literature review for MSc thesis:

‘MULTI-STOREY TIMBER-FRAME BUILDING’
modelling the racking stiffness of timber-frame shear-walls
Introduction
This report presents a literature review concerning stiffness- and force-distribution issues and modelling aspects of timber-frame buildings. This review is a part of the MSc thesis ‘Multi-storey timber-frame building’. The aim of the project is explained first. Secondly, the aspects governing the shear-wall behaviour are discussed. Thereafter experimental test-data is presented, which will be used in the next phases of the master’s thesis. The report on the results of the literature review will be finished with a chapter concerning modelling and analysis of the timber-frame structure. A reference list of the consulted sources is included in the last part of the report.

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1 Aim of the literature review

1.1 Problem definition

Each structural design needs to fulﬁl the demands in ultimate limit state (ULS) and serviceability limit state (SLS). The requirements in ULS have to do with prevention of collapse, failure and rupture. In ULS, the force distribution within the structure is often most important. Prevention of undesirable deformation or vibration is important because these can affect the appearance or effective use of the building. Therefore, the stiffness of the structure is analysed in SLS.

A safe and reliable estimation of the deformations is required before constructing multi-storey timber-frame buildings. In Eurocode 5 (NEN-EN 1995), a calculation method is given to check the timber-frame wall and floor panels for the requirements in the ultimate limit state. The force distribution in ULS is dependent of the stiffness distribution within the structure. In the calculation method, the forces on the wall or floor diaphragm are assumed to be known. However, the way in which the stiffness of timber-frame diaphragms has to be determined is not explicit prescribed in the current code. Some directives are given to calculate a joint slip modulus (K_{slip}) for dowel-type fasteners. However, the applicability of this joint-slip modulus for stiffness estimations of shear-wall diaphragms is unclear, and additional information about the theoretical and experimental backgrounds of this modulus is not given. Hence, the information and directives from the code contain gaps on this topic. Several approaches and assumptions are possible. Aim of this master’s thesis is to gain knowledge about the stiffness characteristics in both SLS and ULS to provide insight in analysis and modelling of the stability-system. This insight will result in more reliable prediction of deformations, and force-transfer, in timber-frame buildings.

1.2 Position of the literature review within in the project

Information on shear wall, anchorage, and fastener behaviour and experimental tests, have been found in literature and will be presented in the next chapters. In this paragraph, an overview will be given of the plan of approach for the next parts of the project. In the remaining chapters of this literature review, the results of the literature review will be presented. These will be analysed further in the next phases of this master’s project. Two different phases can be distinguished. Discussing these phases will show the application of the literature review. First, the standard shear-wall will be analysed (phase A). The standard shear-walls will be analysed to get a clear understanding of the modelling approach and stiffness values for standard walls. With standard walls is meant, walls without openings (perforation), sheeted on one or both sides, height h = 2,7 m, width b = 0,6 ... 4,8 m.

Secondly, when the analysis of standard walls has been finished, another step will be made to get a clear understanding of the modelling approach and stiffness values for perforated walls (phase B). Perforated walls are walls with one or more openings such as windows and doors. These walls can also be a composition of several standard walls or wall parts. In figure 1 on the next page, phase A and B are explained more in detail. These phases will not be elaborated in this report.

1.3 Intended thesis results

On the condition that the comparative research (in figure 1) do not give reason to change the intended plan of approach, it is becoming clear what the result of the master’s thesis will consist of. As described very shortly in figure 1, the result will be a ‘guideline’ on stiffness parameters and modelling suggestions for timber-frame walls. For different wall families, or wall configurations, values will be given, which can be used as stiffness values in modelling. As explained in figure 1, the stiffness values may be based on code directives (K_{slip}). These code directives will be clariﬁed by comparison with experimental information. Aim is to ﬁnd a modelling approach that will be suitable for analysis with the software available in the company of Boorsma Consultants; it has to fit to the possibilities of common framework structural analysis software in 2D as well in 3D. Two different alternatives for modelling will be chosen, first a braced framework model, in which the brace stiffness can contain the information about fastener, sheeting and anchorage properties. Another (more advanced) type of model can consist of plate element(s) with certain shear modulus G. In this case, G can be seen as a stiffness parameter containing the information about fastener, sheeting and anchorage properties, while the plate geometry can be a model of openings in the wall.
literature review

Determine racking stiffness value for standard shear-walls from shear-wall test-data using NEN-EN 594

Determine stiffness value \( k_s \) for fasteners from test-data using NEN-EN 26891

Determine stiffness value for hold-down anchors from test-data using NEN-EN 594

Comparison of determined fastener stiffnes value \( k_s \) with Eurocode 5 approach for joint slip modulus \( K_{ser} \)

Comparison of determined anchorage stiffness with Eurocode 5 approach (using \( K_{ser} \))

Analytical derivation of racking stiffness value for shear walls from:
- determined fastener stiffness values
- determined anchorage stiffness values
- structural mechanics

Analytical derivation of stiffness parameters for modelling equivalent shear modulus \( G \) for modelling approach with equivalent plate

Modelling & Analysis perforated shear-walls using equivalent shear modulus \( G \) in a plate modelling approach

