Analysis of shear walls for multi-storey timber buildings
ANALYSIS OF SHEAR WALLS
FOR MULTI-STOREY TIMBER BUILDINGS

Johan Vessby

LINNAEUS UNIVERSITY PRESS

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**ANALYSIS OF SHEAR WALLS FOR MULTI-STOREY TIMBER BUILDINGS**
**Doctoral dissertation, School of Engineering, Linnaeus University 2011.**

The figure on the front page shows one of the timber buildings located in the Limnologen building complex in Växjö in southern Sweden. To the left is an ArchiCAD drawing by the firm Arkitektbolaget Kronoberg AB, designers of the building, and to the right is a finite element model of the same building.

ISBN: 978-91-86491-73-4
Printed by: Intellecta Infolog, Gothenburg
This doctoral thesis addresses questions of how wind loads acting on multi-storey timber buildings can be dealt with by structural design of such buildings. The conventional use of sheathing either nailed or screwed to a timber framework is considered, together with other stabilizing structures such as cross-laminated timber panels.

The finite element method was employed in simulating the structural behaviour of stabilizing wall units. A series of studies was carried out of walls in which the sheathing was nailed to a timber frame. Different structural levels were studied starting with modelling the performance of single sheathing-to-framing connections, to the use of models for studying the overall structural behaviour of walls. The results of calculations using models for simulation of walls subjected to different loading agree reasonably well with experimental results. The structural properties of the connections between the sheathing and the frame, as well as of the connections between the members of the frame, were shown to have a substantial effect on the simulated behaviour of shear wall units. Both these types of connections were studied and described in appended papers.

Regarding cross-laminated timber wall panels, it was concluded that walls with a high level of both stiffness and strength can be produced by the 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 transmitted from one panel to the next in an efficient manner.

Other topics in the thesis include the properties of connections between shear walls and the rest of the building. Typically high tension forces occur at specific points in a timber structure. These forces need to be transmitted downwards in the structure, ultimately connecting them to the substrate. A lap-joint that may be used for this purpose has been studied using generalized Volkersen theory. Finally the maximum capacity of a conventional rail to substrate connection has been examined using linear and nonlinear fracture mechanics.

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

Den föreliggande doktorsavhandlingen behandlar frågan kring hur vindlast på flervåningshus i trä kan hanteras och hur man genomför olika typer av dimensioneringsberäkningar knutna till detta. Avhandlingen behandlar huvudsakligen konventionella skivbeklädda träregelväggar, men tar också upp andra typer av stabiliserande skivor, så som massivträskivor.


Vad gäller massivträskivor kunde slutsatsen dras att dessa har goda styrke- och styvhetsegenskaper. Detta gäller även då flera skivor av samma typ förbinds under förutsättning att sammanfogningen sker på ett korrekt sätt.

Ytterligare ämnen i avhandlingen inkluderar egenskaperna i förbanden mellan stabiliserande väggar och resten av byggnaden. Det är vanligt att stora dragkrafter förekommer på vissa ställen i ett trähus. Dessa måste successivt föras nedåt i byggnaden tills de når grunden. Ett överlappsförband som skulle kunna användas för att överföra sådana krafter har analyserats med generaliserad Volkersen teori. Slutligen har maximala kapaciteten i ett konventionellt förband mellan syl och grund undersömts med hjälp av linjär och ickelinjär brottmekanik.

Nyckelord: flervåningshus, träbyggnad, vindstabilisering, skivväggar, massivträskivor, förbindare, förband
ACKNOWLEDGEMENTS

The consultancy firm Tyréns AB provided financial support for the work presented in this thesis, which started in the Wood Design and Technology Programme (WDAT) at the School of Technology and Design of Växjö University, and was finalized at the School of Engineering at the Linnaeus University. Part of the work was financially supported by the European Union’s Structural Funds – The Regional Fund in the “Multi-Storey Timber Buildings” project.

