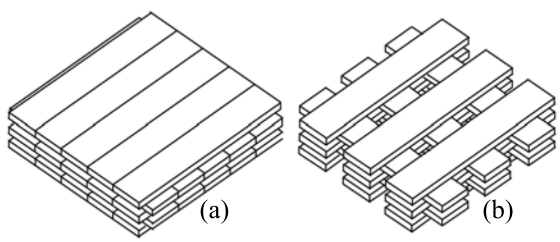

Figure 13. Detail of timber elements consisting of 5 layers
(a) conventional design of a solid timber panel,
(b) alternative design of a panel.

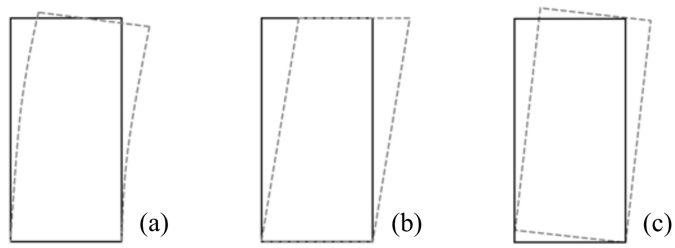

Figure 14. The three deflection components of a solid timber element, (a) bending, (b) shear and (c) anchorage slip (tilting).

3.3 Combining conventional timber frame walls with solid timber panels

In a combined wall system, parts of the wall can be constructed by use of a built using conventional timber frame technique, solid elements being used in other parts. The major objective of this approach is to achieve stabilization within a concentrated, well-defined part of the wall. Since the horizontal load is transferred to this part, the rest of the wall only needs to handle vertical loading. Thus, the major part of the wall does not have to provide stabilization in the system as a whole. This in turn permits the structure to have a more flexible design, something that could not be achieved by use of only the more conventional technique in the load-bearing parts.

4 Topics of the appended papers

There are four papers appended to the thesis. In paper I, different principles applying to the stabilizing systems available for tall timber buildings are discussed. The three papers thereafter contain more specific discussions of different topics, specifically the performance of cross-laminated timber elements in paper III, and the performance and modelling of conventional shear wall elements in papers II and IV. A brief summary of the four papers is given below.

Paper I discusses alternative methods of stabilization. Stabilizing timber structures stabilized by sheets of wood or other material nailed to a timber frame is often the most effective and economical choice, although it can also place major restrictions on the design of the structure. Alternative methods of stabilization can expand the architectural possibilities and provide opportunities for expression not common, if even possible, in traditional approaches involving use of timber. Shear walls both with sheathing nailed on and walls consisting completely or in part of solid elements composed of several layers of wood are discussed, along with stabilization by use of shafts and stairways.

Paper II concerns the influence of different modelling assumptions on the structural behaviour of shear wall elements in which sheathing is nailed to a wooden frame subjected to monotonic loading. Two types of sheathing-to-wood connection models are studied in the numerical simulations: a single-spring model and a spring-pair model. The nonlinear behaviour of the sheathing-to-wood connections is investigated using a nonlinear elastic model and a plastic model. The influence of the ductility of the sheathing-to-wood connections is investigated by varying the descending part of the load-displacement curves of the connections. Five models of stud-to-rail connections representing different stiffness properties are also evaluated. The results of FE-simulations are compared with results obtained in testing wall assemblies consisting of one and three sheets, respectively. The overall agreement between results of simulations and of laboratory tests is good when reasonable model assumptions are employed.

In Paper III the structural performance of five-layer cross-laminated timber elements and connections between them are examined. The five layers consist of boards 19 mm thick, laid successively at right angles to each other and glued together with PU-adhesive, layers 1, 3 and 5 lying in one direction and layers 2 and 4 in the other. The stiffness and strength of four cross-laminated timber elements (4955 mm long, 1250 mm wide and 96 mm thick) toward in-plane bending are studied. Two of the elements are first partitioned into two parts that are reconnected in two different ways prior to testing. The effects of the way in which the cross-laminated timber elements are reconnected are studied and the performance of the reconnected specimens is compared with that of the unpartitioned specimens with respect to strength and stiffness. The experimental tests performed show the cross-laminated timber elements to possess a high degree of stiffness and strength. There was also found to be a marked difference in behaviour between the two different ways in which the elements were connected to each other. One of the two connecting methods studied, in frequent use today, shows poor structural performance, whereas the other one performs well.

Paper IV discusses the properties of connections between sheathing and studs and between studs and rails in shear walls consisting of timber frames with a sheathing nailed or screwed to it. The overall behavior of such walls was found to be strongly dependent upon the stiffness, strength and ductility of the fasteners connecting the sheathing to the frame and upon the properties of the wood and the sheet material in the close vicinity to the fasteners. Another critical connection in such walls is that found between a stud and a rail, where the wood may be compressed perpendicular to the grain. Methods for laboratory investigations of this and results of experimental tests are discussed for connections of these two types. An optical measurement system for high precision continuous collection of the strain field on the surface of a tested specimen is described and is utilized in experiments. The knowledge obtained can be used in the planning of more advanced test setups in which test specimens are loaded bi-axially. Results of such experiments would enable more advanced models for connections to be developed than are presently available.

5 Proposals for further research

Different stabilizing timber-based systems for timber constructions were discussed in the thesis. Of the different systems considered, the conventional timber frame with nailed or screwed on sheathing is by far the most common in practical use today. For constructions more than 4 storeys in height, there are difficulties in achieving designs that provide stability when design rules and design practice in general use today are employed. In addition the architecture currently in fashion, in which there are many large openings in the facade, sometimes makes difficult to stabilize even one- and two-story houses of wood by use of the hand calculation methods available. Further research on these matters is needed. The two fields of research suggested here supplement each other, the first being concerned with the critical connection between sheathing and framing and the other with simulation at a more encompassing level, including as many structural parts of a timber building as needed to gain an adequate understanding of the overall behaviour of the structure.

The sheathing-to-framing connection is one of the important topics for gaining insight into the behaviour of a conventional timber frame wall. Different ways of modelling that connection differ on an overall basis in the results they provide with respect to the load-displacement relations applying to a single shear-wall unit. This connection should be studied in detail in order to obtain more adequate knowledge of the properties of different combinations of fastener and sheathing material and thus enable more accurate calculations of the capacity of a wall to be achieved. Such a study of the sheathing-to-framing connection should include both experimental tests performed in many different directions with respect to the length direction of the timber member and the creation of adequate models for numerical simulations that take account of variations in properties with respect to the loading direction. In finite element models of such connections, the coupling with respect to orthogonal directions should be included and possibly plastic properties as well.

A thorough understanding of the overall behaviour of buildings stabilized by shear walls, including knowledge of the forces between the different elements and the needs for anchorage, requires the development and use of 3-dimensional models of large parts of the buildings involved. The inter-element connections play a crucial role in the overall behaviour of such a structure. These connections include fasteners of different types. The contact between the different bearing parts is also important. Simulations of structures using models that capture the properties of the inter-element connections, in a horizontal as well as a vertical direction, are thus important in order to gain insight into the distribution of forces in an entire wall or an entire building. Such simulations should be performed. If the lateral walls and the flooring are included, the dead load of the structure as a whole can also be studied accurately, enabling rather realistic picture of the vertical loads to be obtained. This is important when considering the effects of horizontal loading. It is also tempting to aim at optimizing the structure as a whole with regard to the number and placement of the fasteners employed, as based on information that can be obtained from such a model.

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Paper I

Stabilizing strategies for multi-story timber frame structures

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Stabilizing strategies for multi-story timber frame structures

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Summary

There is a strong tradition of constructing timber structures stabilized by sheets of wood or other material nailed to a timber frame. In many cases this is the best and most economical choice, but it can also place major restrictions on the design of the structure. This paper discusses alternative methods of stabilization. New methods of this sort can expand the possibilities for architecture, providing opportunities for expression not common if even possible in traditional approaches involving use of timber. Shear walls both with a nailed on sheeting and those consisting completely or in part consisting of solid elements composed of several layers of wood are discussed, along with stabilization by use of shafts and stairways.

1. Introduction

In many countries, use of a timber framework has long been the commonest choice in constructing one- and two- story buildings. In Sweden about 95% of all one- or two-

(a)

(b)

Figure 1. Two tall timber-based buildings in southern Sweden: (a) one of the last multi-story buildings constructed 1902 around the time that the highly restrictive building laws were enacted and (b) Wälludden, the first modern multi-story wooden structure built in Sweden in 1995

story buildings designed for residential purposes are constructed with use of a timber framework. Since during the nineteenth century and earlier, however, several cities were devastated by fires, use of timber frameworks in Swedish buildings more than two stories in height was forbidden during most of the twentieth century.

This only changed in about the middle of the 1990s, when performance-based building codes were created in which safety requirements were no longer tied in this way to the choice of material. Currently no restrictions are placed on the number of floors permitted in wooden buildings, as long as performance-based requirements are met.

Figure 1(a) shows one of the last multi-story buildings in Sweden constructed with a timber framework prior to introduction of the regulations forbidding this. It was built in Kisa in southern Sweden in 1902. Figure 1(b), in turn, shows the first five-story building, called Wälludden, constructed in accordance with the current performance-based code. It was built in 1995 and is situated in the southern Swedish city of Växjö. Since then, timber based structural systems in multi-story buildings have become increasingly popular.

Despite the obvious developments during the past century both in architecture itself and in the technical and economic prerequisites for it, there is a lack of knowledge and experience, and of major developments concerning the design of wooden frame systems and of methods of stabilizing multi-story wooden structures. There is a need of extensive research in this area. Questions of particular interest concern the development of more

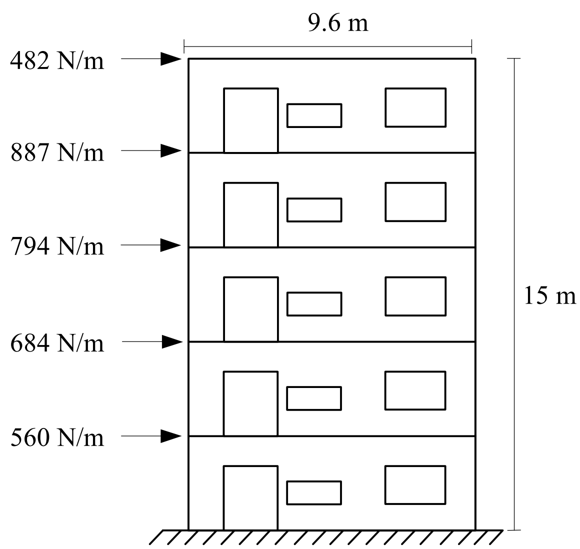


Figure 2. A simplified section of a five-story building showing the horizontal force of the wind load acting on the floor at each level.

efficient production systems relating to this for use in factories and at building sites, and also methods and models for analysing the structural behaviour of such systems when subjected to loading. The task of constructing light weight structures based on wooden framework performing well when subjected to strong horizontal forces such as wind loads can create is particularly challenging.

Wind loads on multi-story buildings can be very heavy, with the large wind-catching area such building possess and in view of the wind speed also being greater high above the ground. In addition, the large sections of glass often used in modern architecture place high demands on the stabilizing system. Figure 2 shows a simplified section of a five-story building, indicating the horizontal force of the wind load acting on the floor at each level.

The present paper aims at providing a brief overview of some of the systems available for stabilizing tall wooden structures, and at discussing construction connected with this and the need of methods and models for analysing the structural behaviour that use of such systems involves.

2. Stabilization by use of shear walls or shafts

Systems for stabilization purposes can be divided into those providing stabilization buy use of shear walls and those providing it by means of shafts and stairways. Figure 3 shows a schematic drawing of the two principles, those of (a) shear walls and (b) shafts. The principle involved in use of shafts and stairways is relatively simple. Robust elements having continuity act as stiff columns, are firmly connected to the ground and are able to carry the horizontal loading transferred to them at each level by the floor. Stabilization through use of shear walls, on the other hand, is less straightforward, since the transmission of forces in the shear walls is complicated. The stiffness and strength of the connections between the different wall elements and between the walls and the floor elements, and the occurrence of openings for doors and windows, are of central importance here. The presentation that follows covers traditional shear walls consisting of sheeting nailed onto wooden frames, and solid timber elements made up of several crossed layers of wood glued together. Stabilization by use of shafts will be discussed briefly and exemplified. The stiffness of the different systems might vary considerably, Ellis et al. (2001).

2.1 Shear walls with nailed-on sheeting

Shear walls having sheeting that is nailed-on are commonly employed for stabilization purposes, especially in low buildings. Figure 4 shows a simple wall in which boards are nailed to a wooden frame. Since the board is very stiff in resisting in-plane deformations the wall’s stiffness depends on the stiffness of the nails and the spacing between them.

Since in low buildings the stabilizing capacity of such a system is very high relative to the horizontal loading which is present, a moderate number of nails is normally sufficient. Relatively simple hand calculations, carried out to be on the safe side, cf. Källsner et al. (1995), can be used for analyzing the system. Figure 5 shows the design

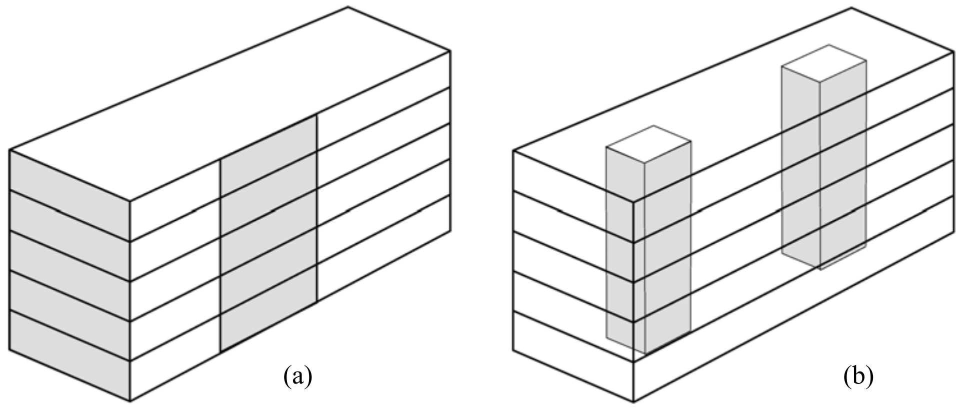


Figure 3. Principles of stabilization by use of (a) shear walls and (b) by shafts.

approach employed. Only the shear stiffness of the continuous vertical strips that are marked in grey in the Figure 5 is accounted for in the analysis. For taller buildings it is much more difficult to obtain sufficient stabilizing capacity, and the design approach of utilizing only continuous vertical strips of shear walls for stabilization purposes is both inconvenient and uneconomical. As calculations indicate, such an approach can lead to unacceptable deformations occurring and to very strong and concentrated uplift forces relative to the ground and to the floors on the lower levels developing.

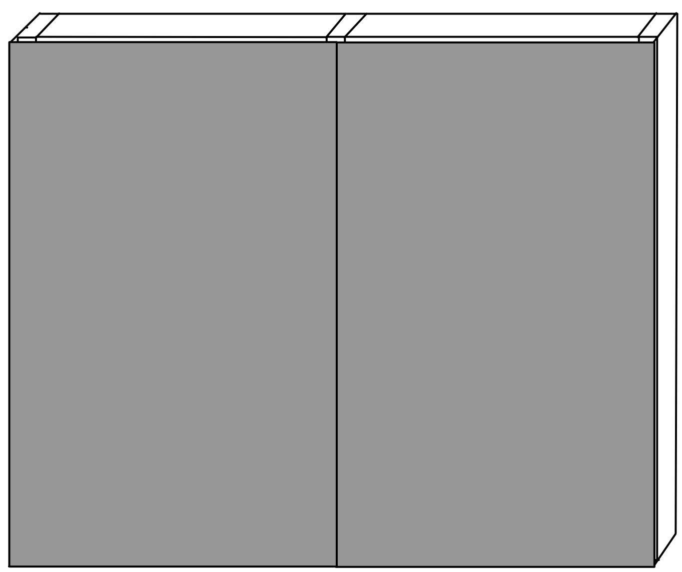


Figure 4. A simple shear wall with boards nailed to a wooden frame.

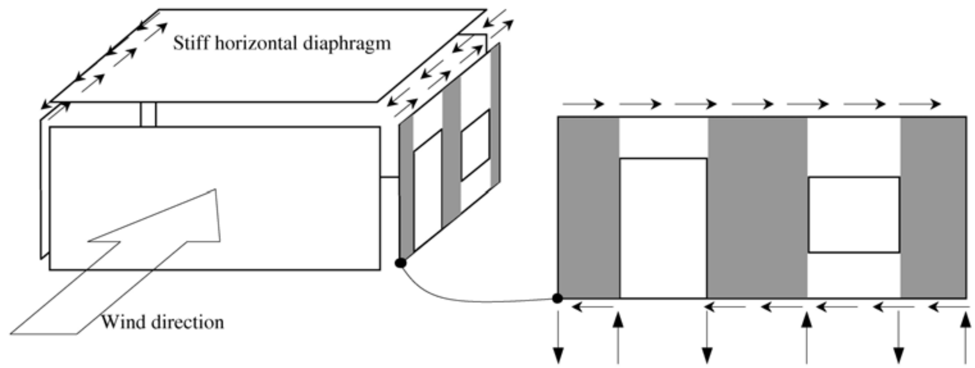


Figure 5. A traditional stabilizing approach used for timber-frame structures. Only the vertical, continuous (grey) strips are accounted for in the analysis.

In reality, elements other than continuous vertical strips in the stabilizing walls can contribute to the overall stiffness of the walls. The possibilities for utilizing this are particularly important in tall buildings. In order to analyze adequately the effects involved, elaborate models making use of the finite element method need to be employed. The development of such models, along with the development of strong components for stiff force distribution at critical positions, i.e. close to openings for doors and windows, are of importance and are in need of further investigation.

2.2 Solid timber elements in shear walls

An alternative to use of shear walls with nailed on sheeting is the use of solid wooden elements consisting of several layers of wood successively glued together perpendicular to each other. Figure 6 shows a drawing of such an element. Such elements are manufactured in dimensions and thicknesses that differ, depending on the purposes to which they are to be put. Further details regarding their technical properties are given by Blass (2004) and by Dujic (2004).

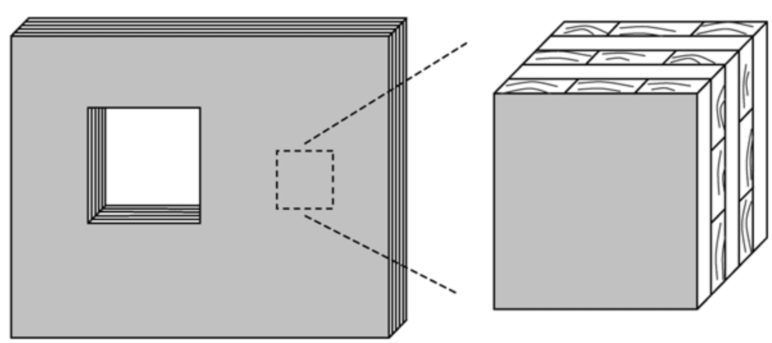


Figure 6. Wooden elements consisting of several layers of wood glued together successively perpendicular to each other.

Figure 7. A four-story building under construction that has walls of solid timber elements.

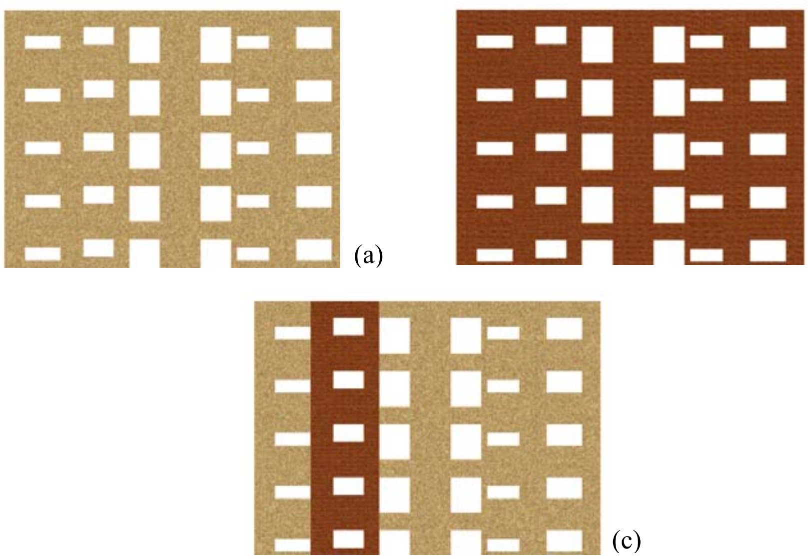
Shear walls of solid elements are much stiffer, of course, than traditional shear walls with nailed-on sheeting are and have the capacity as well of distributing forces around openings for doors and windows. Since some half of the layers of the elements involved are oriented vertically the capacity for vertical loading is also good and very tall buildings and to create challenging architecture, it may be possible in constructing with use of solid wooden elements. Figure 7 shows a four-story building under construction that has solid wooden walls. The construction is built in Trondheim, Norway.

Figure 8. Stiff in-plane connections can be obtained by use of hard board, both screwed and glued to adjacent elements so as to join them.

When solid timber elements are used for stabilization purposes, the connections between these elements, and between these and the surrounding horizontal parts of the system, are key factors in the structural performance of the system and the demands regarding stiffness and strength that are to be met. If them connections are not adequate, the qualities of the solid elements cannot be fully utilized. At least in the case of in-plane connections between elements, very stiff joints can be obtained by use of hard board screwed and glued to the adjacent elements so as to join them, Vessby (2004). Figure 8 shows such a connection.

2.3 Combinations of traditional and solid walls

The quite different shear walls described above, those with nailed-on sheets and solid walls, respectively, can be combined in different ways aimed at finding economically competitive solutions of architectural interest. Figure 9 illustrates (a) a wall stabilized by nail sheathing (b) a solid timber wall and (c) a wall combining solid elements and nailed-on sheathing. Having already discussed the first two alternatives, alternative (c) will now be considered. This sort of wall can be analysed in a manner similar to what was described above for low buildings with walls involving nailed-on sheets and that only utilize vertical continuous strips for stabilization. It may be reasonable in such a case to consider the entire strip of solid material (the dark strip in Figure 9(c)) to provide stabilization to resist horizontal loading, despite it containing openings for windows. This strip is stiffer and broader, than continuous vertical strips of nailed-on sheets are, but for tall buildings the uplifting forces in the lower parts on the one side of the solid elements



(a) (b) (c)

Figure 9. Sections of shear walls stabilized by use of (a) nailed-on sheeting alone, (b) solid timber elements alone and (c) a combination of solid timber elements and nailed-on sheeting.

will still be very strong. The contribution to the overall stabilization provided by the parts of the wall on which there are nailed-on sheets may also of course be considerable, but elaborate models as well as knowledge regarding the stiffness, strength and deformation capacity of various details of the wall, are required to analyze it properly.