Comparison of perforated shear-wall modelling & analysis results with:
- Determined racking stiffness values from test-data
- Results of analytical methods (methods found in literature)

Determine racking stiffness value for perforated shear-walls from perforated shear-wall test-data using NEN-EN 594

Alternative: a stiffness parameter \( k \) for a braced framework modelling approach can also be derived analytical

Test-data of:
- fasteners
- hold-down anchorage
- shear-walls
- perforated shear-walls (with opening)

Eurocode 5:
- joint slip modulus \( K_{ser} \)

Evaluation: If match is proven, between racking-stiffness determined from tests on perforated walls, and racking stiffness derived by modeling perforated walls (using plate model & parameter \( G \), the results could consist of:
- Modeling suggestion: plate model with \( G \) and / or braced framework model with parameter \( k \) or \( k' \)
- Equivalent stiffness values \( G \), \( k \) and/or \( k' \): for a certain range of shearwall geometries and configurations (a certain sheeting material & fastener)

figure 1: Intended plan of approach for the remaining part of the project.
Comparison between stiffness value derived by modelling and analysis, and stiffness determined from test-data.

Figure 2: Illustration of the plan of approach.
2 Effects on structural behaviour as found in literature
Studying the literature it became clear that within the area of timber-frame construction many subjects can be distinguished. Several interesting subjects were discovered. Research into timber-frame structures is already done for many years. Next to this, it is obvious that research topics vary from less profound to very detailed studies. Studying and discussing every paper and piece of literature would not be useful within the scope of this project. Therefore, the following paragraphs will treat only the most important findings. These are results, relevant to the goals of the master’s thesis. A distinction is made between the effects playing a role in stiffness of timber-frame structures, and the experimental results that were found. In this chapter the several phenomena that are of influence on the stiffness are discussed, in the next chapter the experimental information is presented.
To present the findings in a structured way two different levels are distinguished. The effects and influences on stiffness of the structure will be considered on shear-wall level, and fastener and hold-down level. An overview of the next paragraphs is presented below.

2.1 Shear-wall level
- 2.1.1 Force distribution in the shear-wall sheathing
- 2.1.2 Influence of fasteners on the stiffness of shear-wall panels
- 2.1.3 Influence of hold-down characteristics on the stiffness of shear-wall panels
- 2.1.4 Influence of aspect ratio (h/b) on the stiffness of shear-wall panels
- 2.1.5 Influence of number of shear-wall panels on the stiffness of shear-wall diaphragms
- 2.1.6 Influence of asymmetric sheeting on the stiffness of shear-wall panels
- 2.1.7 Influence of openings (perforation) on the stiffness of shear-wall panels
- 2.1.8 Diaphragm action of floors

2.2 Fastener and hold-down level
- 2.2.1 Fasteners
- 2.2.2 Hold-down anchors

2.1 Shear-wall level
In this paragraph, some aspects will be presented which influence the shear-wall behaviour. The discussion of these aspects is based on what came across in the literature research. The findings are presented in a way of comparison and discussion, to prevent copying the literature. Therefore, always reference is made to the original authors of research (see the references section on page 46).

The timber-frame wall is a building element which can be part of the stabilizing structure. In such a case the timber-frame shear-wall has a load-bearing function, not only for the purpose of transferring vertical loads, but also for the transfer of the horizontal forces which act on the building. In case of wind for example, the horizontal wind-load is transferred to the timber-frame shear-wall by diaphragm action of the floors. The racking force on the shear-wall is divided over the wall length, this results in a distributed load on top of the wall. This distributed load will be transferred to the foundation or substructure by the shear-wall.

The shear-wall itself is a composite element. A timber-frame wall consists of timber frame elements, connections, wood-based or gypsum-based panels, and fasteners: screw, nail or staple. The shear-wall is able to transfer the racking forces thanks to the sheathing which acts as a brace to the timber framing elements. The shear-force on top of the wall-element is transferred to the sheathing by means of fasteners which are located along the perimeter of the sheathing.
2.1.1 Force distribution in the shear-wall sheeting

First of all, clarification is given on the force distribution in the timber-frame sheeting. In the analysis of timber-frame shear-walls, assumptions have to be made to facilitate the calculations. These assumptions also include the force distribution in the sheeting. Several approaches can be followed to approximate the force-distribution in the sheeting. These approaches will be shown below. For the discussion of the different methods, use is made of the information provided by (Källsner & Lam, 1995) and (Källsner & Girhammar, 2009).

In figure 4 (a), a force distribution according to a linear elastic model is shown. In a linear elastic model, the sheeting-to-timber joints have linear elastic load-slip characteristics up to failure. The joint stiffness is independent of force direction and grain orientation of the sheeting and timber framing elements. As can be seen from the figure, the load on the fasteners is of different magnitude for each position. From comparison with
Experimental testing can be concluded that the elastic model generally will lead to an underestimation of the shear-wall capacity. Nevertheless, the horizontal displacement of the wall calculated according to the linear elastic model agreed very well with the results of full-scale shear-wall tests.