I would like to express my sincere thanks to Vice-President Tomas Alsmarker, Tyréns, for initiating the research project and for his inspiration and support. Professor Bo Källsner has played an important role in helping me finalize the thesis, and to all of them I am very grateful. Three other persons in particular in this research environment have helped me with the work: Professor Anders Olsson, Professor Erik Serrano and Mr. Bertil Enquist. Working with each of you has played a crucial role in completion of the thesis. I am looking forward to further fruitful collaboration. Professor Ulf Arne Girhammar has played an important part, particularly in one of the appended papers, for which I am most grateful. Professor Hans Petersson is an encouraging leader who started the research environment of which I had the opportunity to be a part during the last few years. Thank you for your support throughout. I would like to thank other colleagues as well, both at the School of Engineering and at the consultant firm Tyréns, for all the interesting discussions during the project.

I would also like to thank my family, my parents in particular, for encouraging me throughout. Finally, I want to express my thanks to Linda and our daughter Astrid for being by my side and supporting me in the process.

Växjö, April 2011

Johan Vessby
APPENDED PAPERS


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1 INTRODUCTION

1.1 General remarks

The use of timber to provide the structural system of single-family houses has a long tradition in Sweden. According to the Ministry of Industry, Employment and Communications [49], timber is contained in the load-bearing parts of some 90% of all single-family houses built in Sweden today. There are many reasons for the choice of wood here, with the two major ones possibly being the long tradition of building with timber, and the ample supply of timber as a raw material. Today, Sweden has about 22.6 Mha of forest land [58], 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 single- or two-storey structures to be of timber. This led to other materials replacing timber, as well as to many broad avenues being built and dividing the cities, and to some buildings being

![Figure 1. Two timber-based multi-storey buildings in southern Sweden. A multi-storey wooden structure, erected in Kisa in 1902, representing one of the last such structures built in Sweden prior to the passage of restrictive laws, is shown to the left, and Wälludden, the first modern multi-storey wooden building in Sweden, built in Växjö in 1995, is shown to the right.](image)
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 being used only for single-family houses that were a maximum of two storeys high. When the regulations were changed after more than 120 years 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 of regulations in the mid-1990s.

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 [50]. This wind loading and the gravitational forces represent the basic load cases that need to be borne in mind when designing a building in a non-seismic region. How such loads can be dealt with in timber structures is of particular 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 architectonic 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, scope and limitations

The aim of this 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 that have been more closely investigated are constructed either of sheets nailed or screwed to a timber frame or of cross-laminated solid timber panels. 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. Several of the papers are based on experimentally obtained data for connections used in shear walls. In general the numerical models may be used for any type of experimentally, or otherwise, obtained data. Also, for the most part North European design conditions are considered.
2 HORIZONTAL LOADING OF TIMBER STRUCTURES

2.1 General remarks

Buildings 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. 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 [30]. Sweden and the other Nordic countries, fortunately, are very seldom exposed to seismic actions of such severity, wind-related disasters being far more costly in terms of property damage and casualties, cf. Davenport [11]. 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 [59]. 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 literature, see e.g. Filiatrault et al. [20] and White and Dolan [65]. Boughton [5] presents the results of full-scale testing of timber structures subjected to horizontal loading. Tests results of this kind provide valuable information on the behaviour of timber structures loaded to failure.

2.2 Horizontal loads acting on timber structures

Various aspects of the general behaviour of a load-bearing 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 with a tension and a compression edge zone or “chord”. 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. For a fully anchored shear wall reaction forces on the substrate are compression at the trailing stud, tension at the leading stud and shear along the bottom rail. A pure shear field is obtained in the shear wall.

The structural behaviour of the one-storey timber structure shown in Figure 2 also partly holds for a single storey in a multi-storey structure, 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, see Figure 3(a). These forces, in particular those acting in uplift, may be very difficult to manage for the structural engineer since they, theoretically, may cause the storeys to separate, see Figure 3(b and c). The major reason for the problem to occur in uplift is the relatively low self weight of wood based structures, in particular when compared to structures based on the use of concrete. The shear forces in turn are normally easier to transmit to the storey below preventing the storeys to slide on top of each other, see Figure 3(d and e).

The structural response that timber shear walls show to loading has been subjected to substantial research. An overview of some of the literature can be found, for example, in Dinehart, Shenton and Harry [13], Dolan and Johnson [14], Filiatrault et al. [19] and Itani and Cheung [32] and in Lam et al. [44].