2.4 Stabilization by shafts and stairways

A method for stabilizing tall timber structures alternative to that of using shear walls is to employ an elevator shaft or a stairway for this purpose. These parts of the structure can be constructed of concrete, steel or timber and represent the main stabilizing system. The system of joists in the system serves as a stiff diaphragm and distributes the forces from the walls to the shaft. The connection between the diaphragm and the shaft is clearly of considerable importance for the functioning of the system. An example in which a concrete elevator shaft is used for stabilization of a timber structures is the building Uppfinnaren in Växjö, shown in Figure 10.

3. Conclusions

Because of the long tradition in Sweden and many other European countries of only building timber structures up to two stories in height, the stabilization of multi-story timber structures has mainly followed the same principles as in the construction of low houses buildings of this sort. Use of nailed-on sheets to stabilize walls is often an appropriate choice for taller buildings as well, although advanced models for analyzing the structural behavior of such buildings are needed in order to take adequate account of the stabilization that the walls provide.

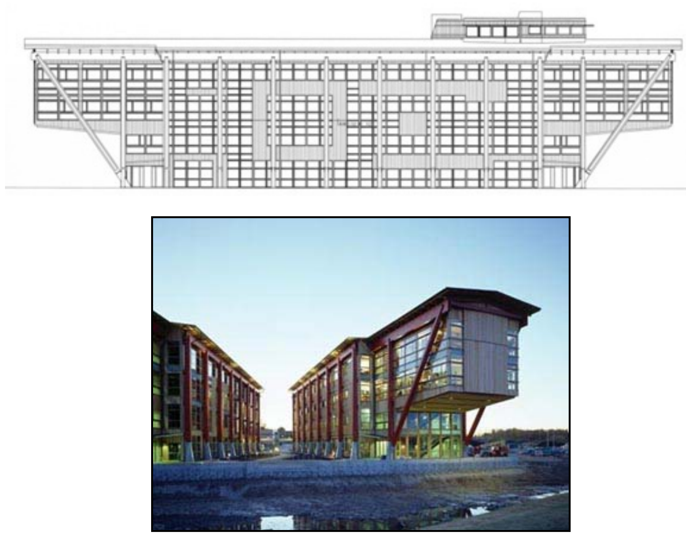


Figure 10. Example of a timber structure stabilized by elevator shafts.

Other types of shear walls can be constructed of solid elements consisting of several layers of wood glued successively at 90-degree angles to each other. This makes the elements very stiff and improves the possibilities of finding modern architectural solutions that are very challenging in character. Such elements can be used in the structure as a whole or as stabilizing elements in parts of the shear walls.

Since the ability of methods and models employed to adequately analyze the structural behavior of the stabilizing systems is very important, in order to enable the potential qualities of the systems created to be fully utilized. Further research concerned with the appropriate modeling of stabilizing systems is needed. Research on the modeling of still taller wooden buildings than those considered heretofore may also be needed as new structural components suitable for handling force distributions of many different types in interfaces between a variety of further elements are developed.

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Paper II

Modelling aspects of wooden shear walls with nailed sheathing

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Paper in preparation for publication.

Modelling aspects on wooden shear walls with mechanically fastened sheathing

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Abstract

The present study concerns the influence of different modelling assumptions on the structural behaviour of a single wall element subjected to monotonic loading. Two types of sheathing-to-framing connection models are studied in the numerical simulations, a single spring model and a spring pair model. The nonlinear behaviour of the sheathing-to-framing connections is investigated using a nonlinear elastic model and a plastic model. The influence of the ductility of the sheathing-to-framing connections is investigated varying the descending part of the load-displacement curves of the connections. Five models of the stud-to-rail connections representing different stiffness properties are also evaluated. The results from FE-simulations are compared with results obtained from testing of wall assemblies consisting of one and three sheets. The overall agreement between simulations and laboratory tests is good when reasonable model assumptions are used.

Keywords: Shear wall, Stabilization, FEM, Wood, Framing, Connection, Sheathing, Stud, Rail, Fastener

1 Introduction

For a long time timber has been the common choice as framework material for one and two storey domestic buildings in many countries. In Sweden for example about 90% of all domestic houses up to two storeys are constructed with stabilizing shear walls consisting of sheets mechanically fastened to a timber-frame. However, since fires devastated several cities before and during the nineteenth century, timber structures higher than two stories were not allowed during the major part of the twentieth century. Consequently none or very few high buildings with timber-frames were built until these regulations were changed in the mid 1990ties in favour of performance-based codes in which requirements were no longer tied to the material itself. Nowadays no restrictions are made with respect to the number of storeys as long as the performance-based requirements are fulfilled.

Even though the development in architecture in the past century is obvious, as well as other technical and economical prerequisites, there has been a lack of development and experience concerning design of stabilization systems for high wooden buildings. Methods and models to analyse the structural behaviour need to be further developed and knowledge concerning acceptable simplifications in calculations and models for practical design situations are required.

Shear walls made of sheets mechanically fastened to a timber-frame are likely to maintain a position as a competitive alternative for multi-storey buildings in the future even though some alternative systems for stabilization of high wooden houses have been developed in the past years and been used in practise, see Vessby and Olsson (2006).

The sheet material used in sheathed shear walls is usually relatively stiff with regard to in-plane deformations and the shear stiffness of the walls depends mainly on the stiffness properties of the sheathing-to-framing connections. When used in low buildings the stabilizing capacity is normally high compared to the actual horizontal loading. Therefore a moderate number of fasteners are sufficient, and simplified hand calculations based on linear elastic connection characteristics, see e.g. McCutcheon (1985), may be used for the analysis of the system. For higher buildings or buildings with many large openings, however, it is much harder to reach sufficient stabilizing capacity. In Sweden a design philosophy is used where the capacity of a wall segment including an opening is disregarded and the wall must be anchored with respect to the full shear force on each side of the opening. A consequence of this philosophy is that a building from a static point of view will consist of a number of continuous vertical strips of shear walls resulting in unrealistically high concentrated tension and compression forces on the foundation and the lower intermediate floor structures.

In reality, the wall segments including openings will contribute to the overall stiffness of the building and the possibilities of utilizing these are important for high buildings. In order to do so sophisticated models using the finite element method (FEM) may be employed in the analysis. Such calculations on single walls have been carried out by e.g. Dolan and Foschi (1991) and Filiatrault (1990) and on entire buildings by Gupta and Kuo (1987), He et al. (2001) and Kasal and Leichti (1992). The use of thorough models, in combination with laboratory tests on different components and connections makes it

possible to understand the overall behaviour of the system. The analyses are also useful for the formulation of requirements regarding stiffness and strength of critical details such as anchorage devices with regard to vertical uplift and reinforcements around door and window openings.

The development within computational mechanics and computer hardware and software has led to that it is nowadays possible, although sometimes very demanding, to simulate the behaviour of almost any structure in detail. For practical design situations it is however important to use models that are able to capture the structural behaviour and the material properties in sufficient detail without spending more computational effort than necessary. Therefore the abilities and limitations of different model alternatives, advanced and simplified, should be thoroughly evaluated. Which model simplifications should be made with respect to the orthotropic material properties of the wood? Is it, for example, important to consider the orthotropic mechanical properties of the sheathing-to-framing connections? Is it sufficient in a particular situation to use elastic (or even linear elastic) load-displacement relations for the connections? Which simplifications concerning the interaction between timber members in the connections should be accepted, and how do the load configurations considered in the analysis influence the structural behaviour?

2 Aim and scope

The aim of this work is to evaluate by numerical simulations the influence of different modelling assumptions on the structural behaviour of a shear wall. The first of two major concerns is the modelling of the connections between the timber members and the sheathing material. Models using one or two nonlinear spring elements for representation of a connection (which includes a fastener and the wood and sheathing material in its close surrounding) are evaluated and compared. The significance of assuming elasticity or plasticity regarding the properties of these connections is studied. Also the importance of the ductility of the connections, i.e. how the stiffness and capacity of the wall is effected by the ductility or brittleness of a single connection after it has reached its maximum capacity, is evaluated.

The second concern is the connections between different timber members in the frame of the wall. Several models using beam elements, solid elements, spring elements, hinges and contact conditions in different combinations are examined. For instance models allowing separation between wooden members are compared with models not allowing separation between members for different load cases.

The work is restricted to analyses in two dimensions, i.e. to the behaviour in the plane of a shear wall. Two different walls are examined. The first one is a wall assembled from one sheet of board material fixed by mechanical fasteners to a timber frame. Most of the model comparisons and evaluations are performed considering this wall. The other wall is an assembly of three sheets. The load cases concerned comprise horizontal loading alone or horizontal loading in combination with vertical loading. For both walls only monotonic loading is studied. Performance under cycling loading is not considered. Finally the results obtained using the different models are compared with results from testing of walls.

3 The examined shear wall

The description below of the examined shear wall starts with the structure and geometry of the wall followed by an account for the material assumptions made regarding wood and board material. Finally load-displacement relations valid for the different connections of the wall are presented.

3.1 Structure and geometry

A single wall assembly consists of three studs, each 2310 mm long, and two rails, each 1245 mm long. The stud closest to the loaded side will be referred to as the leading stud. The two rails will be referred to as the top rail and the bottom rail. The dimensions of the framework are 45×120 mm^2 and the centre distance between the studs is 600 mm. The sheet material considered is fibreboard of 8 mm thickness and quality C 40 with the dimensions 1200×2400 mm^2. It is fastened to the framework with annular ringed shank nails, of length 50 mm and diameter 2.1 mm, at a centre distance of 100 mm along the perimeter, and at a centre distance of 200 mm along the centre stud. Figure 1 shows (a) the timber frame of the wall, (b) a detail of the frame with sheet and nails and (c) a sidewise view of the detail.

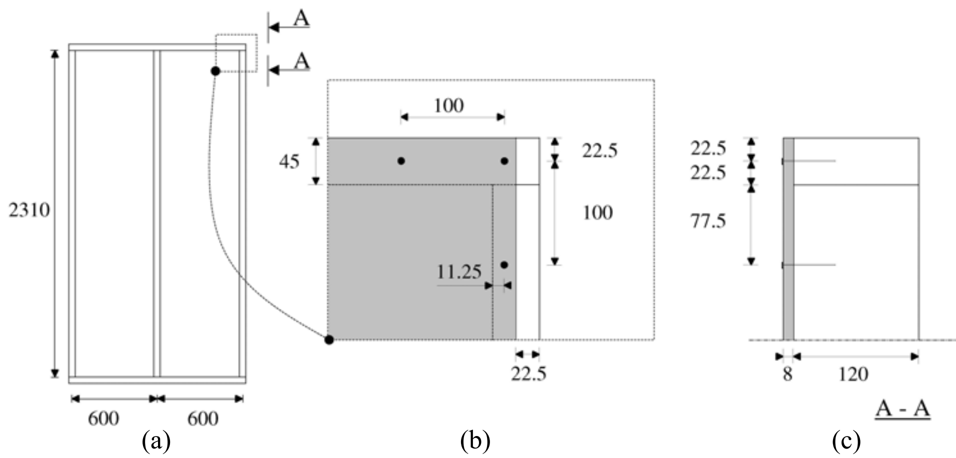


Figure 1. Geometry of the wall assembly showing
(a) the timber frame
(b) detail of framework, positions of sheathing and nails are indicated
(c) sidewise view of the detail.

3.2 Properties of timber members and sheathing material

Wood is an orthotropic material with respect to its elastic properties with three principal directions; longitudinal (l), tangential (t) and radial (r). The stiffness in the directions perpendicular to the longitudinal direction, i.e. perpendicular to the grain direction, is low in comparison with the stiffness in the longitudinal direction. This is of particular interest in regions of the framework where the studs meet the rails. In order to simplify the

material description no difference is made, in any of the models below, between the stiffness in the radial and tangential directions. The longitudinal direction is further assumed to be parallel to the timber members. The material parameters used for the timber are:

$E_l = 12000$ MPa

$E_t = E_r = 400$ MPa

$G_{lr} = G_{lt} = 750$ MPa

$G_{rt} = 100$ MPa

$\upsilon_{lr} = \upsilon_{lt} = 0$

$\upsilon_{rt} = 0.4$

The sheathing material used, fibreboard, is assumed to be isotropic with the modulus of elasticity 6000 MPa and the Poisson's ratio 0.3. Both the timber members and the sheathing material are assumed to have linear elastic properties.

3.3 Load-displacement relations of connections

Two types of connections are included in the wall, the connections between the sheathing material and the wooden frame and the connections between wooden members in the frame. Below the fasteners used are presented as well as load-displacement relations of connections evaluated from tests.

3.3.1 Properties of sheathing-to-framing connections

In the sheathing-to-framing connections annular ringed shank nails with yield strength in the range of 400-500 MPa and with length and diameter of 50 mm and 2.1 mm, respectively, are used as fasteners. In the performed experiments the fasteners are hand-nailed through pre-drilled 1.7 mm holes in the sheet. The load-displacement data used for the connections are based on testing of single fasteners parallel and perpendicular to the length direction of the timber members Girhammar et al. (2004). In Figure 2 these experimental data and average curves that are used below in simulations (bold curves) can be seen. The average curves are based on the average load for a given displacement. In order to achieve a smooth average curve each value on the curve is obtained by weighting the current value with respect to the closest two data points with higher and lower displacements respectively. As shown by the two inserted figures, the sizes of the sheet specimens tested were 150×150 mm^2. In the experiments performed in the parallel direction the length of the timber specimens is 200 mm and in the perpendicular experiment the length is 350 mm. The edge distances of the fasteners with respect to the sheathing and the timber members are 11.25 mm for the specimens tested parallel and 22.5 mm for the specimens tested perpendicular to the longitudinal direction of the timber members, respectively. Further information regarding the performed tests can be found in Girhammar et al. (2004).

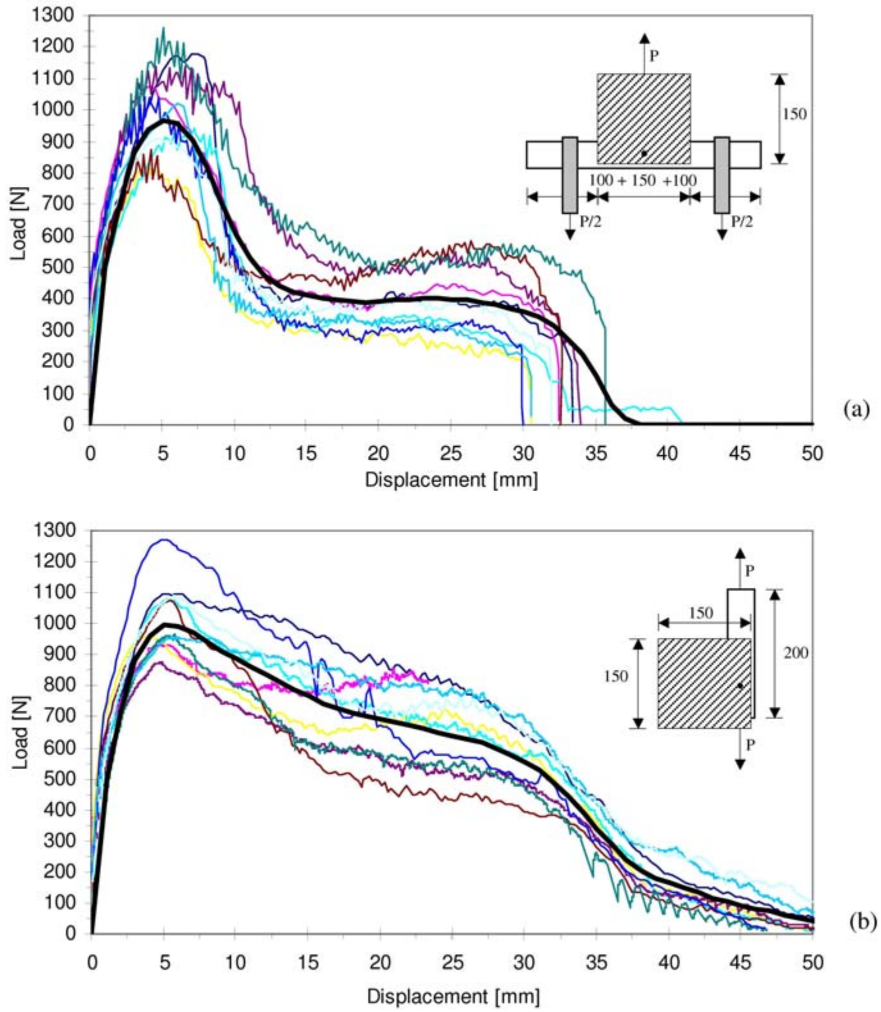


Figure 2. Experimental load-displacement curves and average curves (bold curves) for hardboard-to-framing connections
(a) tested perpendicular and
(b) parallel to the length direction of the timber members.

3.3.2 Properties of stud-to-rail connections

Each stud-to-rail connection consists of two annular ringed shank nails of length 90 mm and diameter 3.1 mm nailed in the grain direction of the vertical studs. In Figure 3 measured load-displacement curves of laterally and axially loaded connections as well as piecewise-linear relations (bold lines) used in simulations below can be seen.

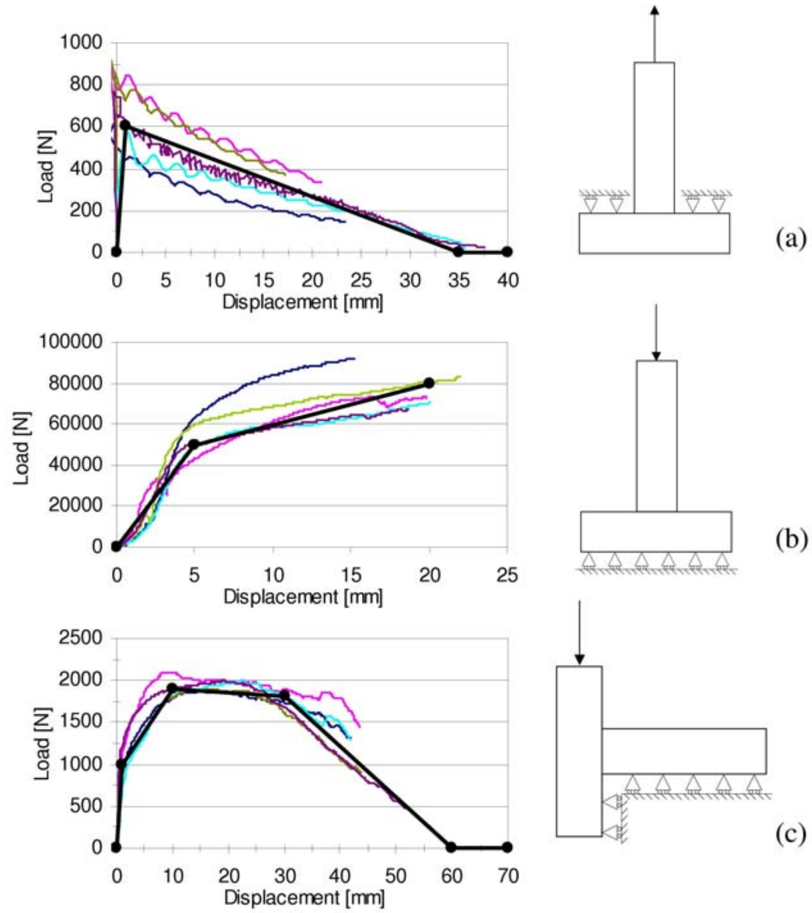


Figure 3. Experimental load-displacement curves and fitted piecewise-linear relations (bold lines) for stud-to-rail connections loaded
(a) in tension,
(b) in compression and
(c) in shear.

4 Modelling of the timber frame and the sheathing material

The modelling of the shear wall described above is carried out using the FE-method. When it comes to the modelling, the wall may be subdivided into four different parts; the wooden members of the frame, the sheathing, the connections between sheathing and framing and the connections between timber members in the frame. The wooden members are modelled using mainly beam elements, but in some cases also using solid elements giving possibility to consider the different stiffness properties in the

longitudinal and transverse direction of the wood material. This is used in areas where forces act perpendicular to the fibre direction and will be discussed further in Chapter 6.

The sheathing is modelled with shell elements. The eccentricity of the sheet material with regard to the centre of gravity of the timber members is however not considered. This implies that only the two-dimensional behaviour of the wall is studied meaning for example that torsional moments acting on timber members are not captured. Though the out-of-plane behaviour of the wall may be of great interest in some respects this limitation is considered reasonable for the purpose of this study.

For the nailed connections between the sheathing and the timber frame and for the connections between wooden members in the frame, i.e. studs and rails, different modelling alternatives are suggested in Chapter 5 and 6 respectively.

5 Modelling the connection between the sheathing material and the timber frame

5.1 Spring models for the connection

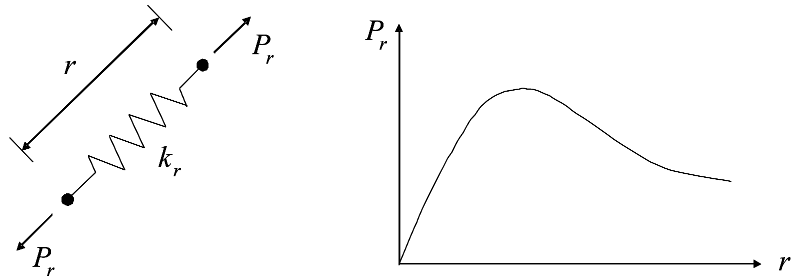
The modelling of the connection between the sheathing and the timber frame is dealt with in this section. Two different models are discussed and employed in the simulations. The first one is the isotropic Single Spring (SS) model which, from a physical point of view, represents a very simple way to model the linear or nonlinear stiffness of a mechanical fastener, including the influence of surrounding timber and sheathing material. From a numerical point of view, however, it turns out that this model has some disadvantages. The second model, the Spring Pair (SP) model is able to represent the stiffness in two orthogonal directions accurately using two uncoupled springs and is from a numerical point of view very stable and simple to implement. Unfortunately, in the case of a nonlinear stiffness relation and when the direction of the displacement does not coincide with one of these two orthogonal directions the stiffness is normally overestimated. In the case of a linear elastic stiffness relation, however, the SS and the SP model will give the same stiffness.