In figure 4 (c) a plastic force distribution is shown, based on the plastic upper bound method. Each of the five timber members rotates around its own centre of rotation. Sheeting-to-timber joints are completely plastic, the properties are independent of the force direction and grain orientation of the sheet and timber members. The loads on the fasteners are of the same magnitude, because the fasteners have plastic characteristics. By experimental testing, the use of this method turned out to result in an overestimation of the racking capacity of the shear wall.

In figure 4 (b), a force distribution is shown according to a lower bound plastic model. In this model, a pure shear flow along the perimeter of the wall is assumed, on the condition of force and moment equilibrium. The force on each fastener is at most equal to the plastic capacity of the fastener. The framing members, the timber framing elements, are considered to be completely flexible. Therefore, the force distribution will become parallel with the framing members. This in contrast with models mentioned in (a) and (c). Model (b) turned out to correspond very well with experimental testing, giving values for the racking capacity lower than, or equal to the exact value obtained by testing. Besides this, the method is suitable for, and very useful to, hand calculation. This may have been the reason to include the plastic lower bound method in Eurocode 5. The method is based on a static theorem, while the upper bound method (c) was based on a kinematic theorem. In a kinematic theorem, a geometrically possible pattern of deformation is assumed. By means of the principle of virtual work, the internal work of all the fasteners is made equal to the work of the external forces. Therefore, the centre of rotation (CR) of the timber frame elements is shown in figure 4 (c).

As a consequence of the fastener forces on the sheet, a force distribution in the sheet will develop as shown in the figure below. The force distribution in the sheeting will be visible when the buckling mode of the timber-frame shear-wall is observed. In case of buckling, the compressive stress in the sheeting was too high, and caused the sheeting to deform out of plane. Due to the remaining tensile forces in the diagonal, the sheeting can maintain its function partly.

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**Figure 5:** Development of tension and compression in sheeting diagonals.

**Figure 6:** Example of buckling in case of a thin plate / sheet, without intermediate stud. Test set-up for experimental verification. (Kessel & Sandauw-Wietfeldt, 2004)
2.1.2 Influence of fasteners on the stiffness of shear-wall panels

From the paragraph before became clear how the fasteners along the edge of the panel provide for the transfer of a distributed load from the timber framing elements to the sheeting. Without fasteners and sheeting the frame would be an unstable mechanism. Consequently, the stiffness of the wall depends very heavily on the stiffness of the fasteners. The disproportionate influence of the fasteners was already considered in 1985. According to William McCutcheon, the racking behavior of a wood shear wall depends primarily upon the strength and stiffness of the fasteners that attach the sheeting to the frame. It is possible to predict the performance of the wall, if the load-displacement characteristics of the individual nail is known. The load-slip properties of the fasteners can be determined by individual nail tests. (McCutcheon, 1985) Similar considerations can be found in publications of (He et al., 2001) and (Girhammar et al., 2004). The latter published a study on the characteristics of sheeting-to-timber joints in timber-frame shear walls. From the numerical work of (Conte et al., 2011) a part of the report is mentioned below:

‘For the horizontal stability of wood framed buildings, diaphragm action in the walls is of crucial importance. The structural behavior and capacity of wall diaphragms are primarily dependent on the sheathing-to-timber joints. (…) Considering the presence of a lateral load $F$ at the top of the shear walls, the timber frame must be regarded as unstable, and unable to counteract external force, which is only resisted by the sheathing and metal fasteners. Apart from the possibility to uplift of the shear wall, the total lateral displacement is function of fastener slip and shear distortion of the sheathing (which is usually relatively small and often neglected). Also the global lateral strength capacity of shear diaphragm is often determined by the resistance of the local joint between timber frame and sheathing panel (…).’ (Conte et al., 2011)

In the figure below, the separate contributions to the horizontal deformation of a shear-wall element made of two shear-wall panels are compared. The racking deformation of a timber-frame shear-wall consists of: (a) slip of the fasteners, (b) strain in the hold-down anchorage, (c) shear deformation of the sheeting, and (d) slip between the bottom rail and substructure. The remaining issues are considered to be insignificant.

Parameters, which can be varied to influence the effect of the fasteners on the shear-wall behaviour, are:

- choice of fastener type: screw, nail, staple,
- choice of fastener properties: diameter $Ø$, and length
- spacing of the fasteners (s) along the edges of a panel, being the distance between individual fasteners
The fastener properties themselves are influenced by the thickness and density of the board material, and the density of the framing timber. What choices can be made to influence the fastener characteristics and what the influence is of these choices will be explained in §2.2 (page 23).

### 2.1.3 Influence of hold-down characteristics on the stiffness of shear-wall panels

A wall panel will most often look like, as drawn in figure 8. When a wall panel is subjected to an in-plane racking force ($F_{i,v,Ed}$), the fasteners will distort, and the toprail of the shear-wall will displace horizontally. If no vertical load is applied, a tensile force ($F_{i,t,Ed}$) will develop in the tensile stud. Uplift of this stud needs to be prevented by the hold-down anchorage.