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

During the mid 20th century, wind gust speeds were often used as a basis for estimates of the wind loading of structures, see Hertig [29]. 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 [18]. Instead, the most common approach in assessing wind loading is to base calculations on the average wind speed for a 10-minute period. For buildings in general, this serves as a reference value, one that varies between different countries and between different regions within a country.

In the Swedish structural code used for wind load (prior to the introduction of Eurocodes), BSV [6], an equivalent characteristic 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. 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 $C_{\text{dyn}}$ accounts for the increase in wind load due to the
dynamic nature of the wind and $C_{\text{exp}}$ is an exposure factor taking into account
variation due to the height above the ground and the influence of the
surrounding landscape.

Taking into account the load distribution over the building, the characteristic
static wind load, $w_k$, is determined by multiplying $q_k$ by the shape factor $\mu$, 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 $C_{\text{exp}}$ has been a subject of discussion for a number of years [30]. 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.

$q_{\text{ref}}$ 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 $q_{\text{ref}}$ is obtained as $0.5 \cdot \rho \cdot v_{\text{ref}}^2$, where $\rho$ is the air density and $v_{\text{ref}}$ 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 probability of
exceeding the given level is $1/R$ any given year. For a given expected lifetime
of a structure, $L$, the risk $r$, see [30], of a wind with a return period of $R$ years
exceeding this speed, can be obtained as

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

<table>
<thead>
<tr>
<th>Return period, R</th>
<th>Expected lifetime, L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30</td>
</tr>
<tr>
<td>30</td>
<td>0.638</td>
</tr>
<tr>
<td>50</td>
<td>0.455</td>
</tr>
<tr>
<td>100</td>
<td>0.260</td>
</tr>
</tbody>
</table>

Table 1. Risk of a wind speed with a return period of $R$ years being exceeded during different expected lifetimes.
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 at least one year is 0.636.

### 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 blown 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, see Figure 4. Four different series of values based on wind measurements that were made are shown in Figure 5. 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, when damping wooden buildings, information concerning the maximum wind gust speed should be interpreted quite differently than information concerning the average wind speed during a 10-minute period.

![Figure 4. SMHI’s weather report site south of Växjö close to the university campus.](image-url)
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 hurricanes.

Figure 5. Wind speed values in Växjö during the Gudrun storm 8th January 2005 according to SMHI.

Figure 6. (a) An old timber structure that failed in the Gudrun storm. Photo S. Palmblad. (b) High wind loads may cause vibrations in tall timber buildings.
After the Gudrun storm, various failures of buildings were reported. One of these is shown in Figure 6(a). 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 velocity and acceleration levels should be regarded as acceptable for those living in tall wooden buildings, see Figure 6(b). If these are established, a serviceability limit state analysis can be performed in order to determine whether a given building fulfils these requirements or not.

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 7. The normalised spectral density is given as a function of the frequency represented on a logarithmic scale. Wind has a broader frequency domain than an earthquake, although its mean level is much lower.

As shown in Figure 7, 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 [30] 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 the damping in timber structures usually is high, displacements are often damped out rather quickly.

Ellis and Bougard [17] have shown that for a six-storey timber-frame structure 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 [23], whereas a practical approach for direct use in structural design can be found in Akerlund [68].
Figure 7. Frequency content of winds and of earthquakes according to Holmes [30].

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

The current praxis in Sweden at design of shear walls in timber buildings is 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 which, the designer needs to verify that the building can resist. It has been 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 giving rise to structural noise and vibrations. Undesired events of this kind indicate that the serviceability limit state can be of considerable importance in the design 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 [9]. Little is said in Swedish design recommendations, however, regarding what top displacement of high timber buildings are allowable and what levels of vibrations can be accepted. In Eurocode 5 the top displacement, \( u \), is restricted to \( u \leq h/300 \), \( h \) being the current height, for the characteristic combination causing permanent damage. The corresponding value in the German DIN- code is \( u \leq h/500 \), allowing less displacement. A further developed discussion on serviceability analysis may be found in Källsner and Girhammar [40]. 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. 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 [10]. 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 [17], 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 [63], 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 [61] and Leung et al. [46] for example.
3 STABILIZING SYSTEMS FOR TALL TIMBER BUILDINGS