More sophisticated models have been proposed, e.g. Judd and Fonesca (2005), but will not be discussed in this paper. The purpose here is to evaluate the overall influence of choosing one or the other of two different and commonly employed models when evaluating the behaviour of the wall configuration described above.

The influence on the overall behaviour of a wall of using the SS or the SP model for the sheathing-to-framing connections is evaluated and discussed in Section 8.2

5.1.1 Single spring model

In the SS model the sheathing-to-framing connection is represented using only one nonlinear elastic spring, see Figure 4. The current direction of the spring is determined by the relative displacement of the two nodal points to which it is connected. The spring



(a) (b)

Figure 4. Single spring model showing (a) a single spring element in the direction of the displacement and (b) a sketch of a load-displacement relation for a single spring.

stiffness, k_r (which may represent either a tangential or a secant stiffness) and the spring force, P_r , see Figure 4(a), depend only on the magnitude of the displacement and act in the direction of the spring. The orthotropic properties of the wood are consequently not considered. From a physical point of view it only remains to adopt a relation between the spring force, P_r , and the displacement, r . An example of such a relation is sketched in Figure 4(b).

When implementing the SS model in finite element software the stiffness properties of the connection should be expressed by an element stiffness matrix. The element degrees of freedom and corresponding nodal displacements and forces are defined in Figure 5 in local coordinates ($\bar{x}$) following the direction of the displacement of the connection, and global coordinates (*x*, *y*) respectively. In local coordinates the stiffness relation may be expressed as $\bar{\mathbf{f}} = \bar{\mathbf{K}}_{\text{sec}} \bar{\mathbf{u}}$, where $\bar{\mathbf{u}} = [\bar{u}_1 \quad \bar{u}_2]^T$ and $\bar{\mathbf{f}} = [\bar{f}_1 \quad \bar{f}_2]^T$ are the current displacement and force vectors respectively, and $\bar{\mathbf{K}}_{\text{sec}}$ is the current secant stiffness matrix. It may also be expressed as $\Delta\bar{\mathbf{f}} = \bar{\mathbf{K}}_{\text{tan}} \Delta\bar{\mathbf{u}}$ where $\Delta\bar{\mathbf{u}}$ and $\Delta\bar{\mathbf{f}}$ are vectors representing displacement and force increments respectively in the current direction of the spring and $\bar{\mathbf{K}}_{\text{tan}}$ is the tangent stiffness matrix. In global coordinates the relations above can be expressed as $\mathbf{f} = \mathbf{K}_{\text{sec}} \mathbf{u}$ or $\Delta\mathbf{f} = \mathbf{K}_{\text{tan}} \Delta\mathbf{u}$ where $\mathbf{u} = [u_1 \quad u_2 \quad u_3 \quad u_4]^T$ and $\mathbf{f} = [f_1 \quad f_2 \quad f_3 \quad f_4]^T$. The stiffness matrices $\mathbf{K}_{\text{sec}}$ and $\mathbf{K}_{\text{tan}}$ are easily derived by transformation of $\bar{\mathbf{K}}_{\text{sec}}$ and $\bar{\mathbf{K}}_{\text{tan}}$ respectively using the angle θ defined in Figure 5.

An element stiffness matrix representing a connection using the SS model only contains stiffness in one direction, namely the direction of the spring. The rank of the element stiffness matrix is equal to one. This may result in a situation where the numerical model

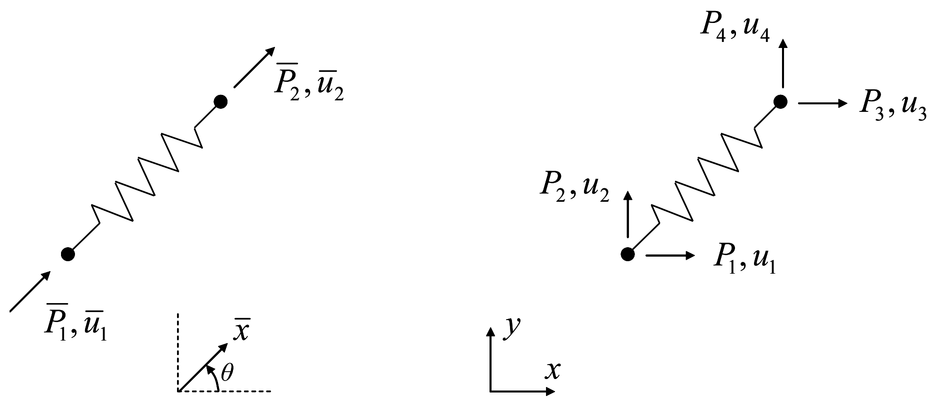


Figure 5. Element degrees of freedom of the SS model expressed in local and global coordinate systems respectively.

representing the wall will contain zero energy modes. To avoid this it may be necessary to add some stiffness to the element stiffness matrices in direction perpendicular to the (primary) spring. If such stiffness is added the element contains stiffness to any displacement in the plane and will be numerically stable, but then it is no longer, strictly speaking, a single spring model.

5.1.2 Spring pair model

The SP model represents the connection by one spring following the longitudinal direction (l) of the timber member, and one spring in the perpendicular direction (p). Figure 6 shows the two orthogonal, uncoupled springs of the model.

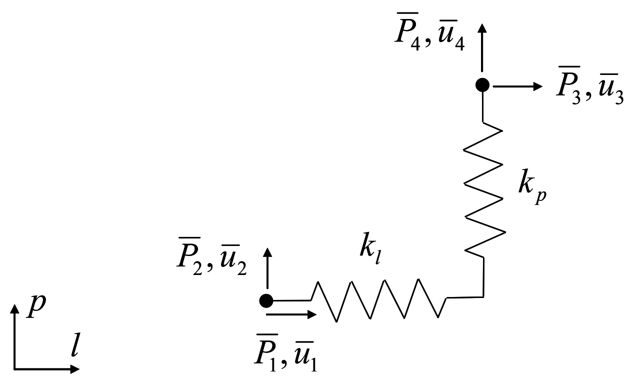


Figure 6. Spring pair model represented by two orthogonal springs following the longitudinal (l) and the perpendicular direction (p) of the timber member, respectively.

Depending on whether a total or incremental formulation is used the relation between displacements and forces in the *l*-*p*-system may be expressed as $\bar{\mathbf{f}} = \overline{\mathbf{K}}_{\sec}\bar{\mathbf{u}}$ or $\Delta\bar{\mathbf{f}} = \overline{\mathbf{K}}_{\tan}\Delta\bar{\mathbf{u}}$ respectively. The stiffness matrices take the form

$$\overline{\mathbf{K}}_{\sec} = \begin{bmatrix} k_{l,\sec} & 0 & -k_{l,\sec} & 0 \\ 0 & k_{p,\sec} & 0 & -k_{p,\sec} \\ -k_{l,\sec} & 0 & k_{l,\sec} & 0 \\ 0 & -k_{p,\sec} & 0 & k_{p,\sec} \end{bmatrix} \quad \text{and} \quad \overline{\mathbf{K}}_{\tan} = \begin{bmatrix} k_{l,\tan} & 0 & -k_{l,\tan} & 0 \\ 0 & k_{p,\tan} & 0 & -k_{p,\tan} \\ -k_{l,\tan} & 0 & k_{l,\tan} & 0 \\ 0 & -k_{p,\tan} & 0 & k_{p,\tan} \end{bmatrix}$$

respectively, with obvious notations. Transformation from the local coordinate system (*l,p*) to the global coordinate system (*x,y*) is easy to perform. An important disadvantage of the SP model is that the uncoupled springs are not able to represent the stiffness accurately when the connection is displaced in another direction than pure longitudinal or perpendicular direction. The stiffness will normally be overestimated because in reality the stiffness in one direction decreases when the connection is displaced in the perpendicular direction, i.e. in reality stiffness in orthogonal directions are certainly not uncoupled.

5.2 Plastic or nonlinear elastic connection properties

In both the SS model and the SP model there are two options to make with regard to treatment of nonlinearity. One can choose to simulate the nonlinear behaviour using an elastic or a plastic model. The difference between the models is related to the unloading behaviour. In the elastic nonlinear model the unloading curve is equal to the loading curve while in the plastic model the unloading curve is determined by the initial stiffness of the load-displacement curve. A von Mises plasticity model with isotropic hardening behaviour is used. This means that the yield surface increases from the initial yield load until maximum load is attained and after that decreases as long as the connection shows softening behaviour. Of course the plastic model is more costly with respect to calculation time than the nonlinear elastic model.

The properties of three different connection models are illustrated in Figure 7 showing the load-displacement curves according to simulations of single connections subjected to loading, unloading and reloading. The three models studied are: the plastic SS model, the plastic SP model and the nonlinear elastic SS model. In all three models the experimentally obtained properties from the perpendicular test are used. In the SP model it is assumed that the two springs are oriented with their length directions at an angle of 45 degrees relative to the displacement direction. As can be seen in Figure 7 the only difference between the nonlinear elastic and the nonlinear plastic model refers to the unloading behaviour. For the nonlinear elastic model the loading and unloading curves are identical while for the plastic model the unloading path is parallel to the initial straight part of the loading curve, cf. the dashed arrows concerning the elastic behaviour and the solid arrows with respect to the plastic behaviour. It can also be seen that there is an obvious difference between the SS and the SP model. As long as the load-

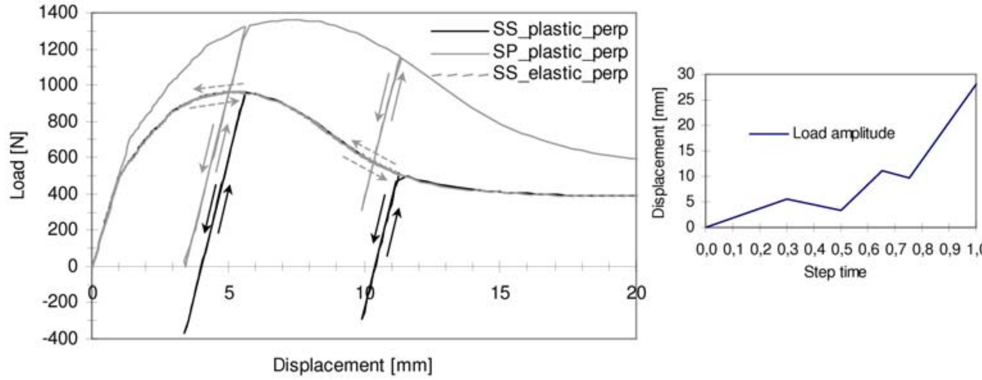


Figure 7. Load-displacement response for a certain load history of a single connection with stiffness properties valid for loading in direction perpendicular to grain in wooden member using the SS model with von Mises plasticity (SS_plastic_perp), the SP model with von Mises plasticity (SP_plastic_perp) and the SS model with elastic properties (SS_elastic_perp).

displacement curve is linear elastic the two curves coincide, but as soon as nonlinearity occurs, the SP model implies a higher capacity than the SS model due to the contribution from two springs. If the SP model is oriented with one of the springs in the direction of the displacement it will give the same load-displacement curve as the SS model.

The influence of using an elastic or a plastic model for the connections is evaluated in Section 8.3.

5.3 Importance of the ductility of connections

In order to investigate the influence of the ductility of connections, i.e. the load-displacement relation of individual fasteners after maximum loading, two artificial load-displacement relations are employed. These represent unlimited ductility and a very brittle behaviour respectively beyond the maximum load level. The curves representing these two extremes are shown in Figure 8 along with the load-displacement curves representing the true relations in the parallel and perpendicular directions respectively. The artificial ductile curve coincides with the curve representing the perpendicular direction until the maximum load-level is reached. The artificial brittle curve coincides with the curve representing the perpendicular direction also beyond the maximum load-level but then drops to zero load-bearing capacity at a displacement of 11.5 mm.

The influence of the ductility of the connections on the mechanical properties of the shear wall examined, is evaluated and discussed in Section 8.4.

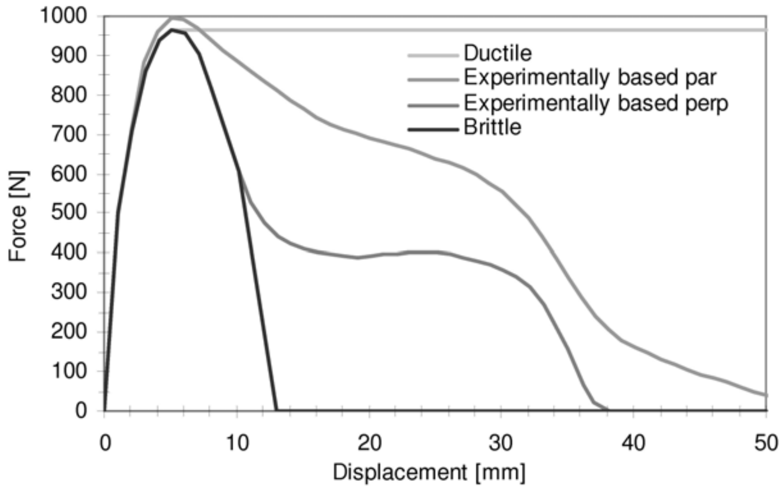


Figure 8. In order to evaluate the influence of the softening behaviour four different assumptions for the load-displacement relations of the connections are investigated. They range from very brittle to very ductile behaviour.

6 Modelling of stud-to-rail connections

The stud-to-rail connections may be represented by advanced or simple models with different ability to represent the true behaviour of the connection in different situations. Five models for the stud-to-rail connections are presented below and evaluated for different load cases in Section 8.5. The five models, illustrated in Figure 9 and summarized in Table 1, may be divided into two groups. Model 1, 2 and 5 belong to the non-separating connection group. These models do not allow the timber members to separate, i.e. the leading stud may not lift from the bottom rail. Model 3 and 4 belong to the separating connection group. These models include spring elements and, in model 4, contact conditions for representing the stiffness of the connection between timber members and allow the leading stud to lift from the bottom rail.

In model 1, 2 and 3 the wooden members are modelled solely by beam elements, while in model 4 and 5 the 200 mm long parts of the wooden members closest to the intersections of the rails and the studs are modelled by solid elements so that transverse stiffness can be accounted for. Other parts of the frame are modelled by beam elements in model 4 and 5 and these models are referred to as combined models in Table 1.

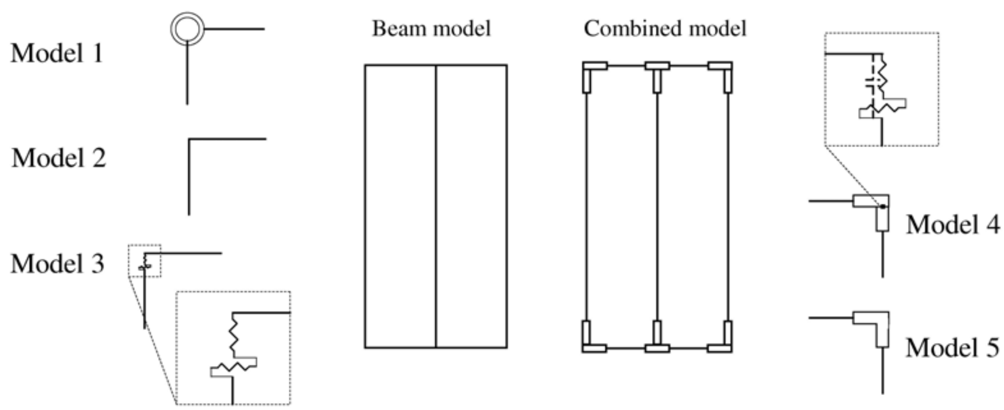


Figure 9. Illustration of alternative models of the framing and for the connections between timber members.

Table 1. Properties of the five models studied.

Model	Model type	Properties
1	Beam model, non-separating connection	- Hinged connection between timber members
2	Beam model, non-separating connection	- Rigid connection between timber members
5	Combined model, non-separating connection	- Rigid connection between timber members
3	Beam model, separating connection	- Coupling between timber members through spring elements
4	Combined model, separating connection	- Coupling between timber members through spring elements and contact condition

6.1 Non-separating connections

Model 1, 2 and 5 do not allow separation between the studs and the rails. In model 1 beam elements representing framing members are hinged connected at the intersection meaning that connections between members are rigid to relative translation but allow free rotation in the plane of the wall. Model 2 differs to model 1 only in the sense that the connection between beam elements is rigid also with respect to rotation. In model 5 the solid elements representing the wood close to the intersections of wooden members are fully assembled, i.e. fully interacting. The difference compared to model 2 is that model 5

captures the low stiffness of wood loaded perpendicular to grain in regions where such loading may occur.

6.2 Separating connections

In model 3 the beam elements are coupled at the intersection points between members by two spring elements. One spring element follows the direction of the stud (vertical) and captures the stiffness to withdrawal of the nails connecting the members as well as the stiffness to compression between stud and rail. Note that only vertical translation of the centre-line of the stud is captured, not stiffness to rotation of the stud in relation to the rail. The other spring element follows the direction of the rail (horizontal) and captures the nail stiffness to shear between rail and stud. Friction is neglected. The piece-wise linear relation for the vertical spring is given in Figure 3(a-b) and for the horizontal spring it is given in Figure 3(c).

Model 4 differs from model 3 as the stiffness to compression between the wooden members, here modelled by solid elements, is mainly captured by contact conditions using Lagrangian multipliers. So-called hard contact is used allowing no penetration of the two surfaces in contact. A spring element is also included in the model in the direction of the stud. In tension it has the same properties as in model 3 but in compression it only adds stiffness corresponding to compressing the nail into the wood (the same stiffness as to withdrawal). The major stiffness to compression is captured by the contact condition. As in model 3 no friction between wooden members is considered and the nail stiffness between members in the direction of the rail is modelled by a spring element with the same properties as in model 3.

7 Load cases and support conditions

The analysis and evaluation of the different model alternatives described in Chapter 5 and 6 is performed using three different load cases, illustrated in Figure 10. In all load cases a point load is applied at the upper left corner of the top rail acting in the plane of the frame. In load case 1 the load is directed horizontally, i.e. $\alpha = 0^\circ$. In load case 2 the load is directed in a 45˚ angle to the horizontal plan, i.e. $\alpha = 45^\circ$ and in load case 3 the load is directed diagonally through the frame, i.e. $\alpha = 63^\circ$. The precise location where the point load is applied is, for all load cases, at the intersection between the centre lines of the two wooden members, see Figure 10. In the beam models, model 1-3 (Chapter 6), this position coincide with necessity with a nodal point, but also in the combined models, model 4-5, a nodal point is located in this position to allow the point load to be applied here.

It should be emphasized that in the simulations presented below the load levels stated are only the horizontal component of the total load and the displacements stated are only horizontal components. The horizontal displacements given are for the right end of the top rail.

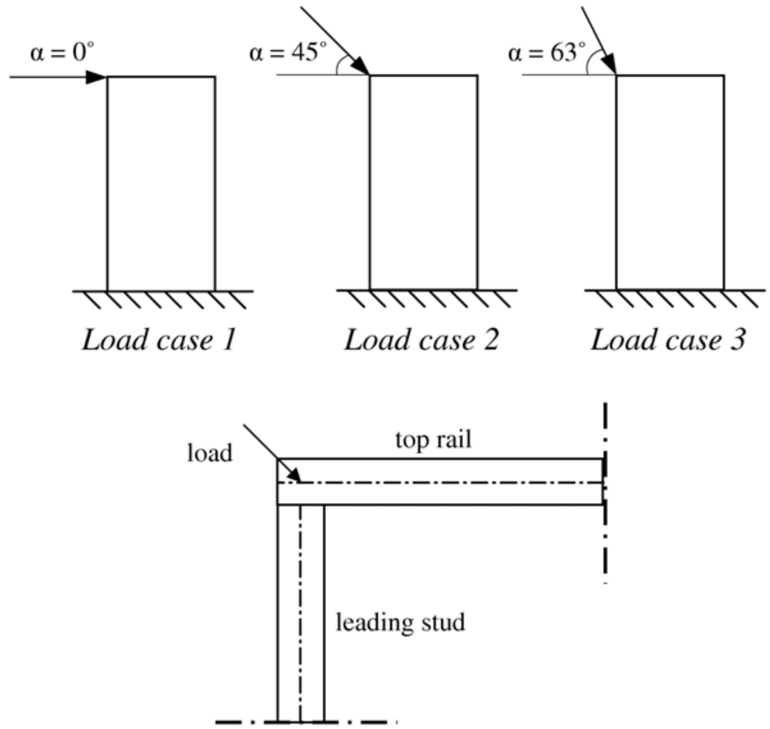


Figure 10. The load cases studied are:
Load case 1 - horizontally applied point load
Load case 2 - point load applied in 45 degrees to the horizontal plane
Load case 3 - point load applied diagonally, at 63 degrees to the horizontal plane.

8 Simulation results and evaluation of alternative models

The wall assembly with a single sheet described in Chapter 3 is analysed below. The behaviour of the wall under loading is investigated and the influence of using the different model alternatives and properties presented above are evaluated. The different load cases presented are used to show the significance of the modelling of various details. A default case concerning modelling, properties and loading, which is used if nothing else is stated, is composed by stud-to-rail connections using model 3 (Chapter 6) and sheathing-to-framing connections modelled by the single spring model (Section 5.1.1) with elastic stiffness properties (Section 5.2) valid in the perpendicular direction (Section 3.3.1).

In the simulations presented below artificial damping is used when necessary during the solution process but it is then always checked that the dissipated energy is only a very small fraction as compared to the total internal energy of the structure.

8.1 Calculated displacement path of fasteners using default model and properties

As described in Section 3.3 the stiffness of fasteners connecting sheathing to framing differs when loaded parallel or perpendicular to the grain in the wooden members. Also, the significance of modelling the fasteners with one or two spring elements depends on the direction of the displacement of the fastener as explained in Section 5.1. Therefore, the displacement path and loading of the fasteners in the wall are presented below.