When doing a racking test, the application of sufficient amount of anchorage, can be equivalent with applying a vertical load. It is also possible to do racking tests by loading the panel with a diagonal load. These three options are all equivalent if no uplift occurs. In figure 9 is shown what type of force distribution will occur in case of sufficient anchorage, and what type of force distribution will develop when no hold-down or, insufficient anchorage is applied. These differences have an enormous influence on the performance of the wall, and on the utilization of material.

As can be seen from the right hand side of figure 9, the tensile forces caused by the horizontal force ($F_{i,v,Ed}$) have to be transferred through the plate to the fasteners in the bottom rail, because of the lack of tie-down anchorage or vertical load. Due to this disadvantageous force distribution, the fasteners in the bottom rail are loaded perpendicular to the edge of the panel, and perpendicular to the grain of the bottom rail. This will cause a very bad failure mode, which is reason for brittle behaviour of the wall panel.

Often no-hold down is applied in the test set-up. Sometimes the argumentation is that it is not common practice to provide the hold-down anchorage, or, from the viewpoint of costs, an engineered solution is not preferred because hold-downs are very costly. Except for the cases where the vertical permanent load can prevent tensile forces to develop, it may be clear that effective utilization of materials only can be reached when structures are equipped with hold-down devices. (See figure 10 & figure 11 on the next page.)
The effect described above is also found in several tests. Among others, Dolan and Heine did experimental testing to get more insight into the effect of hold-down anchorage on shear-wall behaviour. They conclude the following. Failure modes for walls with no overturning restraint were obviously different. For all three walls the typical failure mode was the tension end stud separating from the bottom rail and the sheeting unzipping from the bottom rail (Dolan & Heine, 1997).

**Insufficient hold-down capacity magnifies other design choices**

As could be concluded from the paragraphs above, the prevention of uplift is also very important for other design choices. Walls provided with openings, like windows and doors, are much more prone to the presence or absence of hold-down anchorage. In these cases, the effect and influence of anchors on the stiffness of the wall is relatively higher than for standard non-perforated shear-wall diaphragms. See also §2.1.7 where the influence of openings is elucidated.

Prevention of uplift is very important when load-sharing between different panel sides is considered, in asymmetric timber-frame shear-wall panels, with gypsum and OSB sheeting for instance. The anchorage condition influences the failure mode of asymmetric sheeted shear-wall assemblies. In this case providing enough hold-down anchorage is required to be able to add the contribution of the gypsum sheet in the panel stiffness to the contribution of the OSB sheeting. In §2.1.6 this will be explained more in detail.

*figure 10: Unzipping of sheeting from the bottom rail (Ni et al., 2010)*

*figure 11: Lack of anchorage / vertical load causing the fasteners to tear through the sheeting edge. (Salenikovich, 2000)*
2.1.4 Influence of aspect ratio \((h/b)\) on the stiffness of shear-wall panels

A common design of shear walls for timber-frame buildings also consists of space for doors and windows. Architectural reasons often lead to slender walls. Take for instance the wish for flexibility in plan. In such a case, minimal wall length is required. Large spaces are kept free for having big windows over the full floor height. Due to these openings, shear wall diaphragms consist of shear wall panels with different widths. In the literature review, research was done regarding the effects from height-to-width ratio on stiffness.

From tests on walls with a length in the range of 0.6 until 3.6 m in plan of the building, was found, that walls of 1.2 m and longer tend to develop the same stiffness. Walls 0.6 m long had 50% lower stiffness relative to the long walls. Due to the low racking stiffness, the sheeting connections in these walls did not fail, which lead to the extraordinary ductility (Salenikovich, 2000). The large drifts arise due to a greater effect of body rotation, the uplift displacements of the end studs contribute to horizontal deflections in proportion to the aspect ratio (Salenikovich & Dolan, 2003). In STEP 3 this effect is described is following. A wall panel with a small width to height ratio will reach its maximum load at a rather large displacement. A wall panel with a large width to height ratio will reach its maximum load at a lower displacement. Therefore the full plastic capacity of the wall with small width to height ratio might never be reached. (Källsner & Lam, 1995)

Lower values for the stiffness of walls with a smaller width can be explained by comparing a slender wall with a standard wall. Let the walls be loaded with the same shear load:

As can be seen from figure 13, the walls are executed in the same way. The walls are loaded to the same extend; an equal shear force is developing on the edges of the panel. \(\delta_v\) is the total vertical deformation due to anchorage slip, and lengthening / shortening due to tension and compression in the leading studs. Because of the small width of the slender wall (4:1), the deformation \(\delta_v\) is magnified more than for the wall with ratio 2:1.
The influence of the aspect ratio was also considered in an analytical calculation method (Källsner & Girhammar, 2009). Based on the conditions of full anchorage a plastic model was derived in which: $h/b$ is the height-to-width ratio of the wall, $s_r$ is the fastener spacing of the sheet-to-timber connections, $b$ is the width of the wall panel, $H$ is the horizontal racking load, and $k$ is the stiffness of the fastener sheet-to-timber connections. The stiffness of a shear wall panel can be computed by:

$$k_{\text{frame}} = \frac{k \cdot b}{2(1 + \frac{h}{b}) s_r}$$

(1)

When this relation is plotted for different height-to-width ratios, the decreasing stiffness for higher ratios is evident.