3.1 General remarks

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 a global perspective there are three different main principles for stabilization of buildings, by use of frames, by use of diagonal bracing or by use of shear walls. Since the latter is totally dominating the design used in timber buildings we will focus on that main type here. In Figure 8, different types of such systems and design philosophies are presented. In the top row, Figure 8 (a–b), two different ways of treating conventional shear walls are shown. To the left (a) a system conventionally used today in one- and in two-storey buildings is shown. The wall consists of sheets nailed or screwed onto a timber frame. At the design of this wall type it is usual to neglect the influence of the sheathing above and below openings such as windows and the contribution of narrow sheets. This exclusion of the sheathing in the walls results in the calculations regarding the horizontal load-bearing capacity of the structure being conservative, but also lead to (theoretically) 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. Software is available and the topic is under intense research. There are also other, though rougher, methods available for dealing
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.

with the wall in its entirety. One of these, which can be found in the international building code (IBC) [31], is based on the reduction in capacity due to use of the opening area as compared with the total wall area. A plastic design approach developed by Källsner and Girhammar [40] would also be an alternative method, including the entire wall rather than only the parts indicated in (a). In fact, using this method would also enable the inclusion of lateral walls for hold-down of the shear walls. By doing that, the three-dimensional effects of the building may be included in the analysis, improving it even further.

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 areas) consists of solid timber, whereas the entire 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 the 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 shown in the figure, the walls are viewed as plane elements without any interaction with the laterally located walls. Considering such interaction in the analysis would make it easier to handle the anchoring issue, since the calculated uplift forces would be lower. This is a topic discussed by Andreasson [2], for example. As previously mentioned, this may be included in the analysis using the approach suggested by Källsner and Girhammar [40].

3.2 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, fibre board and oriented strain board (OSB). Achieving sufficient lateral resistance by use of timber frame walls 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 [9], Källsner [37], Degerman [12], Åkerlund [67], Gyproc Handbok [27] and McCutcheon [47]. Some of the design principles used for low-rise structures can be applied on multi-storey buildings as well.

Today timber frame structures are often 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 [22]. 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

![Figure 9. Schematic view for a simplified analysis of a one-storey portion of a timber frame structure.](image)
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 [9], 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. [35], Salenikovich and Dolan [56] and Girhammar and Källsner [24], 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 limit state analysis, underestimating the overall global stiffness of the structural system. Another consequence of the simplifying assumptions is that the analysis leads to very high compression 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 [56] and by Kessel and Dettmann [36].

3.2.1 Types of connections for vertical forces

Various types of connections or joints can be used for transfer of 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 foundation are of considerable importance for the overall performance of a structural system. These connectors should be designed so that their capacity is granted in a long term perspective.
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. One innovative method of connecting two storeys to each other would be to use fibre reinforced polymer (FRP) glued to the studs instead of the mechanical joint suggested in Figure 10(b). A similar type of lap-joint is discussed and analysed in the appended Paper VI. 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 is supported by a piece of metal instead of the rail, so as to avoid loading perpendicular to grain in the bottom rail.

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 large uplift forces can be handled effectively in the ultimate limit state. It is also 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 [38] 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 by tests performed on partially anchored shear walls, Girhammar and Källsner [25].

3.2.2 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 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

![Figure 11. Experimental results for compression of a rail, i.e. compression perpendicular to the grain, see Vessby, Olsson and Källsner [64].](image)
moisture content for the different tests also differ, yet the most important difference is that of loading in the centre versus at the end of the rail. It can clearly be 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 [64]. Practical design recommendations can be found in for instance Larsen and Riberholt [45] and in Eurocode [18].

### 3.3 Use of solid timber panels in the stabilizing system

An alternative way of stabilizing timber buildings is to use solid timber elements (often referred to as cross-laminated timber, CLT). 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 12 shows two 96 mm thick, five-layer timber panels.

![Figure 12. Detail of timber elements consisting of 5 layers](image)

(a) conventional design of a solid timber panel,
(b) alternative design of a panel.
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 CLT-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. CLT-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 13(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 [16].