Figure 11(a) shows the load-displacement relation of the wall element for load case 1, horizontal loading. The model of the wall is the default model described above. Four different load-displacement states along the curve are highlighted, two before and two after the state of maximum loading. The initial position of the sheathing and the displacement paths of the fasteners are shown for the four states in Figure 11(b) in which displacement paths are magnified for visibility. Figure 11(c) shows the corresponding load-displacement state of each fastener (following the curve of the fastener presented in Figure 2a). The most heavily loaded fasteners are those connecting the sheathing to the bottom rail and the direction is close to perpendicular to the direction of the bottom rail, i.e. perpendicular to grain. In the first state no fastener is loaded above half the ultimate capacity, which is approximately 960N (according to Figure 2a). In the second state a few fasteners are approaching their ultimate capacity and in state three the important fasteners connecting the left part of the sheathing to the bottom rail have passed their ultimate capacity. The fourth state presented show that the load capacity of the wall decreases as a consequence of the decreasing capacity of the most important fasteners. In all four states other fasteners than those connecting the sheathing to the bottom rail are not loaded above the initial, close to linear part of the curve. Due to the limited capacity to handle the uplift forces between the sheathing and the bottom rail a maximum capacity of the wall of 3.2 kN is reached at a 19 mm displacement of the upper right corner of the wall.

For load case 3, i.e. diagonal loading, the wall is evaluated in the same manner as described above for horizontal loading. The load-displacement relation of the wall, with four highlighted states and corresponding displacement paths and load-displacement states of the fasteners are shown in Figure 12. The diagonal loading comprise a vertical component that counteracts the uplift forces related to the horizontal component of the loading. This result in a very different situation than what is the case for load case 1 and a very much higher capacity of the wall is achieved. The horizontal component of the loading reaches a maximum of 13 kN at a horizontal displacement of 35 mm of the upper right corner of the wall.

It is clear from Figure 12(b), showing the displacement path of the fasteners, that fasteners close to the corners are those primarily contributing in transferring the force between the frame and the sheathing. Initially those fasteners are loaded in a mixed direction including both horizontal and vertical displacement components. The studs in the frame deform slightly in S-forms. For higher load levels the fasteners transfer forces mainly in vertical direction. The load-displacement states of the fasteners, see Figure 12(c), shows that a few fasteners are approaching their ultimate capacity in state one and that a few have exceeded their ultimate capacity in state two which corresponds to a

loading on the wall where the ultimate capacity is almost reached. For state three and four, past the ultimate capacity of the wall, the capacity is decreasing slowly with increasing displacement. This slow decrease in capacity, i.e. a ductile behaviour, is due to the fact that quite many fasteners contribute significantly in the force transfer between the frame and the sheathing.

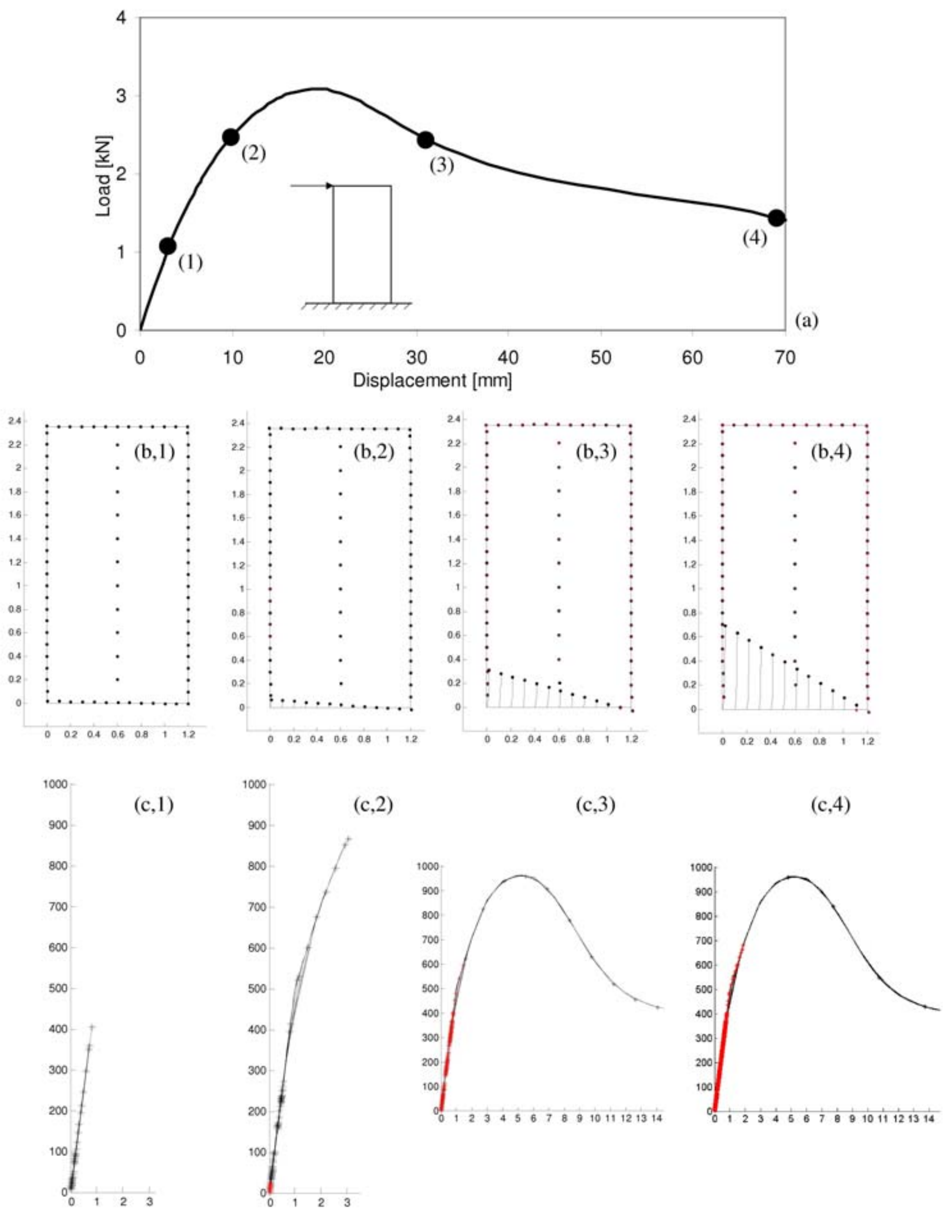


Figure 11. (a) Load-displacement curve of a horizontally loaded wall element with four selected states indicated. For these states are shown (b) exaggerated displacement path of all the fasteners and (c) the corresponding state of each fastener on the load-displacement path valid for individual fasteners.

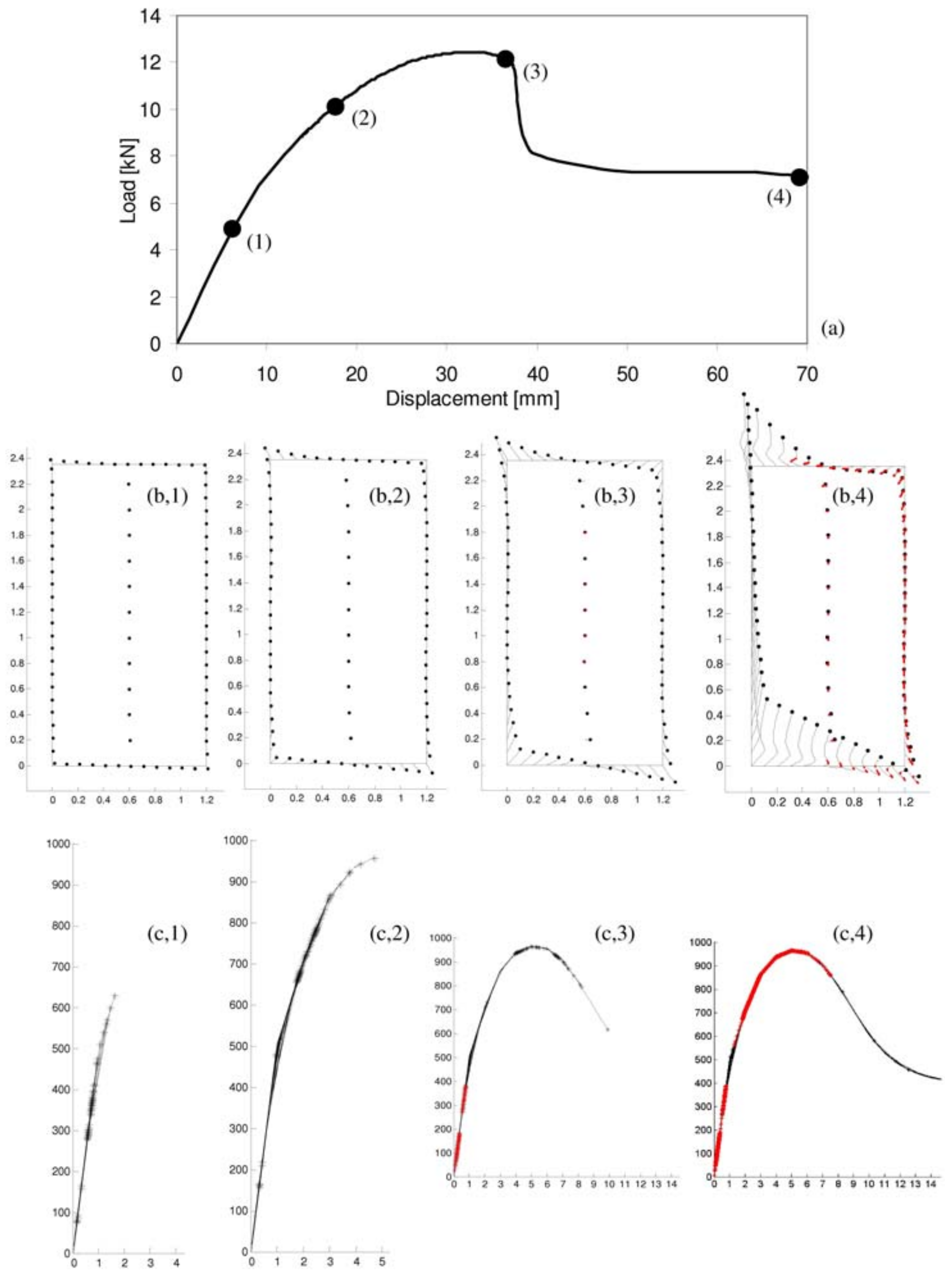


Figure 12. (a) Load-displacement curve of a diagonally loaded wall element with four selected states indicated. For these states are shown (b) exaggerated displacement path of all the fasteners and (c) the corresponding state of each fastener on the load-displacement path valid for individual fasteners.

8.2 Spring models for the sheathing-to-framing connections

The purpose now is to investigate and compare the overall influence on the behaviour of the wall when using the SS model and the SP model respectively. We know from Chapter 5, Figure 7, that the SP model for a single connection may give higher stiffness than the SS model, and that the SS model and the SP model underestimates and overestimates, respectively, the stiffness of a connection when the load-displacement relation is not linear. The significance of this effect on a wall depends, however, on the actual displacement path of the fasteners and on the degree of nonlinearity of the stiffness at a given displacement state. To be able to evaluate the significance of using the SS model or the SP model, and identify the actual reasons of the differences, not two but five different connection models will be tested and compared: the SS model with the load-displacement relation valid for the parallel and perpendicular directions respectively (SS_par and SS_perp), the SP model with one spring acting according to the load-displacement relation of the parallel direction and the other acting according to the perpendicular direction (SP_par_perp), and finally the SP model with both springs acting either according to the parallel or the perpendicular load-displacement relation respectively (SP_par and SP_perp respectively). An overview of the five alternatives is given in Table 2.

For comparison and evaluation load case 1 and load case 3, see Figure 10, i.e. horizontal and diagonal loading are employed. Of the structural models presented in Chapter 6 only model 3 is used in the present evaluation. Figure 13(a-b) shows, for the five different connection models defined in Table 2, the simulated load displacement relations for the wall, with reference to the horizontal component of the displacement at the upper right corner of the upper rail and the horizontal component of the load. In Figure 13(a) the response is shown for load case 1 and in Figure 13(b) it is shown for load-case 3.

Table 2. Properties of the connection models used.

Model	**Properties**
SS_perp	<u>SS</u> model using the stiffness relation valid in the direction <u>perpendicular</u> to the direction of the timber member.
SS_par	<u>SS</u> model using the stiffness relation valid in the <u>parallel</u> direction of the timber member.
SP_par_perp	<u>SP</u> model using the stiffness relations valid in the <u>parallel and perpendicular</u> direction respectively.
SP_perp	<u>SP</u> model using, for both springs, the stiffness relation valid in the <u>perpendicular</u> direction.
SP_par	<u>SP</u> model using, for both springs, the stiffness relation valid in the <u>parallel</u> direction.

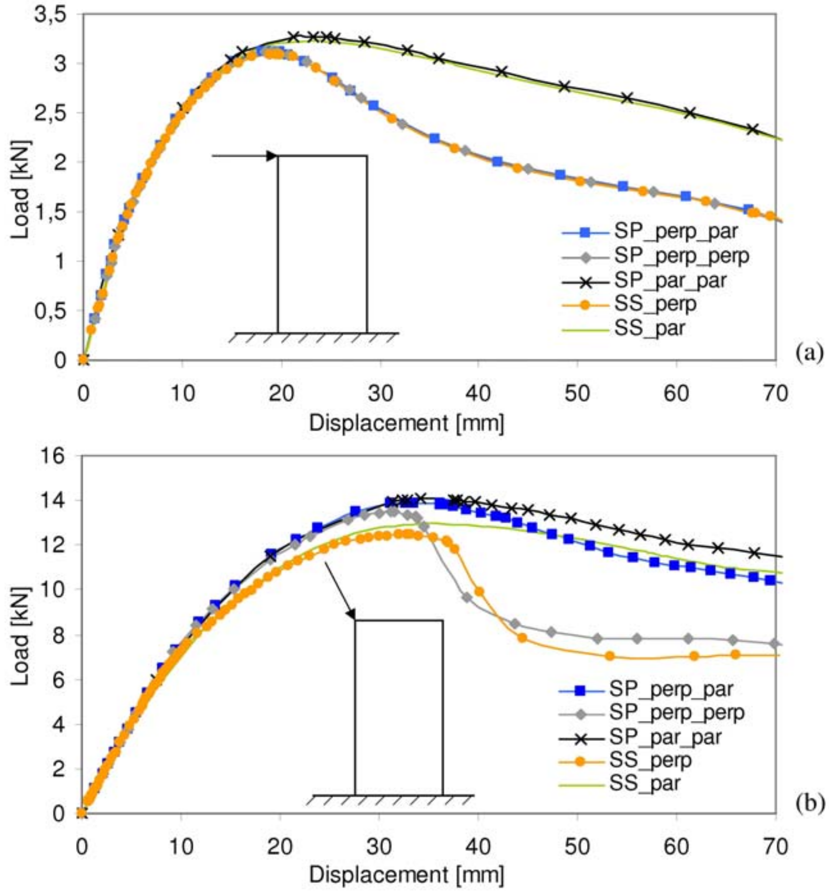


Figure 13. Results from simulation of (a) load case 1 and (b) load case 3 subjected to the wall using five different connection models.

For low load levels only very small differences appear due to the use of the SS model or the SP model and it does not matter if the connection stiffness valid in the perpendicular or the parallel direction is used. This is not very surprising as for low load levels the two stiffness relations are very similar, see Figure 8, and also close to linear.

For higher load levels almost all difference in response occur as a consequence of the difference between the stiffness curves valid for the perpendicular and the parallel direction respectively. The displacements in load case 1 are dominated by connection displacements in the direction perpendicular to the bottom rail since the uplift force must be transmitted by these connections, see Figure 11. Since the leading stud is lifted and the displacements of the fasteners to a large extent follow the perpendicular direction there will be no significant effect of over-stiff behaviour using the SP model.

A significant difference can be seen on the calculated load-displacements curves obtained by using SS_perp and SP_perp on one hand, with stiffness relations valid for the perpendicular direction, and models SS_par and SP_par on the other hand, with stiffness relations valid for the parallel direction. In SP_par_perp the characteristics in the bottom left corner is to a high extent given by the perpendicular spring of the two in the model. This explains why the load- displacement curve is coinciding with SS_perp and SP_perp. It is obvious that the softening behaviour of the connections, which differs a lot, has a fairly large influence on the load-displacement relations of the wall for large displacements. However, the influence on the ultimate load of the wall is for the studied case moderate (6%).

In load case 3 there are considerable displacements in both perpendicular and parallel direction of different sheathing-to-framing connections, see Figure 12. Therefore some connections are displaced in a direction not coinciding with the perpendicular or parallel direction, and as a consequence the SP model (SP_par_perp, SP_par and SP_perp) overestimates the stiffness and the load-carrying capacity of the wall. The difference in capacity is, however, less than 10% due to the over-stiff-effect of using the SP model. This can be concluded by comparing the behaviour of SS_perp with the behaviour of SP_perp, and by comparing the behaviour of SS_par with the behaviour of SP_par. The simulation also shows, however, that after the maximum load has been attained the main difference in response is due to the employed load-displacement relations of the connections rather than to the use of the SS model or the SP model. The SS_perp and SP_perp models, which only use the stiffness relation valid in the perpendicular direction, show a less ductile behaviour than the other models.

Comparing the load-displacement relations on the wall level, Figure 13, with the relation on the single fastener level, Figure 7 it is clear that the difference on the wall level due to the choice of SS or SP model is comparatively small. The reason is that only a small part of the springs are loaded beyond the initial, linear part of the curve in a mixed direction significantly deviating from the parallel or perpendicular direction. The conclusion is therefore that the choice between the SS and the SP model has a limited, although in certain cases significant effect on the simulated behaviour of a wall.

8.3 Plastic or nonlinear elastic sheathing-to-framing connection models

In Figure 7 the difference between a plastic or nonlinear elastic model for a single sheathing-to-framing connection is illustrated for a load case including loading and unloading of the fastener. During unloading from a state above say 2 mm, i.e. from a state in the nonlinear zone, the difference between the elastic and plastic response is large.

If no unloading takes place, however, there would be no difference between the elastic or plastic response. On the wall level individual fasteners may be unloaded even though the loading on the wall level is a monotonic increase in displacement of the upper left corner of the wall as in the load cases employed herein.

Simulation results of load case 1 and 3 are shown in Figure 14(a) and Figure 14(b) respectively using both nonlinear elastic and plastic connection models. For load case 1

no difference between the elastic or plastic response is found showing that no significant unloading takes place in individual fasteners. This is not very surprising considering the rather straight displacement paths of the fasteners presented in Figure 11, i.e. mainly displacement in vertical direction of fasteners connecting the sheathing to the bottom rail.

For load case 3 some unloading of fasteners obviously occurs as the load-displacement relation on the wall level differs using the plastic model compared to the elastic one, see Figure 14(b). Also the displacement paths of the fasteners presented in Figure 12(b) show that the displacement path of the fasteners connecting the sheathing to the bottom rail on the right part change direction during loading which includes unloading of these fasteners. The resulting difference on the wall level of using a plastic model instead of an elastic model is, see Figure 14(b), limited to the behaviour of the wall after the ultimate load is reached. Past the ultimate load the plastic model shows a less ductile behaviour than the elastic model. If such differences, i.e. inaccuracies due to using the elastic model, are not considered as acceptable it must be checked after calculation that no fasteners are unloaded during the simulation before relying on the results.

From the evaluation considering load cases 1 and 3 it may be concluded that an elastic model for the fasteners is sufficient to capture the behaviour of the wall. It should then be noted, however, that such a conclusion would hardly be valid if the load case on the wall level includes unloading as for example cyclic loading does. Then an elastic model would most likely be insufficient and a model for fasteners would probably require a plasticity model with kinematic hardening/softening rather than isotropic hardening/softening. Such an evaluation is, however, beyond the scope of the present investigation.

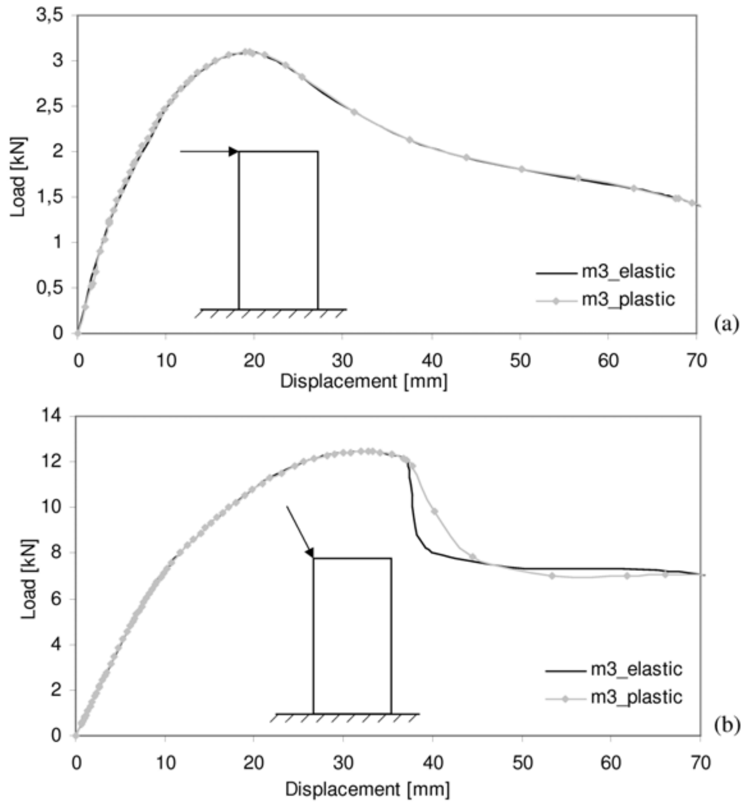


Figure 14. Simulation results showing the load-displacement relation of a wall using elastic model assumption for connections (m3_elastic) and plastic model assumption (m3_plastic) respectively for (a) load case 1 and (b) load case 3.

8.4 Importance of the ductility of sheathing-to-framing connections

The ductility of sheathing-to-framing connections discussed in Section 5.3 is evaluated below. Load case 1 and 3 are simulated and for comparison each of the four different connection stiffness curves presented in Figure 8 are employed for modelling the properties of the connections. The results from the simulations are shown in Figure 15(a) and (b) for load case 1 and 3 respectively. The ultimate capacity does not differ much depending on which connection stiffness curve is employed for any of the two load cases but a slightly higher capacity is achieved using the curve representing a fastener with unlimited ductility or the curve valid for the actual fastener loaded in direction parallel to grain. (The latter curve gives a slightly higher ultimate capacity on the level of a single fastener than the other curves.)