From literature, and simple analysis can be concluded that for walls with slenderness ratio higher than or equal to 4:1, the stiffness properties of the standard wall (2:1) have to be reduced with 50%. Walls with aspect ratios ≤ 2:1 developed identical load-deformation patterns (Salenikovich & Dolan, 2003). On first hand, this 2:1 ‘limit’ might be strange. An explanation, found in literature, will be given in the next paragraph.

### 2.1.5 Influence of number of shear-wall panels on the stiffness of shear-wall diaphragms

In paragraph 2.1.4, the increase in stiffness for ratios $h/b$ lower than 2 was shown. For common design, the stiffness obtained on $h/b = 2$ is the maximum stiffness which can be reached. This is related with the maximum available dimensions of sheeting. Because of the standard dimensions in the manufacturing of the sheet materials, widths of the shear-wall panels up to 1,2 m are possible. Often the height is approximately 2,4 m, which lead to a ratio $h/b = 2$. A shear wall diaphragm consists of multiple shear-wall panels when the length exceeds the 1,2 m limit. The stiffness of the wall diaphragm will not increase by doing this because the sheets rotate separate when the wall diaphragm is loaded. See also the figures below. Evidence is also found in experimental testing. Test results revealed that the performance of the shear wall did not depend on the number of shear-wall panels, where a panel is defined as the elementary timber-frame shear-wall element, with a width of one single sheeting, usually equal to 1,2 m.
2.1.6 Influence of asymmetric sheeting on the stiffness of shear-wall panels

The timber-frame walls, which are often used in a cavity wall application, can be executed with sheeting at one or both sides. Often wall panels are made with two types of sheets. For instance: Gypsum (GKB) sheeting at the interior of the building, and OSB sheeting at the cavity side of the wall. In literature, information is found about the contribution of both sheet materials in the total wall behaviour. Insufficient hold-down anchorage was found to give reason for different results. Eventually a conclusion could be drawn from the publications.

For the described walls, the individual load-displacement curves can be superimposed when sufficient tie-down capacity is provided by hold-down anchorage and / or vertical load. In this way, an equally distributed shearforce will develop along the edges of the shear wall panel. (See also §2.1.2) Another requirement is that the different material / fastener combinations that are used have to show enough deformation capacity until the level of shear-wall diaphragm utilization.

Research performed by Wolfe in 1983 revealed that the law of superposition explained the contribution of Gypsum to the ultimate shear capacity of OSB/Gypsum sheeted shear-walls. Wolfe tested 30 walls to study the contribution of GKB to the racking resistance (Sinha & Gupta, 2009). A first comment to the conclusion of Wolfe could be made after Karacabeyli and Ceccotti did a similar research in 1996. They verified that the law of superposition was valid up to a drift of approximately 50 mm for monotonic tests (Ceccotti & Karacabeyli, 2002).

When these findings are studied further, it appears that the principle of superposition also holds for the stiffness of the walls. The deformation of both sheets will be the same because they are fastened to the same frame. The system can be seen as a parallel set of springs:

\[ F_{\text{total}} = F_1 + F_2 = (k_1 \cdot u) + (k_2 \cdot u) = (k_1 + k_2) \cdot u \]  

(2)

\[ k_{\text{combined}} = k_1 + k_2 \]  

(3)
The possibility of adding the capacity and load-displacement behaviour of gypsum sheeted panel side to the capacity and load-displacement behaviour of the plywood side of the shear-wall panel was also demonstrated by Johnson in 1997. He concluded that GKB will help to resist shear at moderate load level, but plywood resists most of the shear at higher load levels (Sinha & Gupta, 2009).

Comparison of load-displacement curves for wall panels, consisting of normally used combinations of interior and exterior sheeting materials in Sweden, showed that the maximum load is obtained at about the same displacement. Considering the ultimate limit state, the following conclusions were drawn. If a wall panel is built with sheets of different types or thickness on both sides, a reduction of the weaker side has to be calculated with. Based on experimental data a reduction of the load carrying capacity of the weaker side should be in the range of 20%. Only if fastener sheet combinations with completely different slip properties were used, a reduction of 50% is justified (Källsner & Lam, 1995). The same is written in the Eurocode 5, where a reduction of 25% is prescribed. (See §4.2.) These findings can be explained by studying figure 17 more detailed. When ultimate racking load is reached for the combined action, the gypsum part of the wall is already over its ultimate capacity, therefore it is logical to reduce its attribution in the assembly of gypsum and OSB when considering ultimate strength limits.

Not only the load-bearing capacity of the weakest side needs to be reduced when considering combined action, also the deformation capacity of the wall is reduced. This was already stated by Karacabeyli and Ceccotti in 1996, and confirmed from research performed in 2003. Uang and Gatto observed a 12% increase in shear wall strength, and 60% increase in initial stiffness, but also, a decrease in shear-wall deformation capacity of 31%. Independently, Toothman found comparable results (Sinha & Gupta, 2009). Explanation of the effect is, that the material, which originally showed the largest deformation capacity, is subjected to a higher load in the combined situation, than in the situation of single performance. This can be seen from figure 17.