3.4 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 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 METHODS FOR ANALYZING SHEAR WALLS

4.1 Introduction

In timber buildings shear walls with nailed or screwed sheathing are still the most commonly used wind bracing components. Many different techniques have been used to analyse this type of walls, some of which will be discussed in this chapter. In the design of such walls not only the strength of the sheathing-to-framing connections may determine the capacity of the wall, but also the buckling strength of the studs or the sheathing, the compression strength perpendicular to grain at the bottom rail or the capacity of the hold-down connections. Any of these properties may give the critical design load of the wall. Nevertheless, the most crucial design parameter for a single shear wall is normally the capacity of the sheathing-to-framing connections.

Besides the analytical methods available for analysis the finite element method is becoming increasingly popular due to its flexibility and its capacity to simulate any geometry. Therefore an introduction and a background to the possibilities connected to the method applied on shear wall analysis will be discussed in the following as well as the analytical methods available. Still the analytical approach is the most common method on an everyday design basis.

4.2 Elastic analysis

Some of the simplest, and most used, models for analysis of the capacity of shear walls are based on the theory of elasticity. Basic assumptions in these models are rigid framing and sheathing where the framing members are interconnected by frictionless hinges. The bottom rail is assumed to interact fully with the foundation. Based on such assumptions it is possible to derive expressions for the capacity of a shear wall, see e.g. Tuomi and McCutcheon [62], Källsner [37], Källsner and Girhammar [39] and Åkerlund [67]. The results in these models are determined using the elastic approach where the
shear wall capacity is based on the most loaded fastener. Due to the simplicity of the elastic models they are frequently used by practicing engineers in the design process.

4.3 Plastic analysis

A plastic approach for the analysis of a timber building may be considered at several different levels, from the complete building to the detailed analysis of a single fastener. In Källsner and Girhammar [40] an attempt is made to include not only the positive effects of the dead load but also the effects of the lateral walls. Some of the underlying articles are e.g. Källsner and Girhammar [41], [42] and [43]. This type of three-dimensional consideration may be done if the deformation capacity needed is secured and thus indicates the importance of verification of the plastic behaviour, including both fastener strength and ductility. An alternative approach is suggested by Ni C. and Karacabeyli E. [48], taking into account the dead load from upper storeys and the partial anchorage of the leading stud.

The performance of the two models is compared for a simple load case in e.g. Salenikovich A. and Payeur M. [57] and in Rainer H., Ni C. and Karacabeyli E. [54]. There is a potential in the plastic approach to specify more realistic load paths than is the case in an elastic analysis. This can be of great importance for load levels approaching the ultimate capacity as the load transferred by a single fastener changes both in terms of magnitude and direction.

4.4 Eurocode – Method A and Method B

Since the European standard Eurocode 5 [18] was introduced its design methodology is increasingly used in practice. In the code there are two alternative design procedures for horizontal stabilization being referred to as method A and method B, respectively. In both cases the general idea is to calculate the racking strength of a single shear wall as a product of the lateral design capacity of the sheathing-to-framing fastener and the number of spacings in-between these connections. The difference between the two methods depends on the different boundary conditions being assumed. In method A, the leading stud, the one closest to the loaded side of the wall, is assumed to be fully anchored to the substrate and thus resisting the overturning moment. By assuming this, a pure shear flow will be obtained in the shear wall, plasticity being reached for all the fasteners along the perimeter of the sheathing at roughly the same time. In the alternative method, B, the bottom rail is assumed to be the only partly anchored to the substrate, allowing all studs to separate from the bottom rail when subjected to uplift. The failure mode for the wall in case of method B is completely different as
compared to the case with a fully anchored leading stud. The formulas given in method B for Eurocode 5 are based on a semiempirical theory assuming that the wall is a rigid body, which is not completely correct. Using this doubtable assumption, the vertical load on the wall is included in the analysis, however there are no possibilities to take into account the anchoring effect of lateral walls.

Both methods are to be considered as methods based on theory of plasticity since they assume the same load (magnitude) being transferred by all the fasteners. However, no recommendations are given on any larger scale than parts of walls, i.e. single shear walls. Much of the benefits of an over-all plastic analysis, with possibilities to e.g. include the effects of lateral walls, are not indicated and thus not regulated.