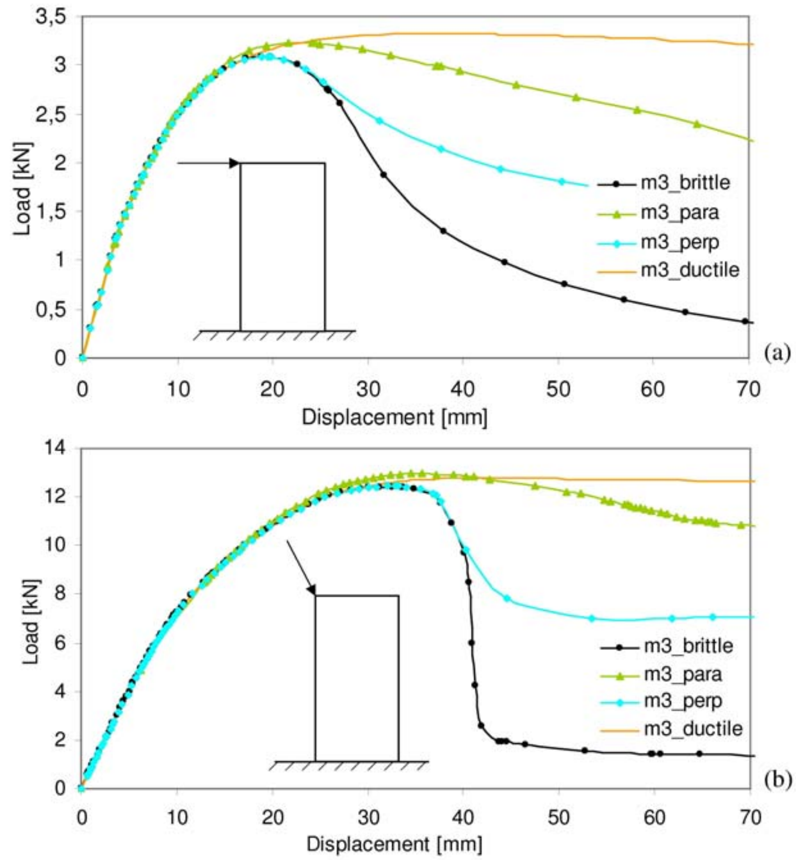


Figure 15. Simulation results showing load-displacement relation of a wall for (a) load case 1 and (b) load case 3. Stiffness curves employed represent brittle fastener connection (m3_brittle), actual fastener loaded in direction parallel and perpendicular to grain (m3_para) and (m3_perp) respectively and finally unlimited ductility (m3_ductile).

The difference in ductility on the level of a single fastener is very much reflected on the level of the wall. For example, using the connection stiffness curve representing a brittle connection on the fastener level, when simulating load case 3, result in a rapid drop in the load-bearing capacity. 85% of the ultimate capacity is then lost over a 10 mm additional displacement of the upper right corner of the wall. Also the use of connection stiffness curve representing the actual fastener loaded in direction perpendicular to grain results in a rapid drop after the ultimate capacity is reached. Considerable capacity is however then preserved even for larger displacements. For load case 1 the stiffness curve on the connection level influence the ductility of the wall in a similar way as for load case 3 but the loss of capacity on the wall level for additional displacement on the wall level is not as rapid as for load case 3.

8.5 Evaluation of stud-to-rail connection models

In Chapter 6 five different models for description of the mechanical properties of the stud-to-rail connections were introduced. In this section these models are applied to the wall configuration previously studied. For load case 1 and 3, corresponding to horizontal and diagonal loading respectively, calculated load-displacement curves are shown in Figure 16 using the different models. For load case 1 the properties of the sheathing-to-framing connections in the perpendicular direction of the timber members are used while for load case 3 the properties in the parallel direction are employed. This choice is done since in the case of horizontal loading the most loaded connections are those loaded perpendicular to the grain direction of the bottom rail (see Figure 11) and in the case of diagonal loading the most loaded connections are those loaded parallel or close to parallel with the directions of the framing members.

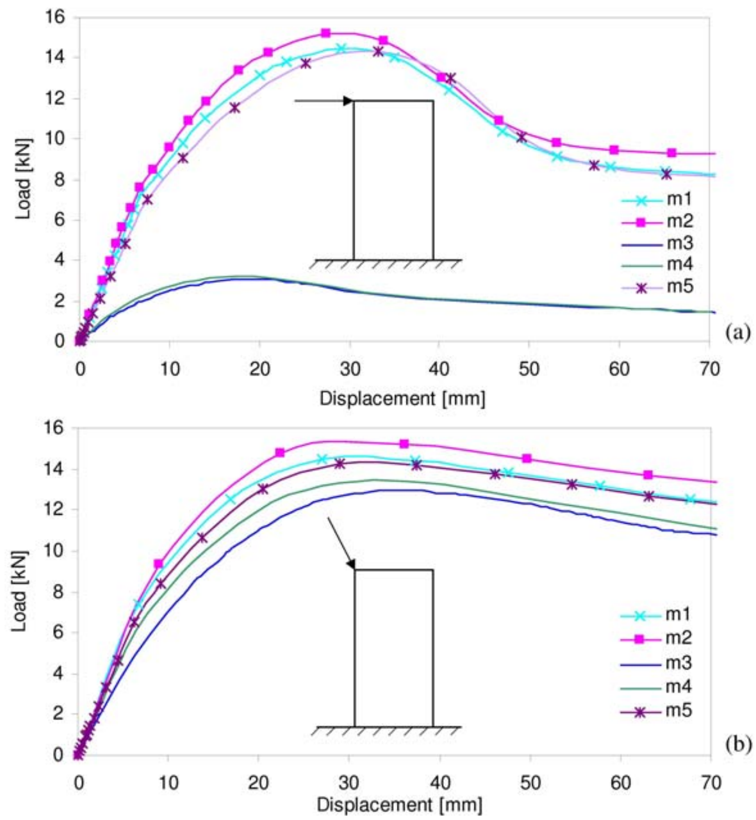


Figure 16. Calculated load-displacement curves for a wall element subjected to (a) load case 1, horizontal loading, and (b) load case 3, diagonal loading, using different stud-to-rail connection models.

It was mentioned in Chapter 6 that the five models can be divided into two groups with respect to their ability to allow separation or not. Thus, the models 3 and 4 allow the timber members to separate while the models 1, 2 and 5 do not allow separation. It can immediately be stated that the models not allowing separation in the stud-to-rail connections do not work for load case 1. The obvious reason is that these models work as if tie-downs had been inserted in the connections, thus, preventing the leading stud from uplift. Consequently, models 1, 2 and 5 can only be used for load case 3 when there is no evident risk of separation in the stud-to-rail connections. Therefore these three models cannot be recommended to be used as general models in analyses of shear walls, especially if it cannot be guaranteed that uplift will not occur.

Of the two relevant models used for the horizontal load case, it is noticed in Figure 16 that model 4 gives a load-displacement curve with slightly higher stiffness and capacity than model 3. The reasons for this deviation will be discussed later in this section.

In Figure 16 it is seen that the load-displacement curves calculated for the diagonal load case have a somewhat higher stiffness and capacity using the non-separating connection models instead of the separating ones. This is a consequence of the fact that no shear displacements take place in the non-separating connection models. As can be expected, model 2 gives rise to a higher load-displacement curve than model 1. This can be explained by the fact that in model 2, the connections between the beam elements are rigid with regard to rotation while they in model 1 are hinged. Since model 5 allows some transformation of bending moments in the stud-to-rail connections it may be somewhat surprising that model 5 gives rise to a load-displacement curve that is lower than the one obtained by model 1 where no bending moments are transferred. This should be a consequence of the low stiffness of the wood material perpendicular to grain in model 5 and it can be observed that this effect occurs not only in the stud-to-rail connections subjected to compression forces but also in the ones subjected to tension forces since the timber members are assumed to be fully interacting in the combined model.

Comparing the response of the connection models 3 and 4 on the load-displacement curve of the diagonal load case in Figure 16, it is observed that the beam model gives a curve with somewhat lower stiffness and capacity than the combined model. There are a few reasons for this deviation between the curves. One of these is the modelling of the compression stiffness. In the beam model the compression stiffness is based on stiffness data determined by testing of real stud-to-rail connections including influence of real imperfections as initial gaps between the timber members and not plane wood surfaces. In the combined model the wooden members are modelled by linear elastic solid elements assuming that the contact between the members is perfect and that there is no penetration of the contact surfaces. Using this assumption the compression stiffness of the stud-to-rail connections using the combined model will be higher than the one using the beam model. Another reason to the lower stiffness and capacity using model 3 instead of model 4 is that the stud-to-rail connection in the beam model is assumed to be hinged with respect to rotation while some bending moments can be introduced into the studs by the combined model. This bending moment originates from the uneven distribution of the compression stresses between the wooden members. Such a stress distribution is shown in Figure 17 for the upper left corner of the top rail in the case of diagonal loading and

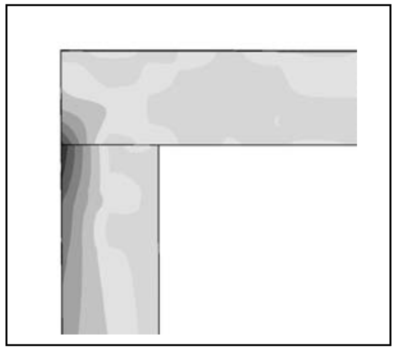


Figure 17. Calculated vertical stresses in the top left corner of the single sheet wall element, using model 4. High stress levels are represented by dark colours.

maximum load. The dark fields indicate high compression stresses in the vertical direction. It must here be emphasized that the stresses have been calculated assuming linear elastic material properties for the wood, resulting in somewhat too large bending moments introduced in the studs.

8.6 Second order effect of vertical load

The second order effects of the vertical load on the leading stud are of two kinds. One is due to the horizontal displacement of the top rail. The other concerns the influence of the axial force on the lateral displacement of the leading stud. The total effect on the wall can be investigated by studying a fully anchored wall element subjected to different magnitude of vertical load on the leading stud. This is carried out using model 2, implying no separation between the wooden members in all stud-to-rail connections. The results of such calculations are shown in Figure 18 for the load cases 1, 2 and 3 using parallel properties for the sheathing-to-framing connections. The horizontal capacities of the three load cases are 14.7, 15.0 and 15.2 kN, respectively. The relative decrease in capacity between load case 1 and 3 is 3.4% and specifies the second order effect due to applying a vertical load corresponding to full anchoring of the leading stud.

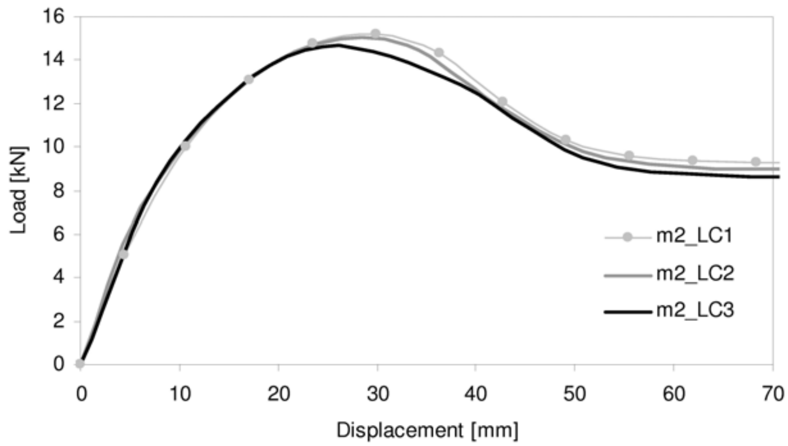


Figure 18. Calculated load-displacement curves for a single sheet wall element subjected to load case 1, 2 and 3 using connection model 2.

9 Comparison between laboratory tests and FE-simulations

In this chapter results from finite element simulations are presented together with corresponding results from experimental tests. Evaluated structures are a wall with only one sheet, i.e. the single wall element used for the comparisons between the models presented above, and a larger wall consisting of three sheets fastened to a wooden frame. The latter wall configuration is introduced in this chapter.

The aim is to verify the finite element modelling used in simulations by comparing with results from experiments. Out of the many models presented above only three different models are considered here. The first one is the default model defined in Chapter 8. The second one is a model that differs from the default model in the sense that the stiffness relation of the sheathing-to-framing connections in the direction parallel to grain is used, i.e. the stiffness relation shown in Figure 2(b) is used for all directions in the plane of the sheathing. (In the default model the stiffness relation shown in Figure 2(a) is employed.) In the third model the SP model is used for the connections between sheathing and wood and different stiffness relations are thus used for displacement parallel and perpendicular to grain.

9.1 Comparisons for a single wall element

The wall described in Chapter 3 is now tested experimentally. Considered load cases are horizontal loading, corresponding to load case 1 in the simulations, and diagonal loading, similar to load case 3 in the simulations. In a test series four specimens were tested for each load case.

9.1.1 Laboratory test setup – load cases and support conditions

All tests where performed under displacement control with a constant rate of 8 mm/min corresponding to a failure time of about 5 minutes. In the horizontal load case the load was applied by a hydraulic jack acting in line with the top rail pressing the upper left corner of the wall to the right. In the diagonal load case the load was applied such that an external force acted on the upper left corner of the wall towards the original position of the bottom right corner of the wall. This means that the angle of the external force to the horizontal plane changes as the wall is deformed which is a difference compared to simulations using load case 3.

For both load cases the bottom rail was prevented from moving by clamping devices placed at equal distances along the entire rail. Further the top rail was prevented from moving out of the plane of the wall. More information regarding the test setup can be found in Girhammar and Källsner (2004).

9.1.2 Results from laboratory tests and simulations

Test results, in terms of the horizontal component of the external force at the upper left corner of the wall as function of the horizontal displacement of the upper right corner, are shown together with simulation results in Figure 19(a-b) for the horizontal load case and for the diagonal load case respectively. For both load cases there are considerable variation in initial stiffness as well as in ultimate load capacity between the experimental tests performed.

For the horizontal load case there is a fair overall agreement between the simulations performed and the test results. Considering initial stiffness and load-displacement response up until a loading of about 3 kN simulation results are in very good agreement with test results. According to simulations the ultimate load capacity is then almost reached and for displacements above say 20 mm the load capacity decreases. The calculated ductility of the wall depends on the connection model employed but according to the analysis presented in Section 8.1 the SS model with stiffness properties valid in the perpendicular direction, i.e. the model denoted “m3_perp” in Figure 19(a), would be expected to capture the true response best for this load case. In comparison to simulation results, however, the models employed underestimates the ultimate load capacity of the wall and also the displacement at which the ultimate load is reached. The slope of the calculated response curve past the ultimate load is in good agreement with the test results.

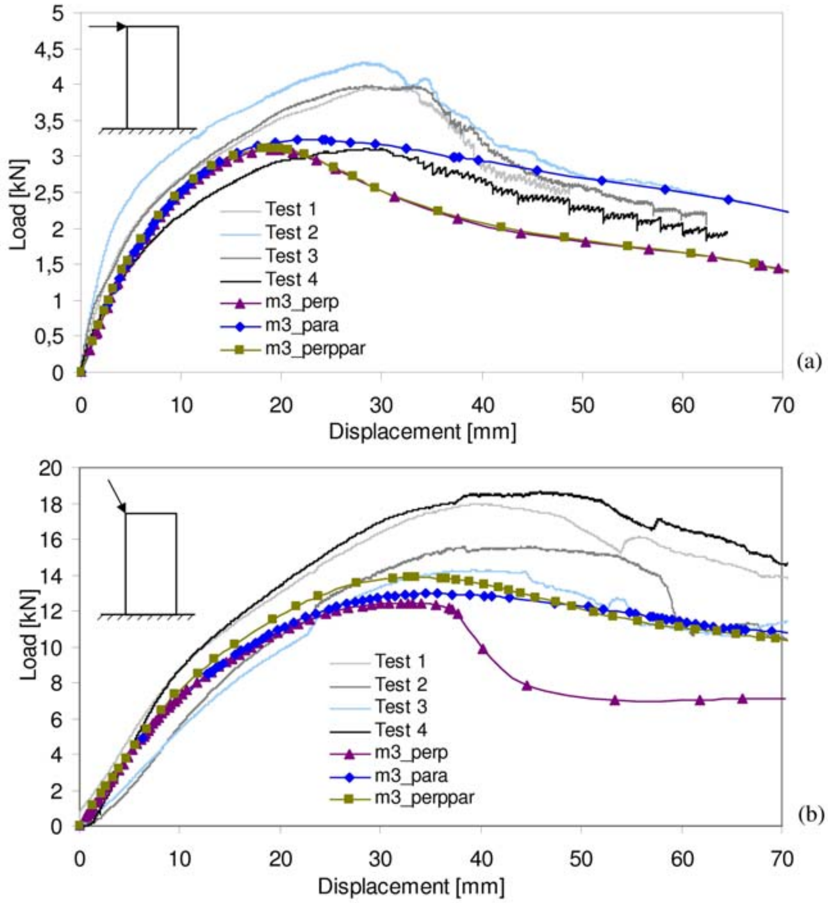


Figure 19. Test results in terms of horizontal load as function of horizontal displacement according to laboratory tests and simulations using the default model (m3_perp), a model with connector stiffness properties valid in parallel direction (m3_para) and a SP model with both perpendicular and parallell stiffness properties for connections between sheathing and frame (m3_perppar). Horizontal loading (a) and diagonal loading (b) are considered.

9.2 Comparisons for a wall assembly of three sheets

A wall consisting of three sheets, seven studs and continuous top and bottom rails, is now examined. Each sheet is connected to wooden members in the same way as the sheathing of the single wall element, see Figure 1, and the modelling of components and connections is identical to the modelling of the single wall element. No contact between the sheets is considered in the modelling. As for the single wall element a horizontal load

case and a diagonal load case are considered. Four walls are tested experimentally for each load case. The same model alternatives as for the single wall element are included in the comparison.

9.2.1 Test setup – Load cases and boundary conditions

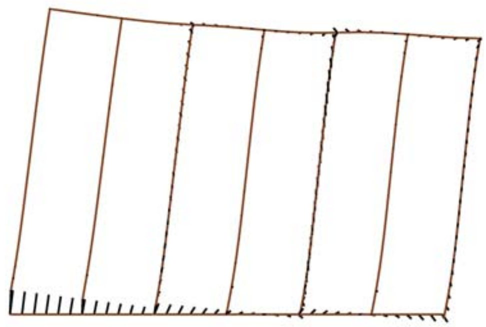
As for the single wall element all tests where performed under displacement control with a constant rate of 8 mm/min corresponding to a failure time of about 5 minutes. In the horizontal load case the load was applied by a hydraulic jack acting in line with the top rail pressing the upper left corner of the wall to the right. In the diagonal load case the load was applied such that an external force acts on the upper left corner of the wall towards the original position of the bottom right corner of the wall. Due to the width of the wall assembly the angle of the load to the horizontal plane, in the diagonal load case, is only one third compared to the angle for the single wall element. Just like for the single wall element there is a difference between the load case in the laboratory test and in the simulations as in the simulations the angle to the horizontal plane does not change when the wall deforms.

For both load cases the bottom rail was prevented from moving by clamping devices placed at equal distances along the entire rail and the top rail was prevented from moving out of the plane of the wall. More information regarding the test setup can be found in Girhammar and Källsner (2004).

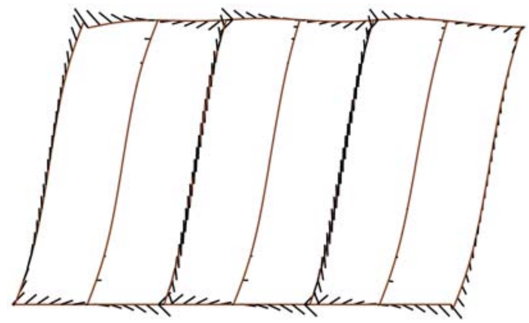
9.2.2 Results from laboratory tests and simulations

Figure 20(a-b) shows exaggerated displacements and deformations of the timber frame of the wall assembly and the displacement directions and relative magnitudes of the connections between sheathing and frame at ultimate horizontal and diagonal loading respectively. The results are calculated using model 3 described in Chapter 8, for a single wall element. For the horizontal load case the dominating behaviour of the wall is that some studs, particularly the leading stud, separate from the bottom rail and that the connection between the sheathing and the bottom rail determine the capacity of the wall. This means that the connections between sheathing and wood are loaded mainly in direction perpendicular to grain. For the diagonal load case the load response of the wall is such that the connections between sheathing and frame are loaded in mixed directions in a pattern that for each sheet reminds of the loading of the connections of the single wall element.

Test results, in terms of the horizontal component of the external force at the upper left corner of the wall as function of the horizontal displacement of the upper right corner, are shown together with the corresponding simulation results in Figure 21(a-b) for the horizontal load case and for the diagonal load case respectively. For both load cases the variation between the four different tests performed is low compared to the variation when testing the single wall element. The overall agreement between experiments and simulations is good for both load cases. For the horizontal load case the initial stiffness is, however, lower according to simulations than according to laboratory tests. Also the ultimate load carrying capacity, the displacement at which the ultimate capacity is reached and the ductility is lower according to simulation using the default model, referred to as “m3_perp” in Figure 21, than according to the experiments. One



Horizontal loading



Diagonal loading

Figure 20. Exaggerated deformations of frames and load directions of sheathing-to-frameing connections at ultimate horizontal and diagonal loading respectively, according to simulations using model 3.

explanation for this is that contact and friction between sheathing units are not captured in the simulation performed but on the other hand a similar difference between simulation results and laboratory tests was found for the wall with only one sheet. For the diagonal load case, see Figure 21(b), the initial stiffness and the ultimate load capacity according to simulations are in very good agreement with the test results. According to simulations using the default model or the SP model, i.e. the models referred to as "m3_perp" and "m3_perppar" in Figure 21(b), some of the load capacity is, however, rapidly lost after the ultimate load is reached. This is not the case for the laboratory tests performed.