Sinha and Gupta performed an unusual study to get better insight in the combined action and effects of combined action. Using digital optical imaging techniques, they were able to record the strains on the surface of the shear-wall panel sheeting. 16 Full-scale timber-frame shear-walls were tested, 11 were sheeted on both sides, with OSB on one side and GKB on the other. Five walls were tested without GKB. From the tests they derived the following conclusions. Upon failure of the GKB sheeting, the GKB carries a load four times higher than the OSB sheeting because of the higher initial stiffness of the gypsum sheeting. However, when GKB starts to fail, the strain in OSB will increase on a higher rate. Hence, OSB is taking over the load from the gypsum sheeting. This continues up to complete failure of the GWB sheeting. On first sight, the research of Sinha and Gupta showed different results than the previous mentioned tests. The elastic shear stiffness is increased by 50%. However, the contribution of GKB to the strength of the wall is very low, namely, 0.8%. See also figure 19.

Sinha and Gupta explained the difference between these results and the test results of Karacabeyli and Ceccotti, and Toothman, by the variation in the size of walls and different fasteners used for attaching OSB and GKB to the shear-wall panel framing (Sinha & Gupta, 2009). However, are the different fastener characteristics really the explanation for the difference in test results?
It seems very logical to attribute the brittle behaviour of the gypsum sheeting material to the sheeting-to-timber fastener behaviour. However, when a wall is sufficiently tied down, a shear force parallel to the framing members will develop along the edge. From the experimental information (§3.2) can be concluded that a dry-wall screw Ø2,31x41,3 will not show brittle behaviour. The shift from load taken by the gypsum panel to the OSB panel should develop more gradually. The correct explanation for the low contribution of gypsum in the ultimate strength is the vertical uplift of the stud. The hold-down capacity of the anchorage provisions was too low. This results in a force on the fasteners perpendicular to the edge of the gypsum panel in the lower corner near the hold-down anchor. Fasteners tearing through the gypsum board are the reason for the brittle behaviour. (See also §2.1.2.)

Conclusion
As could be seen in the present discussion concerning load-sharing between gypsum and OSB in a shear-wall assembly, three conclusions can be drawn.

- Load is shared by both OSB and GKB initially in a shear wall assembly. When the gypsum sheet is starting to fail, the load will shift to the OSB panel.
- GKB will contribute to the overall strength of the shear wall, and it will increases the stiffness of the wall, provided sufficient tie-down capacity or vertical load is applied.
- For OSB also Plywood or another similar wood-based panel can be read.
2.1.7 Influence of openings (perforation) on the stiffness of shear-wall panels

In this paragraph the influence of openings (perforations) on the racking stiffness of timber-frame shear-walls will be discussed. Several reports and papers on this topic were found in literature. An abstract of these will be presented below. Reference is made to §3.2, where test-data is presented regarding timber-frame shear-walls with opening.

Reduction of stiffness

Doudak and Smith tested seven different configurations of shear-wall panels. Only geometry and boundary conditions were changed. The fastener and sheeting configuration remained the same for all panels. Wall lay-out: OSB $t = 11,1$ mm $1200 \times 2400$ mm, overall wall dimension: $b \times h = 2400 \times 2400$ mm, studs spaced 400 mm center to center, fastener: nail $\phi ??x60$, fastener spacing $s = 150$ mm

Vertical load was applied on top of the walls respectively $4,16$ kN/m on wall 2 and $2,08$ kN/m on walls 4, 6 and 7. No significant uplift was observed. A certain initial stiffness was calculated from the test-data. These stiffness values are stated in table 1. From the tests can be seen that perforations can cause a reduction in the stiffness of the wall-diaphragms. Despite the fact that tie-down anchorage is applied in the inside bottom corners and in the door openings, adding a door (4) to the standard wall (2) is the reason for a reduction of 44% in stiffness. The door dimensions were $1938 \times 838$ mm which is 35% of the wall width. An explanation for the magnification of the effect, could be the reduced panel width. The standard wall is sheeted with OSB. The width of these sheets, $1200$ mm, will decrease to $781$ mm because of the door. This is a reduction in width of the wall of 35%. In §2.1.4 was explained that this will lead to a relatively higher reduction in stiffness (44%).

<table>
<thead>
<tr>
<th>Wall test</th>
<th>Initial stiffness (N/mm)</th>
<th>Ultimate racking load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>558</td>
<td>20,8</td>
</tr>
<tr>
<td>3</td>
<td>313</td>
<td>9,3</td>
</tr>
<tr>
<td>4</td>
<td>380</td>
<td>11,2</td>
</tr>
<tr>
<td>5</td>
<td>416</td>
<td>10,9</td>
</tr>
<tr>
<td>6</td>
<td>562</td>
<td>12,1</td>
</tr>
<tr>
<td>7</td>
<td>603</td>
<td>15,3</td>
</tr>
</tbody>
</table>

Table 1: Calculated values for initial stiffness of tested shear wall panels with perforations. (Doudak & Smith, 2009)