4.5 Introduction to finite element 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 using the finite element method, see e.g. Foschi [22], Gupta and Kuo [26], He, Lam and Foschi [28], and Itani and Cheung [32]. 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 overview of various important choices to be made regarding the modelling when simulating the behaviour of walls. For an introduction to the finite element method the reader is referred to e.g. Ottosen and Petersson [52].

In shear walls there are three different components that can be identified: the timber frame, the sheathing and the fasteners. Both a physical and a finite element representation of these three components are shown in Figure 14. The framing in the shear wall is normally modelled by use of beam elements, each element being assigned a cross-section and being placed in the symmetry line of the physical member. This approach is normally effective with respect to computational cost yet it introduces a geometrical approximation into the model since the precise location of the interface between the members is not specified. In addition, the effect of compression perpendicular to the grain direction needs to be modelled with use of certain simplifying assumptions, e.g. spring elements with a given load-displacement characterisation possible in a full 3-dimensional analysis. The contact area 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, see e.g. Pang and Shirazi [53]. This also provides the possibility of including further material parameters, such as distortions due to variation in moisture content, see e.g. Ormarsson [51].
The geometry of the sheathing is ideal for modelling by use of shell elements. It may be important to include the effects of in-plane interaction between the sheets, especially in the early stages of the loading history.

The third and most crucial element in obtaining sufficiently accurate simulation results is the 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. In a two-dimensional model the connections can be modelled by use either of a single-spring model or of a spring pair model. In order to reflect the different strength and stiffness properties in the two main directions in the connections between the timber members, the spring-pair model is normally suitable. For the connections between sheathing and framing, the choice of a model is less self-evident. The single spring model is the simplest alternative, but the differences in characteristics between the parallel and the perpendicular loading direction with respect to the wood fibres are typically not considered in a realistic manner. These differences can be taken into account by use of the spring pair model, although with the drawback that this model normally (uncoupled spring-pair models) overestimates the strength and the stiffness of the connection. In both models there is also a choice to make between the use of nonlinear elasticity or the use of plasticity. Further information regarding models for elasticity and plasticity can be found in Ottosen and Ristinmaa [55], for example. Considerable research has been directed towards the question of how the connection between sheathing and framing should best be modelled. Some of this work is found in e.g. Foschi [21], Judd and Fonseca [33] and Pang W.C. and Shirazi M. H. [53]. A further specialized topic is

Figure 14. 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.
that of the cyclic loading of connections. This topic is dealt with in part by e.g. Dolan and Madsen [15] by Tarabia and Kamiya [60] and by Xu, and Dolan [66]. 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 representing the nails, for example, as an elastic medium in the model rather then as individual connections, see e.g. Kasal, Leichti and Itani [34] and Alsmarker [1].
5  OVERVIEW OF APPENDED PAPERS AND GENERAL CONCLUSIONS

5.1  Topics of the appended papers

There are seven papers appended to this thesis. In paper I finite element modelling of conventional shear walls for light-frame timber buildings is examined for different prerequisites. In paper II, in contrast, the properties of cross laminated timber panels are investigated. These panels are common for stabilizing purposes in modern timber buildings why the investigation is motivated. The papers III to VII are more detailed in their scope, mainly aiming at further highlighting different aspects of connections in stabilizing systems. A brief summary of the appended papers is given below.

Paper I deals with the influence of different modelling assumptions on the structural behaviour of shear wall elements in which the sheathing is nailed to a wooden frame 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 that includes softening, i.e. a decreasing load bearing capacity at increasing deformation after the maximum capacity has been passed. The influence of the ductility of the sheathing-to-framing connections is investigated by varying the softening, 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 by testing of one wall segments. The overall agreement between results of simulations and of laboratory tests is good when reasonable model assumptions are employed.

In Paper II 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 no. 1, 3 and 5 lying in one direction and layers no. 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) exposed to 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 non-partitioned 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. The purpose of the paper is to investigate the potential in using CLT-panels for stabilisation.