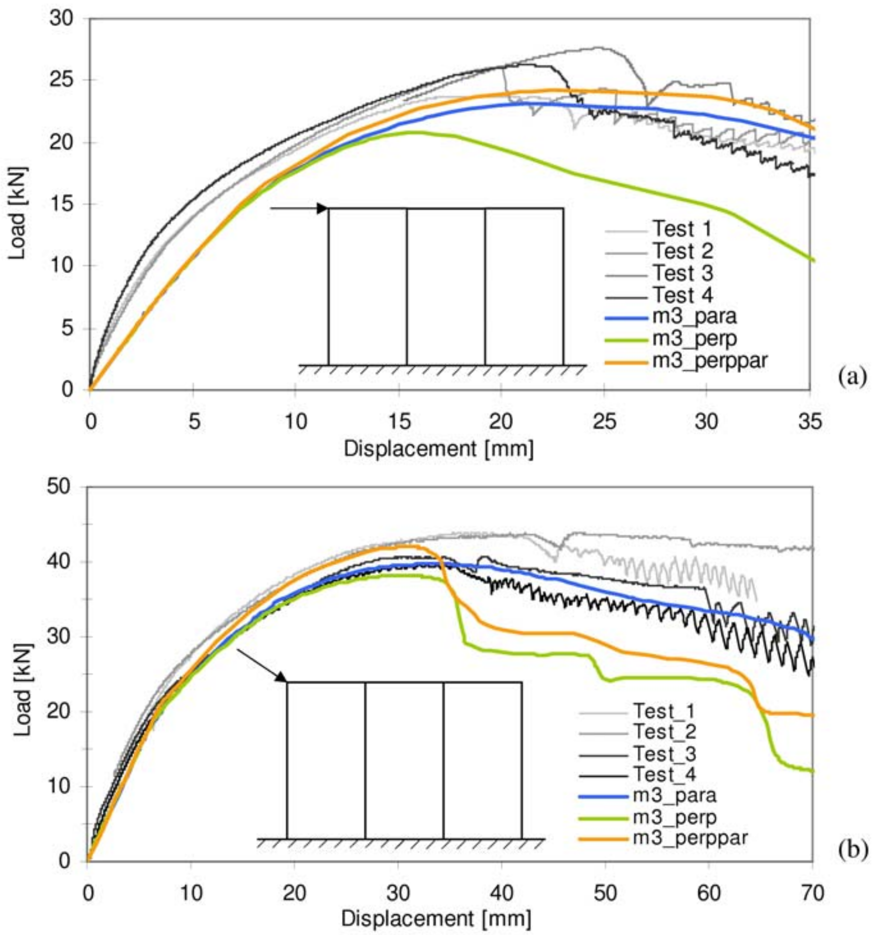


Figure 21. Test results in terms of horizontal load as function of horizontal displacement according to laboratory tests and simulations using the default model (m3_perp), a model with connector stiffness properties valid in parallell direction (m3_para) and a SP model with both perpendicular and parallell stiffness properties for connections between sheathing and frame (m3_perppar). Horizontal loading (a) and diagonal loading (b) are considered.

10 Conclusions

The present investigation has been focused on studying the influence of different modelling assumptions on the structural behaviour of a single wall element subjected to monotonic loading.

One of the key issues is the modelling of the connections between the sheathing and the timber members. Two types of connection models have been studied in the numerical simulations, a single spring model and a spring pair model. The advantage of using the single spring model is that it has the same stiffness in all directions and therefore is easy to implement. The disadvantage of this model is, however, that it does not handle connections where the stiffness is different in different directions. The spring pair model, consisting of two independent springs oriented perpendicular to each other, has the advantage of capturing the properties of connections with direction dependent stiffness. The disadvantage of the spring pair model is that the strength and stiffness of the connections are overestimated when they are displaced in directions not parallel to directions of the individual springs if the linear load-displacement relation is exceeded. Choice of model type is dependent on wall configuration, load case and the direction dependence of the sheathing-to-framing connections used. In general it can be stated that the single spring model works best when the stiffness properties of the sheathing-to-framing connections not are direction dependent while the spring pair model works best when there is a direction dependence.

The nonlinear behaviour of the sheathing-to-framing connections have been investigated using a nonlinear elastic model and a plastic model. The calculations show that no essential unloading of the connections takes place and that the elastic connection model is sufficient to capture the behaviour of the wall studied.

The influence of the ductility of the sheathing-to-framing connections were investigated varying the descending part of the load-displacement curves of the connections. The maximum capacity of the wall assembly studied was not much affected by the different curves. However, the descending part of the load-displacement curves of the wall assembly was very much affected.

A second issue of great importance is the modelling of the stud-to-rail connections. Five different models were investigated of which two allow the timber members to separate while three models do not allow separation. The models not allowing separation in the stud-to-rail connections do not work for horizontal loading as far as the leading stud is not fully anchored. The obvious reason is that these models work as if tie-downs had been inserted in the connections preventing uplift. The main alternative for modelling of the stud-to-rail connections is to use beam elements for the timber members coupled by two independent nonlinear spring elements in the direction of the rail and the stud.

The results from the FE-simulations are compared with results obtained from testing of wall assemblies consisting of one and three sheets. The overall agreement between simulations and laboratory tests is good when reasonable model assumptions are used. For a single wall element with only one sheet the initial stiffness and the ductility according to simulations is in very good agreement with test results. The ultimate load capacity is, however, underestimated for both horizontal and diagonal load cases. For the wall assembly consisting of three sheets the initial stiffness and ultimate load capacity is underestimated when simulating horizontal loading. For diagonal loading, results according to simulations are in very good agreement with test results up until the ultimate load is reached but the ductility past the ultimate load level is somewhat underestimated. A reasonable explanation for underestimated performance of the wall assembly

consisting of three sheets is that contact and friction between the sheets are not considered in the models.

11 Further work

In the modelling of the sheathing-to-framing connections some disadvantages were identified with regard to the mechanical properties of the spring models used. Development of a coupled spring-pair model based on experimental data from testing of connections would be valuable for improvement of shear wall design.

In the analyses carried out in this paper no consideration was taken to contact between adjacent sheets. This influence is of particular importance in connection with openings in wall panels, such as windows, where considerable contact forces may appear. These contact forces should be taken into account in order to minimize the need of tie-downs around openings.

The present study was restricted to analyses of walls in two dimensions. By using a three-dimensional analysis where the entire building is considered a much more fair force distribution is obtained with regard to anchorage. In order to perform such calculations more information about the mechanical properties of floor systems and inter-components for coupling of the different structural parts is needed.

An important area for further work is to develop user-friendly methods for design of entire buildings that can be incorporated in codes and handbooks.

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Paper III

Experimental study of cross-laminated timber wall panels

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Experimental study of cross-laminated timber wall panels

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Abstract

The use of cross-laminated structural timber elements is becoming increasingly popular. The number of layers varies normally from three upwards. The structural performance of five-layer cross-laminated timber elements was investigated. The five layers consisted of boards 19 mm thick, laid successively at right angles to each other and glued together with PU-adhesive, layers 1, 3 and 5 lying in the one direction and layers 2 and 4 in the other. The stiffness and strength of four cross-laminated timber elements (4955 mm long, 1250 mm wide and 96 mm thick) were studied during in-plane bending. Two of the elements were first partitioned into two parts that were reconnected in two different ways prior to testing. The influence of the way in which the cross-laminated timber elements were reconnected was studied, the behaviour observed being compared with the test results for the unpartitioned specimens with respect to both strength and stiffness. The experimental tests performed showed the cross-laminated timber elements to possess a high degree of stiffness and strength. There was also found to be a marked difference in behaviour between the two different ways in which the elements were connected to each other. One of the two connecting methods studied, in frequent use today, showed poor structural performance, whereas the other one performed well.

Key words: cross-laminated timber element, joint, experimental test, timber structure

1. Introduction

Multi-storey timber-based structures have long been built in many different countries. Various challenges are connected with constructions of this type. One such challenge is that of stabilising the structure against horizontal wind loads. The most common stabilising system involves use of a sheathing material such as OSB, plywood or gypsum, connected to the timber frame by nails or screws. Shear forces are thus transmitted by the connectors from the timber frame to the sheathing. The wall elements between separate stories are often connected by means of special brackets or some other form of hold-down devices. The design principles involved and the calculation methods employed have been dealt with extensively in the literature (Källsner and Lam 1995, Kasal et al. 2004 and Ellis and Bougard 2001). In many cases, a stiffer wind bracing system of greater strength may be of interest, particularly in the case of narrow houses that are tall but have a relatively small foundation area. For houses in which the walls are perforated by door and window openings special structural design measures are often needed. In such buildings, use of cross-laminated timber elements, either as parts of the walls or in the walls as a whole, can be of strong interest, a matter which has been studied by Dujic et al. (2004) Blass and Fellmoser (2004) and Moosbrugger et al. (2006), for example. In heavily loaded parts of the walls, the connections between the cross-laminated wall panels are highly important for structural performance. The aim of the present study was to gain greater insight into the behaviour of structures of this type by investigating cross-laminated timber elements experimentally. Both the stiffness and strength of the elements and the ways of connecting the wall elements with each other were studied.

2. Tested specimens

Four wall elements of cross-laminated timber 4955 mm in length, 1250 mm in width and 96 mm thick were tested. The elements consisted of five layers of sawn boards, the successive layers being glued crosswise to each other. The fibres of the two outermost layers, 1 and 5, and of the middle layer, 3, extended in the direction of the length of the element, whereas layers 2 and 4 extended in the direction perpendicular to this, see Figure 1. The 19 mm thick sawn boards of which these timber elements were composed were made of Norway spruce and were on the average about 120 mm in width. In the long direction, the boards were placed butt to butt, there being no finger joints connecting them. The equilibrium moisture content of these elements was found to be about 13 % at the time of testing. The highest and the lowest moisture-content values in a given board differed by about 2 %. It is assumed that in the elements tested the variations in the material properties due to differences in moisture content were negligible.

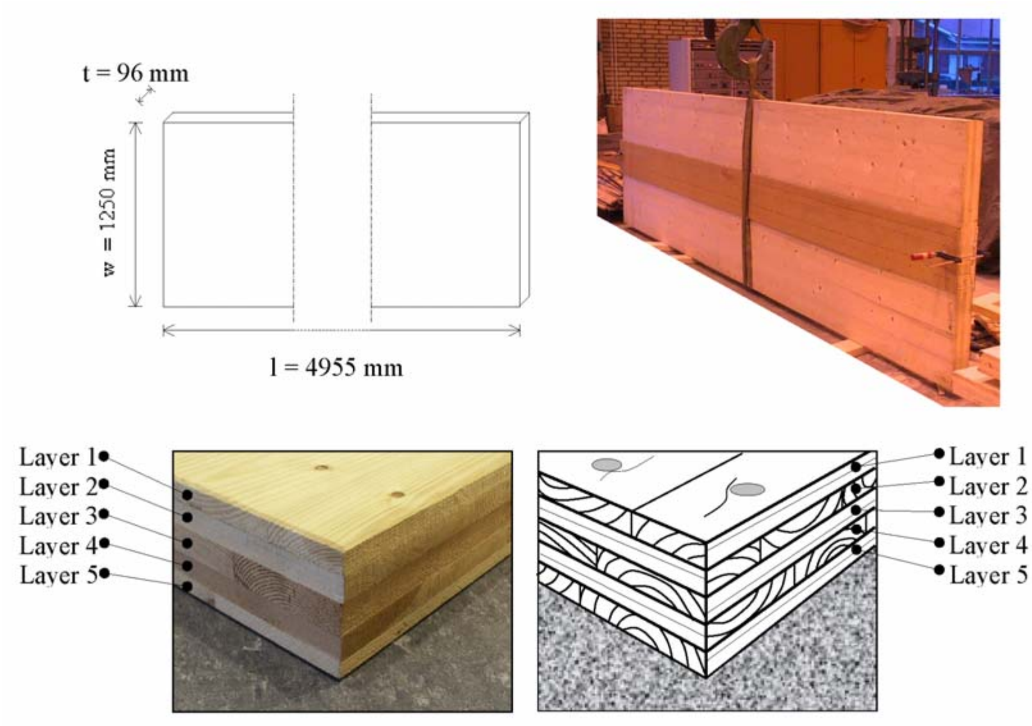


Figure 1. The five-layer cross-laminated timber-wall elements tested.

3. Testing of joints

After stiffness testing of the elements 1 and 2 at a low load level, these two cross-laminated timber elements were sawed into two parts. These parts were then reconnected by use of two different jointing methods. The new wall elements having longitudinal joints were renamed as element 5 (made from element 1) and element 6 (made from element 2).

For element 5, a purely mechanical joint was employed. After this element had been partitioned, a 60 mm deep and 25 mm wide slot was cut into each of the two halves. A sawn board 120 mm wide, 25 mm thick and graded as C24 was fitted into the slot. Hexagonal-head wood screws, 96 mm long and 8 mm in diameter, were screwed both from the left and the right into the two sides of the element. The screws were mounted in predrilled holes as shown in Figure 2(a).

For element 6 both an adhesive and screws were used for securing the joint. A sheet of fibreboard (quality C40) 300 mm wide and 8 mm thick was applied to both sides of the partitioned specimen, see Figure 2(b). The adhesive used was a single-component polyurethane with a curing time of approximately 12 hours. The sheets of fibreboard

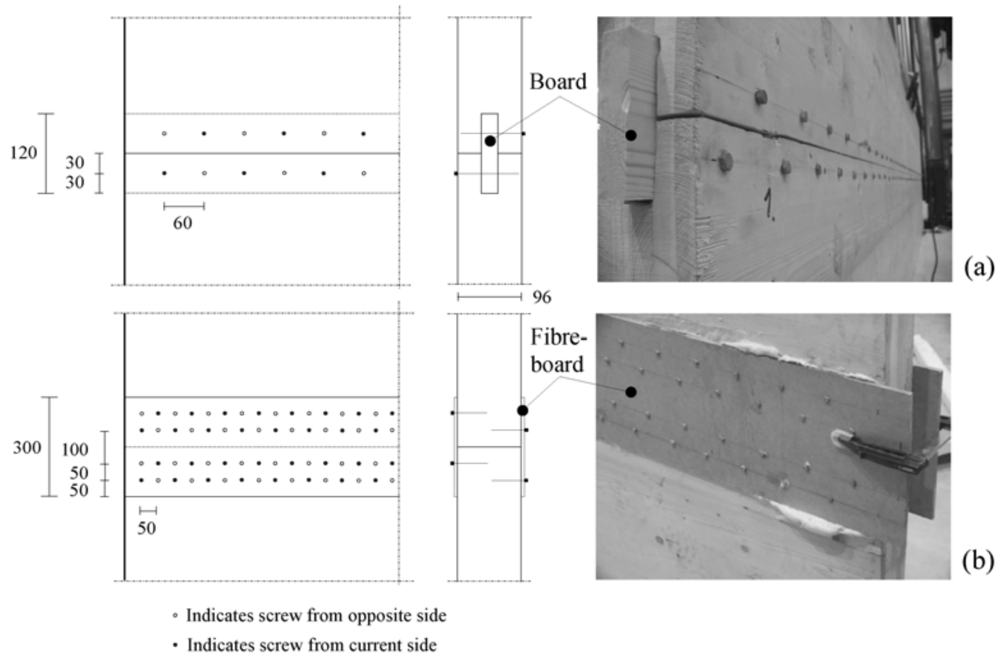


Figure 2. Joints of the longitudinal wall-elements: (a) element 5 with a sawn board screwed to the parts of the element and (b) element 6 with fiberboard sheets glued and screwed to the parts of the element.

were also fastened mechanically by means of 50 mm long hexagonal-head wood screws 6 mm in diameter located in two parallel rows, as shown in Figure 2(b).

4. Tests of stiffness and strength

An overview of the tests performed for the wall elements, numbered 1 to 6, are given in Table 1. A schematic diagram of the testing setup is presented in Figure 3. The loading in the vertical plane of a simply supported cross-laminated timber element, acting as a horizontal beam, is similar to the loading of a cantilever wall element of half the length fixed to the ground.

The elements 1 to 4 were first loaded in a non-destructive way at a low loading level in order to determine their initial stiffness. Each of the elements was loaded by the testing machine, see Figure 4, under displacement control to a load of 200 kN and was then unloaded. The crosshead movement of the hydraulic jack proceeded at a rate of 2 mm/min.

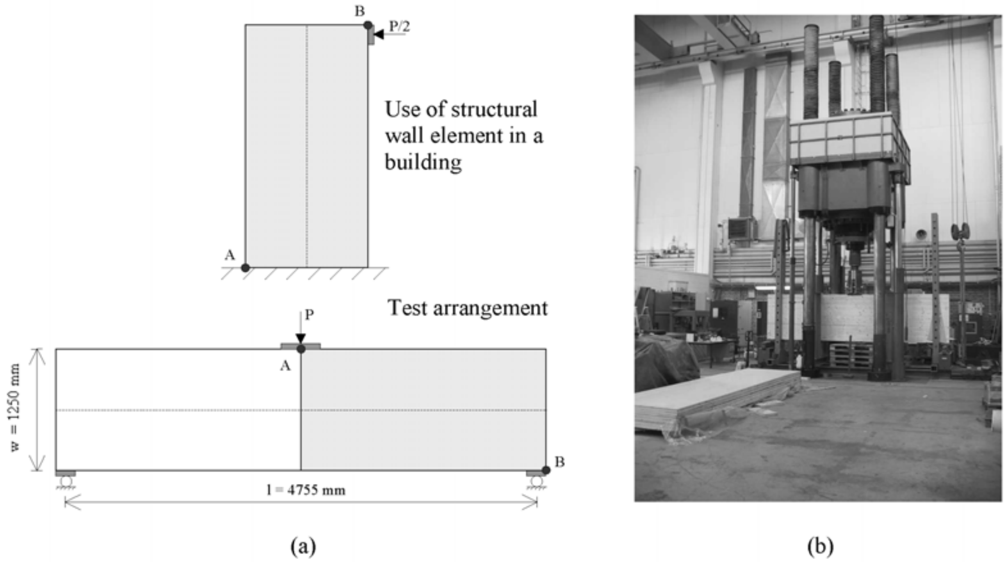


Figure 3. (a) Schematic plan of the testing setup, (b) photograph of the testing machine. (The dotted line indicate a longitudinal joint such as for elements 5 and 6.)

The hydraulic testing machine had a capacity of 20 MN and the accuracy in force measurement was about ± 2.3 kN for the maximum load used in this test.

Table 1. Course of events for the elements tested.

Element	**Course of events**
1	Stiffness only tested in the load interval 0- 200 kN.
2	Stiffness only tested in the load interval 0- 200 kN.
3	Stiffness test. Loading to failure.
4	Stiffness test. Loading to failure.
5	Made by cutting element 1 into two halves and then reconnecting the two halves by use of a sawn board screwed to the parts of the element. Loading to failure.
6	Made by cutting element 2 into two halves and then reconnecting the two halves by screwing and gluing of fibreboard sheets to the parts of the element. Loading to failure.

In the stiffness tests the displacements were measured by use of gauges, as shown in Figure 4. Gauges 1 and 3 were placed at half-height above the centre of the support on the left and right side, respectively, their measuring the vertical deflection above the supports, relative to the supports. Displacement gauges 2 and 4, in turn, measured the vertical deflection at half-height and mid-span on each side of the element. Gauge 5 measured the mid-deflection at half-length on the underside of the element. The displacement gauges 6 and 7 were added in testing elements 5 and 6. These two gauges were used to measure the relative slip at the ends of the longitudinal joint connecting the two parts of the elements studied.

The supports provided at the two ends held the element for vertical translation, the element being free to move horizontally at both supports. These two solid steel supports were 200 mm long and 50 mm thick, whereas the steel plate at the loading point was 400 mm long. Horizontal translation of the elements was prevented by friction at the loading point. Translation out of the plane of the elements tested was prevented by supporting surfaces of low friction. These surfaces were placed in couples on both sides of the element (at x= 350 mm, x= 1450 mm, x= 3500 mm and x=4600 mm, x being the distance from the left support). The testing setup is shown schematically in Figure 4, together with photos of a displacement gauge and the support arrangement.

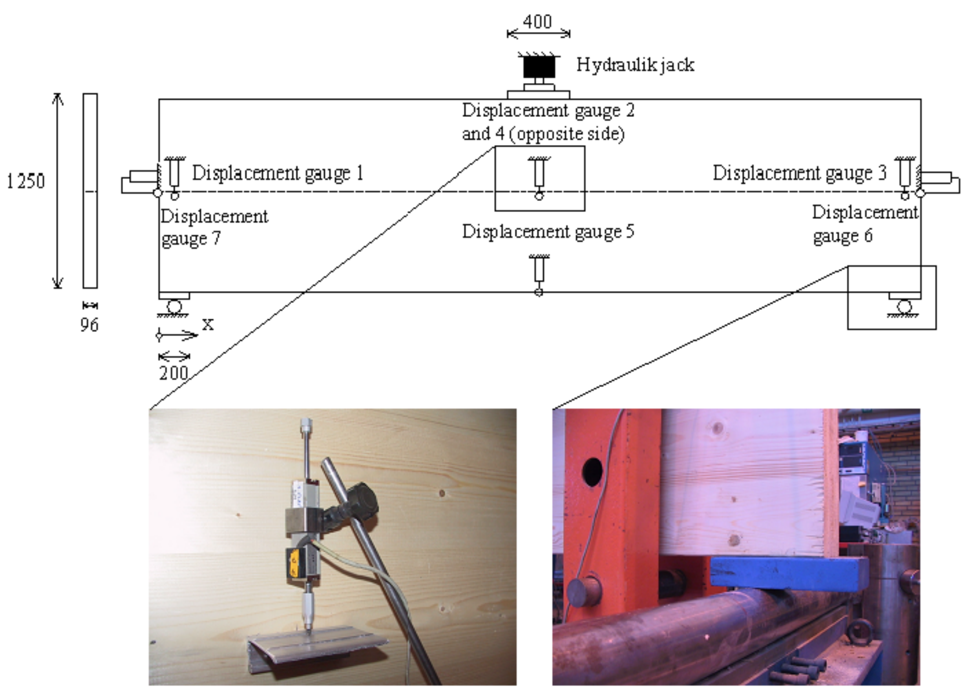


Figure 4. Setup for measuring the displacement.

5. Results

5.1 Stiffness

Considering the specimens tested as constituting deep beams, the deformations observed can be seen as representing contributions of three major types, those of from bending, shear and local material compression, at the loading and supporting areas. Due to the orthogonal orientation of the different layers in the timber elements, the material compression could be kept relatively small. Both at the support and at the loading point, the timber was loaded parallel to the grain in two of the five layers.