On first sight the table reveals a strange effect, when adding a window to the standard wall, the initial stiffness of the wall increases. This effect could happen because of the method to calculate the initial stiffness. ‘Because the response of the walls is nonlinear, the initial stiffness is taken as a secant stiffness defined using racking loads and associated racking deformations at 0,1 and 0,4 times the ultimate racking load. This is intended to characterize the response in the recoverable deformation range.’ (Doudak & Smith, 2009) Because the ultimate racking load decrease, the curve length over which the initial stiffness is calculated is much smaller, this results coincidentally in a higher initial stiffness. Another remark stated in the paper is the sensitivity of the wall behaviour for small
changes in boundary conditions. Take for instance the difference between wall 6 and 7. Because the tie-down anchor in wall 7 was mounted to the outside edge of the tensile stud, instead of the inside bottom corner of the wall, the stiffness increased with 7.3%. According to Doudak and Smith, the wall perforations have a disproportionate influence on the stiffness of the wall panels when hold-down anchors are omitted. In such a case, the wall stiffness of wall 2 was reduced with 56% by adding a door (wall 3).

**Sugiyama and Yasumura empirical equation**

In 2006, Dujic, Klobcar and Zarnic reported about research done by Sugiyama and Yasumura in 1980. (Dujic et al., 2006) Sugiyama and Yasumura conducted experimental tests on timber-frame shear-walls sheeted with plywood. These were not only full-scale tests but also reduced scale tests (1:3). Based on the results of their research they developed a method to reduce the racking stiffness and ultimate racking capacity of timber-frame shear-walls.

When the racking stiffness and ultimate racking capacity of a certain wall without perforations is known, the racking properties of the same wall with perforations can be calculated using the formulas developed by Sugiyama and Yasumura. In the Sugiyama and Yasumura method, the ‘panel area ratio’ \( r \) is calculated. Using this ratio the stiffness and strength of a perforated shear-wall can be determined as following:

\[
K' = \frac{r}{3 - 2r} \cdot K
\]  

and:

\[
F' = \frac{r}{3 - 2r} \cdot F
\]

In which:
- \( K' \) is the reduced racking stiffness of a perforated shear-wall
- \( K \) is the racking stiffness of the non-perforated shear-wall
- \( F' \) is the ultimate racking capacity of the perforated shear-wall
- \( F \) is the ultimate racking capacity of the non-perforated shear-wall

And:

\[
r = \frac{1}{1 + \frac{\alpha}{\beta}} = \frac{H \cdot \sum L_i}{H \cdot \sum L_i + \sum A_i}
\]

In which:
- \( r \) is panel area ratio
- \( h \) is height of the wall element
- \( L \) is length of the wall element
- \( \sum L_i \) length of full height wall segments
- \( \sum A_i \) sum of openings
- \( \alpha = \frac{\sum A_i}{HL} \) is ratio of openings in wall element
- \( \beta = \frac{\sum L_i}{L} \) is ratio of full wall segments

Results of testing done by Dolan and Johnson in 1996 showed the applicability of the empirical relation determined by Sugiyama and Yasumura. The results also indicated that the ‘panel area ratio’ method is slightly conservative especially at smaller ratio of openings (Dujic et al., 2006). Sugiyama and Yasumura developed the empirical equation provided hold-down anchorage is only applied on the end studs of the wall. No additional tie-down anchorage at intermediate locations of individual shearwall segments was provided.

According to the results of another series of tests performed by the American Plywood Association in 1994, the empirical equation was also proven to provide correct or slightly conservative values. The difference between the equation and test results does not exceed 10% (Douglas & Sugiyama, 2001).

Douglas and Sugiyama stated that the method is applicable to use in practice. The statements below were especially introduced to receive wider acceptance of the procedure in the traditional US design methodology:
• Shearwall segments are considered to be part of the total full-height length, if the length of the segment is equal to, or larger than, one-half of the wall height.
• Shear capacities of the sheeting materials have been limited to a maximum ultimate shear capacity of 1500 plf. (Equal to 21.89 kN/m.)
• Hold-downs with the capacity to resist the maximum overturning in a wall segment are required at each corner and at all discontinuities that occur in the wall line.

From research of Dolan and Heine can be concluded that the amount of tie-down anchors influences the load-displacement characteristics and failure mode. The magnitude of the influence of these anchors depends on the amount of openings in the wall. If there is a higher amount of perforation, every additional tied-down anchor on a stud near to an opening will be more effective than in a situation with lower amount of perforation. The study also reveals an interesting issue with respect to the curve determined by Yasumura and Sugiyama (figure 23). The amount of conservatism of this relation increases as the amount of opening in the wall increase. This can also be stated as following. The lower the panel area ratio, the larger the underestimation by the Yasumura and Sugiyama curve of the stiffness of the perforated element. (Dolan & Heine, 1997)

**Conclusion**

To conclude with, the most important remarks that can be made to the empirical equation are:

• only hold-down anchorage was applied on the edges of the wall diaphragm, no tie-down provisions on studs next to openings was applied
• tests which were the basis for the equation, and tests confirming the applicability of the equation, did not take in account the effects of vertical loading

An analysis will be made if the Sugiyama and Yasumura equations are applicable to the test-data as found in literature. This analysis will be done in a later phase of the master’s project.
2.1.8 Diaphragm action of floors

Horizontal forces caused by the wind on facade and roofing are transferred to the shear-walls by the horizontal diaphragms. These are the floor and roof planes of the building. How the horizontal diaphragms distribute the horizontal loading over the shear-walls is not only dependant of the stiffness of the shear-walls themselves, but can also be dependant of the stiffness of the horizontal diaphragm. This is explained with the figure below where the stiffness of the shear-walls themselves is not regarded.