Paper III deals with the detailed modelling of the sheathing-to-framing connection in a shear wall. Four different elastic models for sheathing-to-framing connections are presented and evaluated at the single connection level and at the level of a shear wall. Since the models are elastic they are suitable mainly for cases where the sheathing-to-framing connections are subjected to monotonically increasing displacements. Of the four models one is uncoupled and the others are coupled with respect to the two perpendicular displacement directions in a two-dimensional model. Two of the coupled models are non-conservative, while the third is conservative, indicating a path independency with respect to the work done to reach a defined state of deformation. When the different models are compared it is obvious that the uncoupled model gives strength and stiffness values higher than the others; however it is not obvious which of the models to use in a shear wall analysis. Each of the models do have their respective advantages and disadvantages. For the experimental data used as input to the analyses of this study however, a coupled non-conservative model seems the most appropriate. As previously indicated the sheathing-to-framing connection is crucial for the performance of the wall. This fact implies that the accuracy of the model used for simulating the connection is of great importance.

Recently, a Swedish handbook for design of shear walls has been presented based on the assumption that the sheathing-to-framing can be modelled using ideal plastic characteristics [40]. The authors of Paper IV have developed a simplified design method utilizing transverse walls for anchoring of shear walls against uplift. Calculated load-carrying capacities of transverse walls using the simplified method seem to be in fairly good agreement with calculated capacities using the finite element method and agree fairly well with measured capacities obtained from full scale testing where transverse walls were included.
In Paper V the influence of contact between sheets in a shear wall is discussed. The analysis models used for shear walls often neglect the contact forces between the sheets. In partially anchored shear walls, especially, these contact forces can become significant. In the present study, which only comprises walls without openings, the effect of these contact forces is evaluated for different loading and geometrical conditions using the finite element method. The analyses show that the internal force distribution in the wall is significantly affected by taking the contact forces into account. However, with respect to the load-carrying capacity of the wall, these contact forces have only a minor effect. The main reason for this is that the contact forces between the sheets can be replaced by forces transferred via the sheathing-to-framing joints along the top rail and to some extent also by forces transferred via the sheathing-to-framing joints along the upper parts of the vertical studs joining the different sheets. Paper V as well as Paper IV, are contributions to the design approach in situations where larger parts of a house or a wall are included in the analyses. The papers thus address some of the difficulties associated with these types of analyses, often referred to as global analyses.

In Paper VI a wood-FRP lap-joint is scrutinized. Wood specimens to each of which a laminate of carbon fibre reinforcement polymers (FRP) was glued (creating a lap joint in each case) were loaded to failure. A total of 15 specimens of three types differing in the glued length (anchorage length) of the FRP laminate (50, 150 and 250 mm respectively) were tested, their strength, stiffness and strain distribution being evaluated. Synchronized digital cameras (charge-coupled devices) used in testing enabled the strain fields to be measured. These results were also evaluated both analytically on the basis of the so-called generalized Volkersen theory and numerically by use of the finite element method. The lap joints showed a high level of stiffness as compared with mechanical joints. A high degree of accuracy in the evaluation of stiffness was achieved through the use of the contact-free evaluation system. The load-bearing capacity for joints of this type was found to be dependent upon the anchorage length in a non-linear fashion, i.e. showing a size (length) effect. The experimental, analytical and numerical results were shown to be in close agreement with respect to the strength and the strain distributions obtained.

The paper aims at one of the largest challenges with timber construction, that of anchoring the shear walls. The technique assessed herein may be a suitable solution for some buildings.

Paper VII relates to the topic of anchorage of shear walls. At times eccentric forces between the sheathing and the anchoring devices may be introduced in the bottom rail. In severe cases such forces may cause the rail to split and to fail in a brittle manner. In this study, fracture mechanics was applied to develop a simple closed-form hand calculation expression for estimation of the ultimate load capacity of the rail. Finite element analyses (FEA) using both linear elastic fracture mechanics (LEFM) theory and a nonlinear fictitious crack model (FCM) were also used to predict the ultimate load bearing
capacity of the rail. The hand calculation model was compared with the FE-models and a good agreement was obtained. The results obtained with the various fracture mechanics models were compared with results available from previously performed experimental tests and again good agreement was obtained. A general conclusion is thus that the fracture mechanics theory is an adequate approach for the case studied, and that the hand calculation expression developed could be useful for structural design.