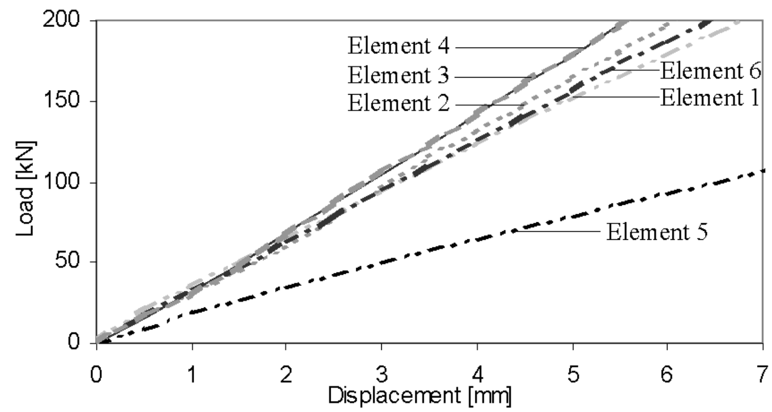


Figure 5. Load-displacement relations for elements 1 to 6, used for determining the total stiffness in regard to the bending and shear modes.

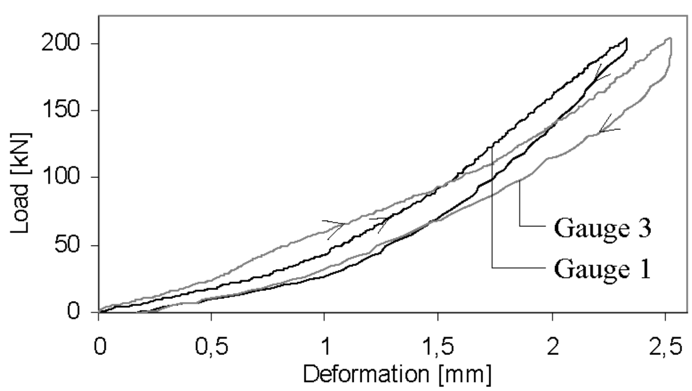


Figure 6. Measured deformation of element 2 above the supports for a low load level.

The mid-deflections measured in the six specimens tested were used for determing of the overall bending and shear stiffness. In Figure 5 the mid-displacement curves obtained for elements 1 to 6 are shown. The values presented were determined by subtracting the respective averages of the values obtained at gauges 1 and 3 from the corresponding averages obtained at gauges 2 and 4, see Figure 4. The behaviour for each of the six elements is nearly linear. For a load of 100 kN the mid-displacement was about 3 mm for each of the elements except for element 5, in which the displacement was much larger.

The displacements measured by gauges 1 and 3, see Figure 4, provide a measure of the local deformations above the supports. Some of the results for the low load values, obtained for element 2, are shown in Figure 6, illustrating a behaviour that was typical. Due to a non-perfect fit between the specimen and the supporting steel plates the response is first nonlinear. From a load of about 30 kN and upwards, the behaviour then becomes almost linear. After unloading, the displacement that remained due to local deformations was less then 0.2 mm.

5.2 Loading to failure

After loading the specimens up to 200kN and reloading, they were loaded to failure. The displacement at the loading point was gradually increased, the magnitude of the load being recorded. The load-displacement curves for elements 3 to 6 are shown in Figure 7. Up to failure, which occurred suddenly each of the elements behaved almost linearly. Element 5, with its simple mechanical fasteners in the longitudinal joint, was much weaker than elements 3, 4 and 6. This indicates a considerable difference in behaviour between the two alternative ways of connecting used at the joints. In contrast, the difference in behaviour between element 6, which had glued and screwed fiberboard sheets in the joint, and elements 3 and 4 without any joints, is small. It is notable how very small the difference is between the load- deformation curves for elements 3 and 4, except for the difference in failure load.

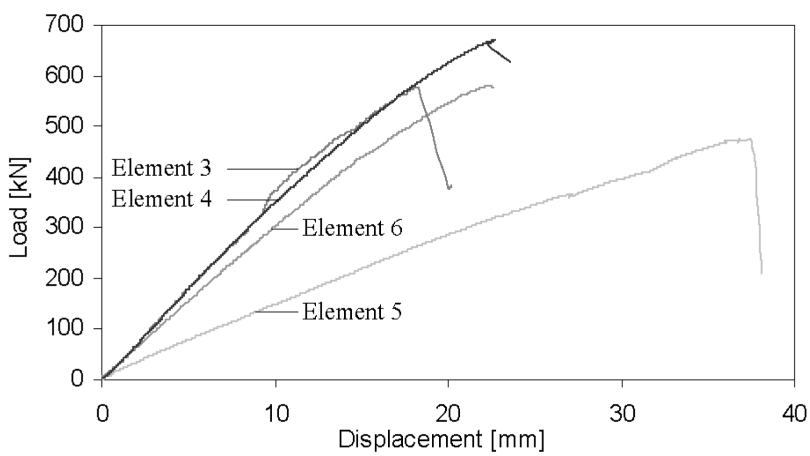
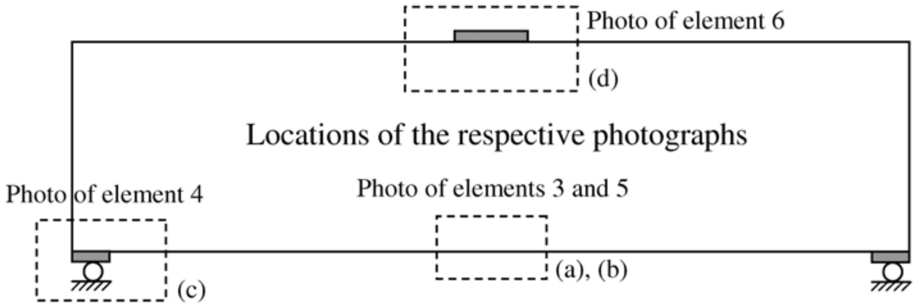


Figure 7. Load displacement curves for elements 3 to 6 while being loaded to failure.

It was characteristic for all four specimens that failure occurred suddenly. Once the failure load had been reached, the entire load-bearing capacity disappeared at once, there being no ductile behaviour. Two main types of failure occurred: bending failure and local failure at the support and loading point, respectively. Elements 3 and 5 both failed in bending due to the high tensile forces the boards were subjected to, whereas elements 4 and 6 failed at the loading point and at the support, respectively. The course of event for the latter two elements was similar. At the failure load a sudden loud noise was heard when rupture of the wooden material at the support or at loading point occurred. Each of the failure modes can be seen in Figure 8.



(a) Element 3

(b) Element 5

(c) Element 4

(d) Element 6

Figure 8. Failure modes and location of each for the four elements, bending failure being involved for elements 3 and 5, and failure at the support and failure close to the load application point for element 4 and element 6, respectively.

Table 2. Stiffness and failure load for the specimens tested.

Element number	Stiffness [MN/m]	Failure load [kN]	Failure modes
1	29.7		-
2	33.0		-
3	35.7	577	Tensile failure in bending
4	35.8	672	Local compression failure at the support
5	14.6	475	Joint failure. Tensile failure in bending
6	30.8	580	Local compression failure at the loading point

Table 2 provides an overview of the experimental results obtained. The comparative stiffness values given are based on the load intervall of 0 - 200 kN. Failure load values are presented for elements 3 to 6. The highest failure load, reached for element 4, was about 40 % higher than for element 5, which likewise had an inferior joint design. Note that element 6, jointed by use of glued fibreboard, had a somewhat higher failure load than element 3, without any joint. This indicates clearly that the joint design used for element 6 worked well from a structural standpoint.

The relative slip at the ends of the joints in elements 5 and 6 was measured. The displacement gauges 6 and 7 were placed horizontally on each side of the two partitioned elements 5 and 6, respectively, see Figure 4. The relative slip horizontally measured (as absolute values) between the two halves, can be seen in Figure 9. The slip for element 5 was much greater than in the case of the glued and screwed connections used for element 6. This shows further the very low degree of stiffness of element 5, which resulted in a much larger deformation of the specimen than for the other specimens tested, as can be seen in Figure 7.

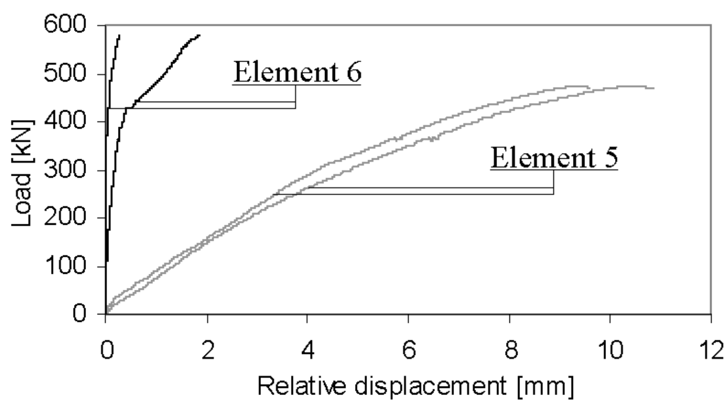


Figure 9. The slips measured in the joints of elements 5 and 6, the measurements being made at both ends of the respective longitudinal joint.

5.3 Remarks on experimental results

The smaller failure load for element 5 was caused primarily by a cracking failure in the board's longitudinal joint, see Figure 10. As can be seen, the board failed due to a splitting crack. This splitting perpendicular to the grain is due to the transverse shear forces transmitted to the board by the screw fasteners. The failure mode obtained clearly indicates that the tensile strength perpendicular to the grain of the connecting board was too low. The sawn board should be replaced by another type of board connector, one made of plywood, for example. This would be preferable since it would increase the tensile strength in the critical direction of the material.

Figure 10. The sawn board of element 5 after loading to failure.The board was used as a connecting medium in the longitudinal joint. A crack appeared along one of the screw rows.

Table 3. Approximative stress values at failure load for the specimens tested.

Element number	Maximum load [kN]	Average support stress [MPa]	Effective bending stress [MPa]
3	577	37.6	48.0
4	672	43.8	55.9
5	475	30.9	39.5
6	580	37.8	48.3

In Table 3 various of the calculated stress values are presented, corresponding to the failure loads of the specimens. The average support stress at the supports and at the loading points was calculated by assuming the stresses to be concentrated to the two layers of the cross-laminated elements in which the grain direction is oriented perpendicular to the supporting plane. Also, the so-called effective tensile stress due to bending was calculated in an approximate way by considering only the three layers in which the grain direction was parallel to the length of the specimens.

For an adequate serviceability state analysis of a reasonably accurate estimate of the stiffness is needed. To achieve this, the experimental results for the combined bending and shear stiffness were compared with the results of simple hand calculations based on ordinary material data and simple beam theory, including shear. The influence of each of the five layers of the cross-laminated specimens was taken into account.

The longitudinal modulus of elasticity was set to E_l= 12000 MPa and the transversal modulus of elasticity to E_t= 400 MPa, corresponding to an effective longitudinal modulus of elasticity of E_{eff}= 7360 MPa for the five layers. The shear modulus G_{eff} was assumed to be 750 MPa. On the basis of these assumptions, of the formula used, and of a loading of 200 kN, the mid-deflection becomes 7.1 mm, 55 % of which is due to bending, the rest being due to shear. In the experimental test, the average displacement was 6.1 mm, indicating the value of the longitudinal E- modulus of the specimens tested to be somewhat higher than the assumed value of 12 000 MPa.

6. Concluding remarks

Testing a number of cross-laminated timber element specimens experimentally with respect to their stiffness and strength provided useful results. Two of the specimens were partitioned into two parts and were then reconnected, two alternative joining methods being employed. It was shown that for one of the two jointing alternatives the connection between the joined parts was just as strong and stiff as for the corresponding elements without a joint. The weaker alternative of using only mechanical connectors and a sawn board as a connecting medium is not to be recommend since the resulting strength and the stiffness were much too low.

The high level of stiffness obtained for the cross-laminated wall elements and the possibility this provided of obtaining sufficiently strong and stiff connections indicate the use of cross-laminated timber elements to have a strong stabilising potential in building construction involving timber.

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Paper IV

Contact-free strain measurement of bi-axially loaded sheathing-to-framing connection

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Contact-free strain measurement of bi-axially loaded sheathing-to-framing connection

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Summary

Shear walls in timber houses often consists of timber frames with a sheathing nailed or screwed to it. The overall behavior of such walls is strongly dependent on the stiffness, strength and ductility of the fastener itself and on the properties of the wood and the board in the close surroundings of the fasteners. Another critical connection in walls is that between a stud and a rail where wood may be compressed perpendicular to grain. Methods for laboratory investigations and results from experimental tests are discussed for these two connections. An optical measurement system for high precision continuous collection of the strain field on the surface of a tested specimen is described and utilized in the experiments. The knowledge obtained will be used in the planning of more advanced test setups in which test specimens are loaded bi-axially. Results from such experiments will enable the development of more advanced models for connections than are presently available.

1. Introduction

There is a long tradition of building timber structures stabilized by sheets of wood or other materials, such as gypsum, nailed or screwed to a timber frame. In many cases this is the most economical choice for stabilization of timber structures. Simplified methods for hand calculations of the shear resistance in such walls are available, e.g. Källsner and Lam [1], but these methods needs to be refined in order to more accurately describe the behavior when the structure is being loaded. This specially concerns high timber houses or timber houses with large and many openings. Development of refined methods and models are strongly dependent on advanced experimental tests on material and assembled structures.

In particular the stiffness, strength and ductility of the connections between the sheathing and the framing has great influence on the overall behavior of a shear wall. This is well known and much research on the properties of fasteners has been carried out, see for instance Filiatrault and Foschi [2] and Judd and Fonseca [3]. What is novel in this paper is that the strain field in the close surrounding of a loaded fastener is evaluated by use of a rather newly developed optical, contact-free data collecting system that is described below. Results from tests on sheathing to timber fasteners and from tests where wood is compressed perpendicular to grain, as may be the case when a stud is compressed to a rail, are presented. The aims of the work are to show the abilities of the optical data collecting system used and to prepare for more advanced experiments on connections in

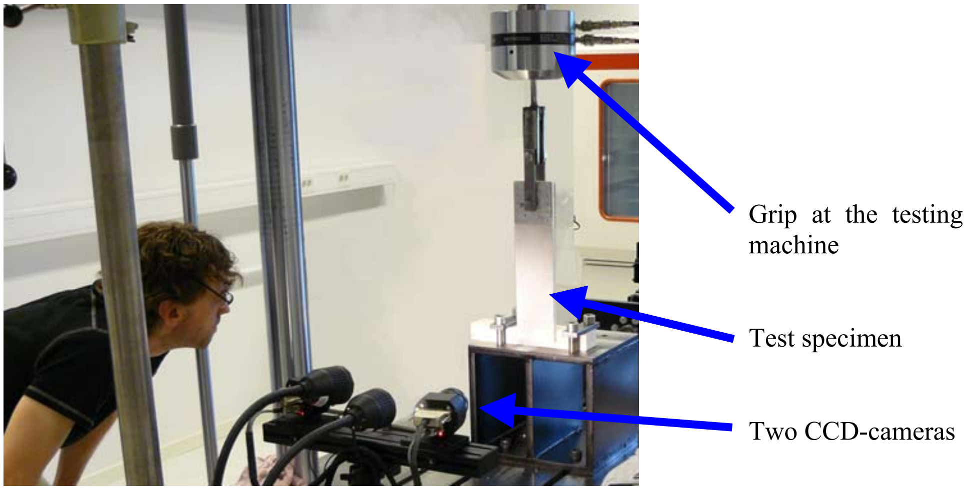

Figure 1. MTS testing machine arranged for testing a screwed sheathing-to-wood connection where the wooden member is loaded perpendicular to grain. The contact- free ARAMIS™ measurement system with CCD-cameras is mounted in front of the test specimen.

which test specimens are loaded bi-axially. Results from such experiments will enable the development of more advanced models for connections than are presently available.

2. Testing equipment

An overview of the employed equipment is shown in Figure 1. The loading equipment is a servo hydraulic testing machine (fabricate MTS) with a load capacity of ±100 kN. The testing machine may be used to apply a prescribed force or displacement in two orthogonal directions simultaneously but in the present investigation only uni-axial displacement controlled tests are performed. Different types of transducers for displacement measurements, e.g. extensometers or LVDT:s, are connected to the testing machine when performing standard tests. The optical measurement system ARAMIS™ was used for detailed deformation measurements. The system is based on evaluation of a random or regular black and white pattern, which is normally applied by speckling the surface by spraying it with a thin spray film that deforms along with the surface. In the experiments performed here a matt, light- colored paint was sprayed on the area in order to obtain good contrast. Thereafter a random pattern of small black dots was applied by spraying black paint at the surface from a distance. Two CCD cameras (1280 by 1024 resolution) in front of the specimen takes photos in an angle to each other and thereby enables stereoscopic pictures of the patterned surface, see Figure 1.

Post-processing of the pictures starts with identification of a reference state. In the present case this represents the unloaded specimen. Each picture is then subdivided into partially overlapping pictures, so called facets. The size of the area to be covered by the

pictures and the amount of overlap are decided by the user and determine the spatial resolution and accuracy that will be achieved. The gray-scale of the pictures is used to perform cross-correlation calculations for the facets, such that each facet position can be tracked with sub-pixel accuracy from one pair of pictures to the next. In this tracking, it is possible to correlate a non-square facet in the deformed configuration back to its square, undeformed original state. Further information about the optical system can be found in Sjödin et al. [4].

3. Evaluated connections

Two different types of connections are evaluated. The first one is a screwed or glued connection between sheathing and framing, loaded in the plane of the sheathing, and the second one is a connection in which a wooden member is compressed perpendicular to the grain as may be the case when a rail is compressed by a stud. The optical measurement system is used to analyze the local deformations in both these types of connections.

3.1 Sheathing-to-framing connections

Four different tests are included in this study. The test specimens consist of three different parts; a gypsum board, (Gyproc GF 15, 15×100×400 mm^3) a wooden member of Norway spruce with a cross-section of 45×70 mm^2 and either a single screw (QSTR 41 Quick, 3.9×41 mm^2) or a layer of glue (PVAc) in between. The gypsum board covers half the thickness of the wooden member, i.e. 22.5 mm. Screws are located at a distance of 11.3 mm from the edge of the board. In glued specimens glue is applied over the entire overlap area between board and wood. Both screwed and glued specimens are tested in two different setups. The first, see Figure 2, is arranged such that the wooden member is loaded in direction perpendicular to grain. The second setup is arranged such that the wooden member is loaded in direction parallel to grain. For each setup two screwed specimens and one glued specimen are manufactured and tested which in total makes six specimens in the test series. The CCD cameras are calibrated to a measurement area that includes the entire overlap area of the gypsum board and the wood with some marginal.

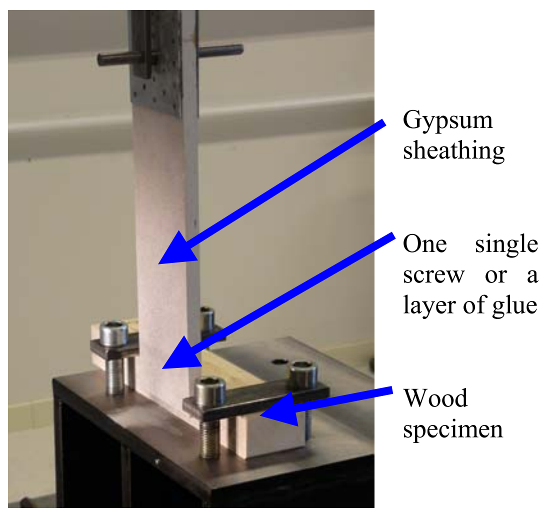

Figure 2. Test setup for experiments performed pulling the gypsum sheathing perpendicular to the length direction of the timber member.

3.1.1 Results

For the six specimens tested load- displacement curves acquired with the testing machine, and thus including the deformation of the entire specimens, are shown in Figure 3. Two curves show the tests with screws loaded in direction perpendicular to the length direction of the timber member, i.e. perpendicular to grain, two show the test with screws loaded in direction parallel to grain, and two show the glued connection (one in each direction). As expected stiffness and strength are high and ductility is low in the glued connection compared to the properties of the screwed connection. The results also show that the screwed connection is almost twice as strong when loaded parallel to grain as when loaded perpendicular to grain.

Results similar to those presented in Figure 3 are often presented in literature but the more precise behavior, i.e. the two-dimensional displacement- or strain field, is usually not evaluated from experimental results. With the optical measurement system used here the load- displacement curves can be complemented with strain plots, at different load levels.

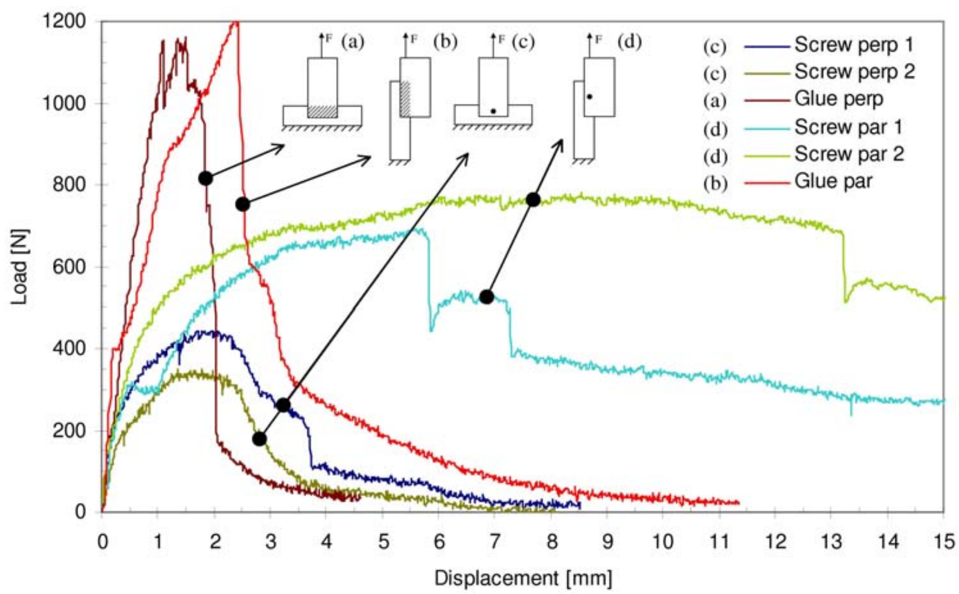

Figure 3. Load- displacement curves of the piston for tests on screwed and glued connections respectively, and for loading in direction parallel and perpendicular to grain of the wooden member respectively.