5.2 Concluding remarks and new findings

The overall aim of this work has been to investigate different ways in which tall timber structures can be stabilized against the effects of strong horizontal wind loading. The main emphasis has been on systems based on the conventionally employed shear wall, although the performance of cross-laminated timber panels has also been examined in one of the papers. The most important results from the study may be summarized as follows:

- When analyzing a shear wall, anchorage of the leading stud is vital for the result. Previously, the majority of the models have assumed hinged interaction between timber members, representing the case of full anchorage. This thesis discusses different models for the timber-to-timber interaction and possibilities for a broader variety of anchorage conditions are analysed.

- Transmission of uplift forces between units of shear walls built on top of each other may be challenging for the practising engineer to handle. An example of a wood-FRP lap-joint is studied in detail experimentally, analytically and numerically. This type of joint may be used in some special cases to resist uplift forces.

- If a plastic method for analysis of shear walls is to be used, the plastic behaviour of the connections has to be ensured and brittle failure modes must not occur. However, using anchor bolts along the bottom rail may cause splitting failure in the wood material due to tension and shear stresses perpendicular to grain direction. The thesis describes a fracture mechanics approach to determine the maximum capacity of the bottom rail.

- It has been shown that the choice of model for simulating the sheathing-to-framing fastener has a great impact on the result. In the thesis it is shown that it is important to take into account the different properties of the connections in the different directions, parallel or perpendicular to the length direction of the stud.

- Coupled finite element models of sheathing-to-framing joints may be used to simulate the complex behaviour of a single fastener interacting with wood material. Such models are suggested in the thesis on a
theoretical basis. In order to be truly useful they must be complemented with advanced biaxial experimental studies.

- Cross-laminated timber panels were tested experimentally as deep beams. The stiffness and strength of the panels were found to be significant, indicating their potential use as load-bearing elements.

5.3 Proposals for further research

Different stabilizing systems for timber structures have been discussed herein. Of the different systems considered, the conventional timber frame with nailed or screwed sheathing is by far the most common in practical use today. For structures more than four storeys in height, difficulties may arise in achieving designs that provide enough stiffness and load-carrying capacity if design rules and design practice in general use today are employed. In addition, contemporary architecture, including a large number of substantial openings in the facade, sometimes makes it difficult to stabilize even one- or two-storey houses of wood when using the hand calculation methods available. Further research on these aspects is needed. The two fields of research suggested here complement 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 purpose of the latter could again be considered in two parts: serviceability limit state analyses and models that could be used to evaluate a more comprehensive calculation model for the ultimate limit state to be implemented in Eurocode.

The sheathing-to-framing connection is one of the important topics for gaining insight into the behaviour of a conventional timber frame wall. The ductility, crucial for an overall plastic analysis, depends very much on the ductility of the sheathing-to-framing connection. The overall dependency on the fastener ductility should be examined further. Furthermore, different ways of modelling the connections will result in different predictions of the load-displacement relations when applied to a single shear-wall unit. Such connections 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. A study of that type of sheathing-to-framing connection should include both experimental tests performed in different directions with respect to the length direction of the timber member and the creation of adequate theoretical models for numerical simulations taking into account the dependence of the properties on the loading direction. In finite element models of such connections, coupling with respect to orthogonal directions should be included and possibly plastic properties as well. A
theoretical basis for the models to be used was presented in Paper III. The next step would then be to perform experimental investigations with the aim of verifying the general theoretical assumptions made and also giving further information on how the behaviour of the connection should be interpreted.
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 three-dimensional models of large parts of the buildings. The inter-element connections play a crucial role in the overall behaviour of such a structure. Apart from the fasteners of such connections, contact between the different bearing parts is also important. The contact forces are normally larger at locations where openings, such as window openings, occur. The overall dependency of the fastener ductility should be examined further. Simulations of structures using models that capture the properties of the inter-element connections in a horizontal as well as in a vertical direction are 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 a quite realistic picture of the vertical forces 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 respect to the number and placement of the fasteners employed, as based on information that can be obtained from such a model. In addition much could be found about the serviceability state from such a high level model, and it could also be used to predict realistic force paths through the building. This in turn may facilitate the development of more accurate and sophisticated hand calculation models to be used in the structural codes.
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