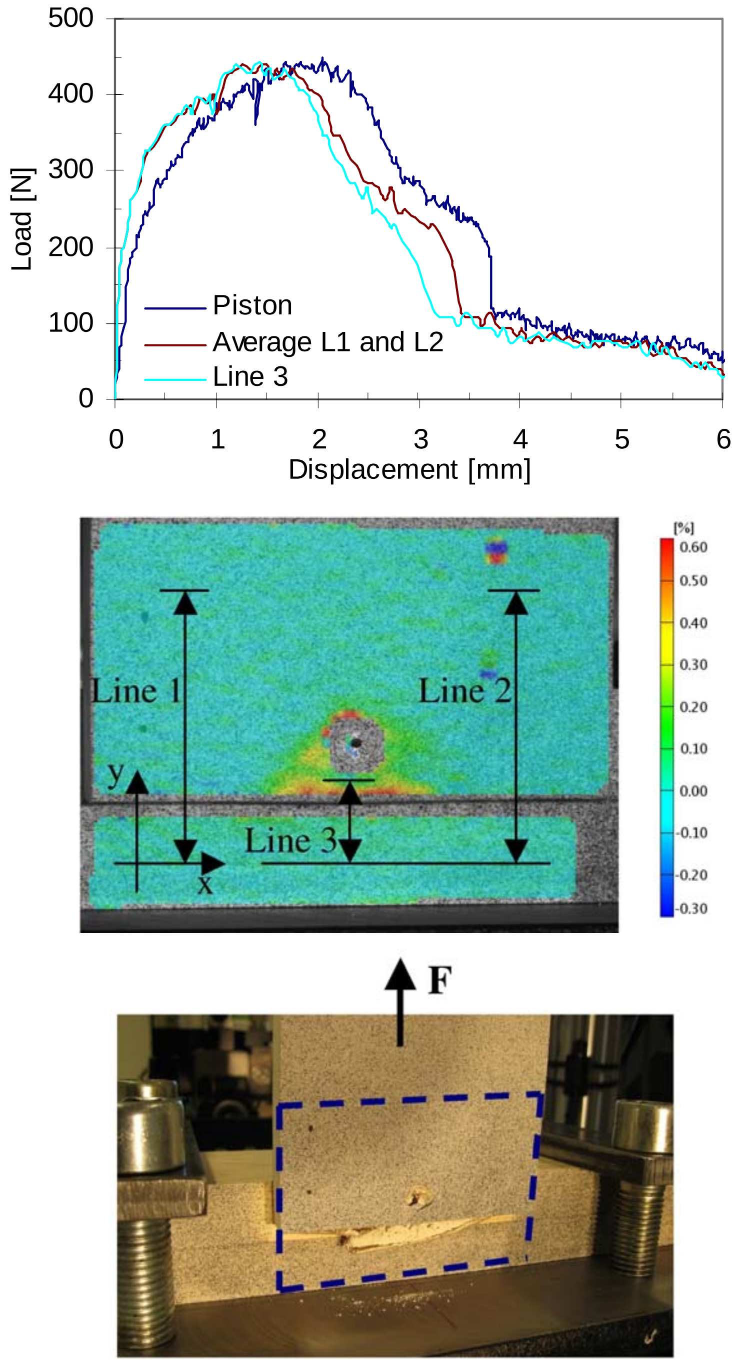

Figure 4. Load- displacement relations (top), strain field in y-direction from the optical system (mid) and the failure mode of the test specimen (bottom).

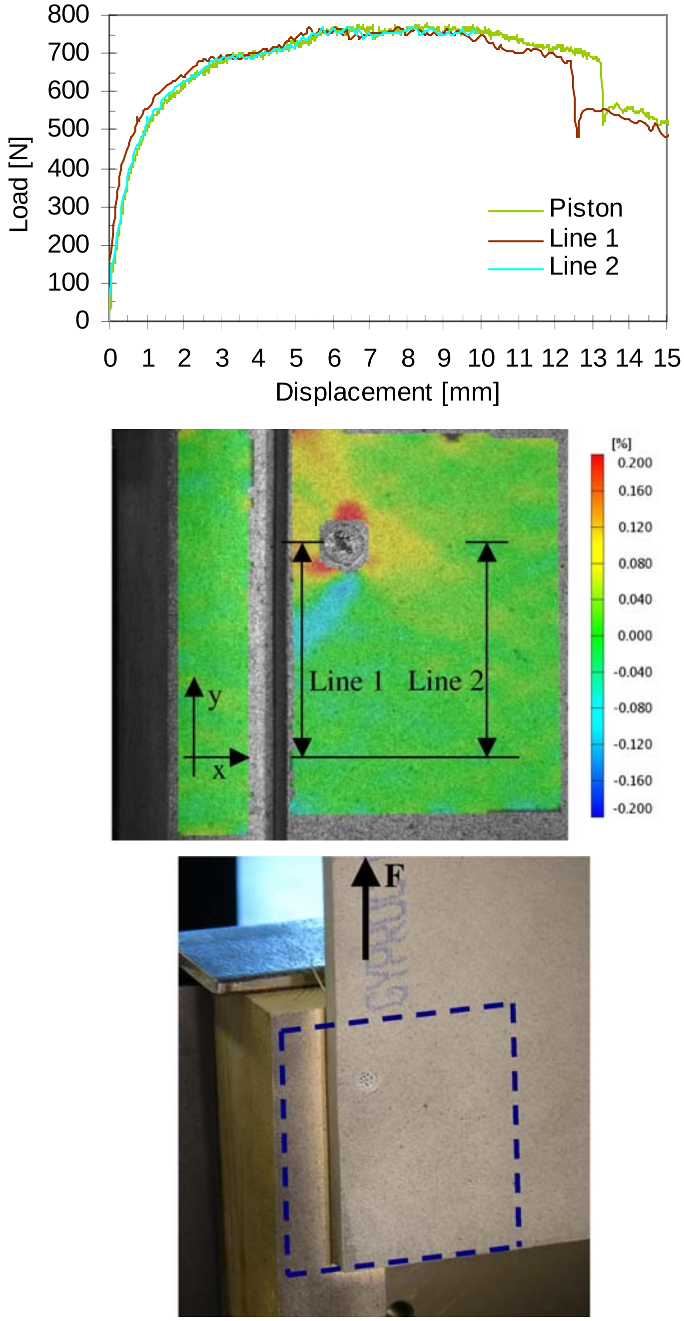

Figure 5. Load- displacement relation (top), strain field in y-direction from the optical system (mid) and test specimen loaded in direction parallel to the wooden member (bottom).

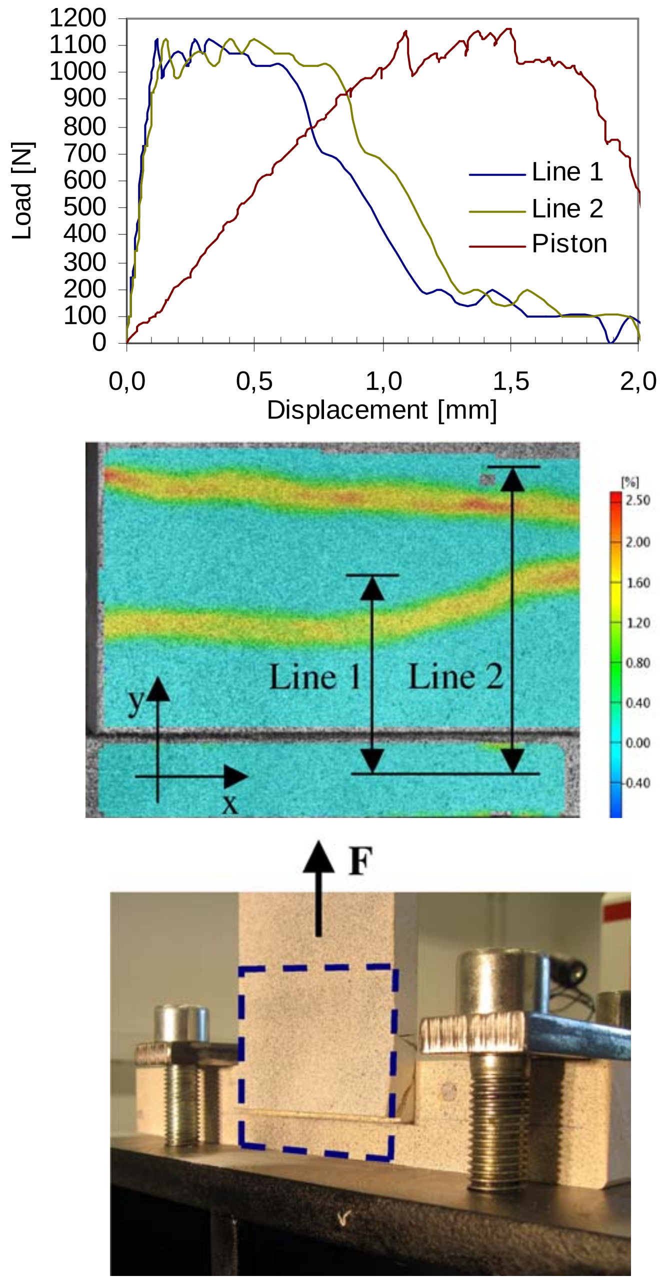

Figure 6. Specimen with glued on sheathing (PVAc) loaded in direction perpendicular to the direction of the wooden member. Curves from piston as well as from the optical system (top), plot of strains in y-direction at maximum load (mid) and the failure mode of the test specimen (bottom).

An example of such a strain plot, for a specimen that is loaded in direction perpendicular to the direction of the wooden member, is shown in Figure 4. The strain distribution in the y-direction on the surface of the gypsum board, in an area close to the screw, is shown at maximum load (mid). Figure 4 (bottom) shows the failure mode for the tested specimen. A triangular piece of gypsum between the screw and the edge of the board comes loose. Figure 4 (top) finally shows the load- displacement curve of the piston compared with the load- displacement curves representing the lines in the close surrounding of the connection. The lines are oriented vertically and span from a reference plane fixed to the timber member to selected points on the sheathing. The discrepancy between the local load- displacement relations and that of the piston indicates the significance of the choice of evaluation points. Results for a connection loaded in direction parallel to the direction of the timber member are shown in Figure 5. As for the load case presented in Figure 4 it can be seen in the strain overlay image that the influenced area on the sheathing is concentrated to a rather narrow surrounding of the screw. The failure mode obtained where shear failure in the screw after some preceding damage in the gypsum board. Figure 5 (top) shows force as function of elongation of Line 1, elongation of Line 2 and displacement of the piston respectively.

For the specimen with glued on sheathing (PVAc), loaded in direction perpendicular to the direction of the wooden member, Figure 6 shows the strain distribution at maximum load (mid) and the failure mode (bottom). In this case the strains are localized to two more or less horizontal cracks across the gypsum sheathing. The lower of these are at the close surrounding of the edge of the timber member and the upper one is where the 15 mm thick sheathing starts to taper off, i.e. decrease in thickness, to the edge. The strains concentrated to two cracks would not be possible to localize by simply measuring the displacement between a few points. According to the load- displacement curves shown in Figure 6 the elongations of Line 1 and Line 2 are almost the same up to the maximum load. The divergence between the two curves, appearing after 0.2 mm displacement, means that the upper crack then starts to contribute considerably to the elongating of Line 2.

3.2 Compression between the stud and the rail

The stiffness and load capacity of a connection in which a wooden member with dimensions 45×95 mm^2 is compressed perpendicular to grain by another, much stiffer member is evaluated. The stiffer member in the tests performed is made of steel with dimensions 45×95 mm^2. The wooden members are of Norway spruce with a density of 375 kg/m^3 at the moisture content of 10.3 %.

Having in mind the geometry of a wall frame where a stud is compressed to a rail it is relevant to consider two different geometries, namely when the compressed wooden member ends on one side of the stiff vertical member and when the compressed wooden member is continuous on both sides of the stiff vertical member, see the infolded drawings in Figure 7. Thus two different tests are performed. The test series is limited to three test specimens, two where the wooden member ends on one side of the stiff vertical

member compressed to it and one where the wooden members is continuous on both sides of the vertical member.

The three load- displacement curves from the tests, representing the piston of the machine, are shown in Figure 7. After an initial compression showing a low initial stiffness full contact is achieved. The elastic stiffness is then almost equal for two of the specimens, one of each type but it is somewhat lower for one of the specimens of the type where the wood ends on one side of the stiff vertical member. At higher load levels it is obvious, however, that the specimen with continuous wood on both sides of the stiff vertical members has considerably higher load bearing capacity and stiffness compared to the specimens where the wood ends at one side of the stiff vertical member. The very limited test series performed is not sufficient for a quantitative evaluation of stiffness and strength. What is interesting, however, is that strain plots from the optical measurement system capture details in the deformation that gives precise information about how wooden members are affected by the loading. Figure 8 shows strain plots for the tested specimens at the load 19.6 kN corresponding to an average stress of 4.6 MPa over the contact area. The strain plots suggest that in the test specimen with a continuous wooden member a wider part of the wood has significant strains in the y-direction than what is the case for the other specimens. The strain plots also show a striking difference in stiffness and strength between early wood and later wood in the annular rings.

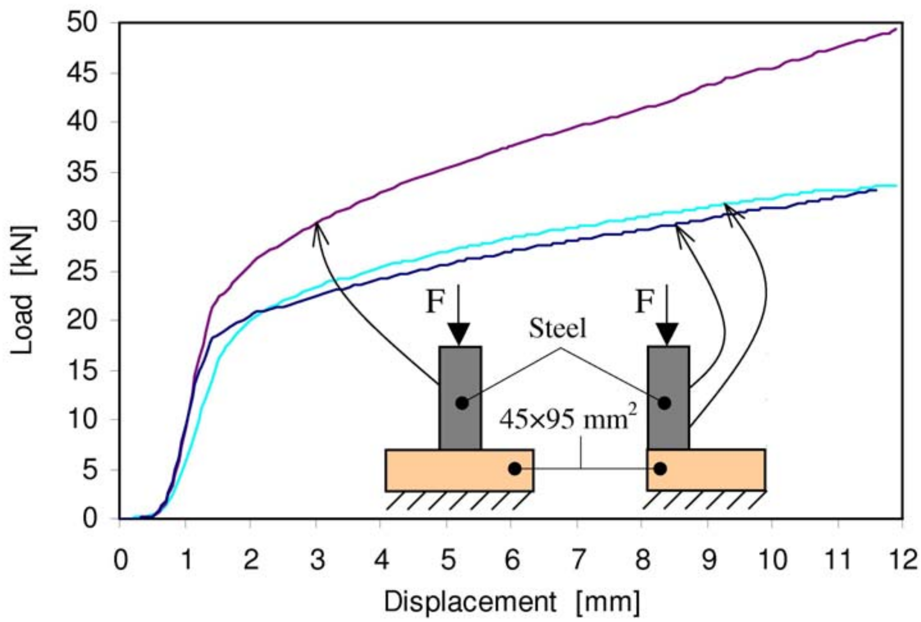

Figure 7. Load- displacement relations for specimens in which the compressed horizontal wooden member is continuous on both sides of a vertical steel member or ends at one side of the it respectively.

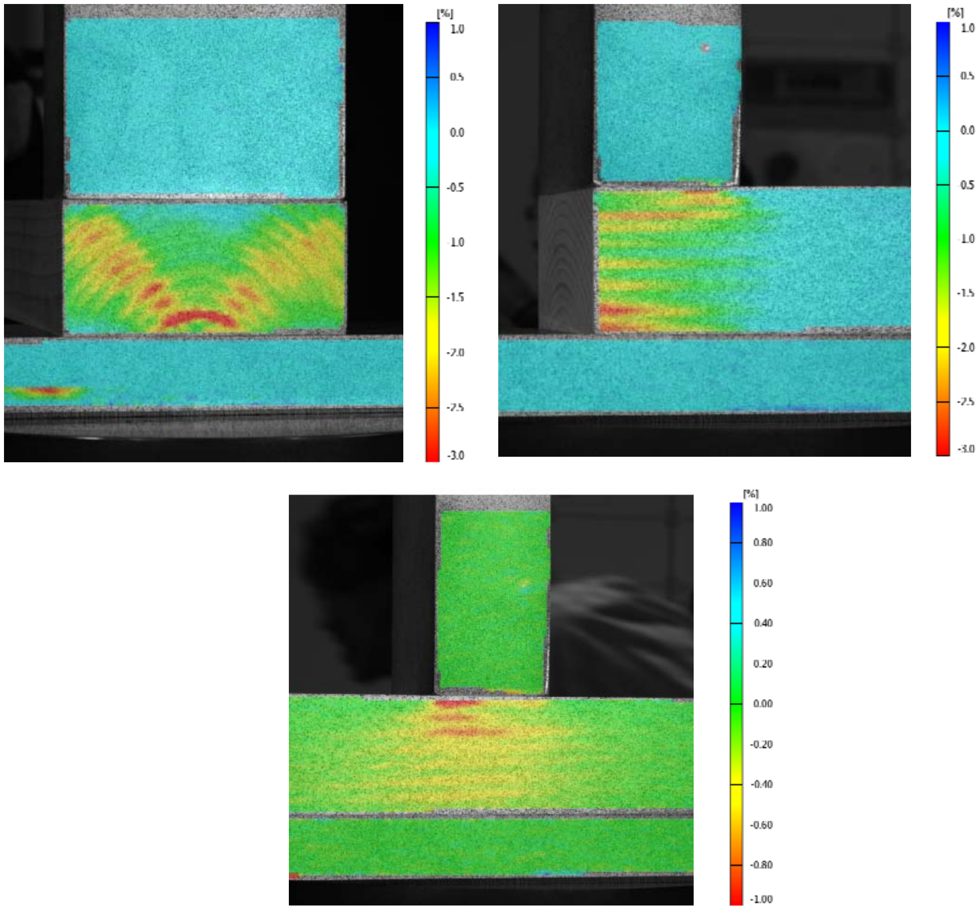

Figure 8. Strain plots at the load 19.6 kN corresponding to an average stress of 4.6 MPa over the contact area. Strain plot, for the first and second specimen where the wooden members end at the left side of the stiff vertical member, from a side-wise view (left) and from a front-wise view (upper right) respectively. Strain plot for specimen with continuous wooden member on both sides of stiff vertical member (lower right).

If the optical system is applied on experiments with bi-axial loading of specimens, that would also give information regarding stiffness to horizontal slip between members, it may add valuable information and contribute in the development of advanced models for this type of connections.

4. Bi-axial testing setup

From the experiment with the fastener connecting the sheathing to the framing it has been shown that the stiffness and strength of the connection are strongly dependent on the loading direction of the sheathing in relation to the timber member. Bi-axial tests would

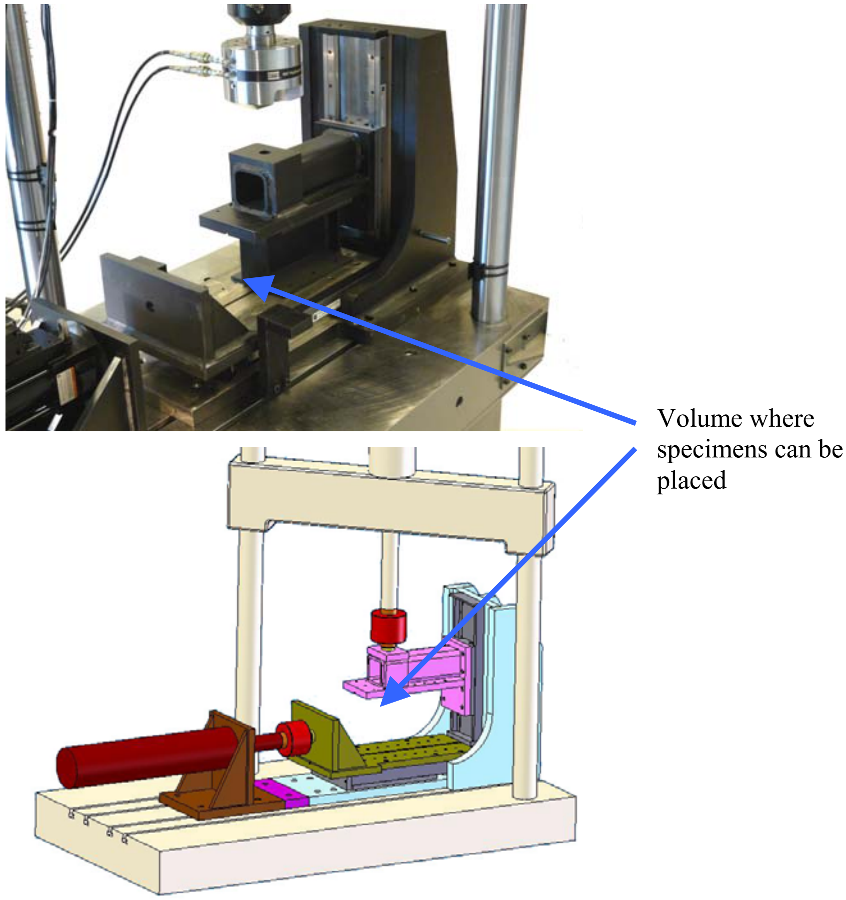

Figure 9. Photography of fixture for bi-axial tests mounted on the testing machine (top) and a rendition of a 3- dimensional model of the fixture (bottom).

make it possible to evaluate the stiffness in one direction when the fastener is already loaded in another direction and thus establish a coupled stiffness relation for the connection. It has also been stressed that the evaluation of the other connection, in which a wooden member is loaded perpendicular to grain, would benefit from bi-axial tests where the specimen could be loaded in shear at the same time. Bi-axial tests with simultaneous loading in two directions may be performed using the equipment shown in Figure 9. Figure 9 (left) shows a special fixture for specimens mounted on the testing machine. Figure 9 (right) shows a rendition of a 3-dimensional model of the same fixture. The volume where tests can be performed is indicated. Test specimens with size up to about 500×400×200 mm^3 can be fixated within this volume. In the bi-axial tests that are planned also CCD-cameras will be used in order to obtain complete information about the strains while the specimen is loaded bi-axially.

5. Conclusions

The overall performance of stabilizing shear walls is strongly dependent on the stiffness, strength and ductility of the fastener connecting the sheathing and the frame. By using an optical data collecting system, giving strain fields for loaded specimens, it was possible to show the location and magnitudes of strains in detail in the area surrounding a screw. Another critical connection in shear walls is that between a stud and a rail where wood may be compressed perpendicular to the grain. Also for tests on such connections the optical system was used to identify strain fields in detail. The gained experience will in the future be useful in the planning of more advanced test setups in which test specimens are loaded bi-axially. Results from such experiments will enable the development of elaborate models able to capture the coupled stiffness when the load history on connections comprises loading in more than one direction. The promising results from the present investigation imply that the optical data collecting system will be very useful in that development.

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