![Figure 24: Differences in force distribution over the shear-walls in case of a flexible (A) and rigid (B) horizontal diaphragm. (Banga & Graaf, 2002)](image)

Hence, the in-plane stiffness of the floor and roof diaphragms is influencing the distribution of forces over the shear-walls. On the question how to determine if timber-frame floor diaphragms can be seen as either rigid or flexible, information is found in literature.

Horizontal diaphragms can only be considered as rigid, when the configuration of the diaphragm satisfy a number of requirements. One of the demands can be found in a publication of Kasal, Collins, Paevere, and Foliente from 2004. They state that the flexibility of the diaphragm has to be defined relative to the rest of the structure. The magnitude of expected deflection of the horizontal diaphragm relative to the shear-wall deformation can be used to characterize a diaphragm as rigid or flexible. A rule is stated in the International Building Code that a diaphragm is rigid when the deformation of the diaphragm is less than two times the average racking deformation of the shear-wall diaphragms. For most wood-frame buildings, the diaphragms can be considered to be rigid, due to the significant larger shear wall deformations compared with the floor-diaphragm deflection. (Kasal et al., 2004) Comments on the backgrounds of the IBC statements however are not present in the paper. More research into this topic would be justified.

A qualitative requirement would be that the sheeting panels are fastened to the framing along the entire perimeter. Therefore, the floor structure has to be provided with so-called blocking.

![Figure 25: Blocking timber applied between the main joists. (Falk & Itani, 1989)](image)

The effect of blocking was demonstrated in experimental testing performed by Bott in 2005, and numerical modelling done by Falk and Itani in 1989. If blocking is left away, a force distribution will develop different from what was already explained in §2.2.1 and drawn in figure 9 (page 10). Forces perpendicular to the sheeting edges and the phenomenon of buckling will result in unfavourable force distribution and failure mechanisms. Falk and Itani investigated the effect of blocking on the stiffness of the floor diaphragm by analyzing the diaphragm with blocking, and without blocking. From their numerical work could be concluded that leaving blocking from the floor structure could reduce the stiffness of the horizontal diaphragm with 35 to 49 % (Falk & Itani, 1989). The effect of blocking was also shown by experimental testing. Research was done on the effectiveness of measures.
to increase diaphragm stiffness by several structural choices. These were the application of blocking, the application of foam adhesive, the effect of openings in the sheeting (staircases), the influence of walls along the edge of the diaphragm, and the sheeting nail density. A limiting factor however, is the choice to study all issues separate. No combination of effects was studied. In conclusion can be said that the most effective measure was to provide the diaphragm with blocking. It provided the largest individual increase in diaphragm stiffness with an average of 135% increase in shear stiffness. Only qualitative results were published. These include the following conclusions.

Using a smaller fastener distance, applying sheeting connected to each other with glued tongue-and-groove finished edge, or gluing the perimeter of the sheeting to the floor joists, can increase the stiffness of the diaphragm enormous.

Perimeter walls can effectively stiffen a diaphragm if the bottom plate is continuous and adequately fastened-down. Due to the resistance of the bottom plate to the tension forces developing in the diaphragm edge through bending action, the primary effect of walls on diaphragms is increased flexural stiffness. (Bott, 2005)

A few years ago an extensive research program (NEESWood) was conducted in the United States. In this project, full-scale buildings were tested to verify their behaviour when supposed to seismic loading. From these tests a lot of knowledge was gained. In addition, information about the stiffness of the floor-diaphragms was acquired. Christovasilis, Filiatrault and Wanitkorkul reported about the research on a full-scale two-storey timber-frame building in 2007. The building is shown in figure 27.

Exposed to different magnitudes of seismic loads, the building became damaged. From observations and interpretation of this damage and the measurements done during testing was concluded that the horizontal diaphragms of the two main parts of the building acted as a rigid plane. However, because of the atrium and the staircase, the in-plane stiffness of the floor and roof at this location was lower than the rest of the structure. Although the application of gypsum sheeting to the ceiling increased significantly the in-plane stiffness of these diaphragms. Damage was observed in the gypsum sheeting, in the atrium / staircase are, after testing. The gypsum sheeting in the two main parts of the building had a minimal effect on the floor diaphragm since the structural floor system alone provided enough in-plane stiffness to act as a rigid diaphragm. (Christovasilis et al., 2007)
During the tests, the deformations were recorded with multiple sensors. The shapes of deformation could be composed from these measurements. These shapes show the rigid behaviour of the diaphragm in the main parts of the building, and the flexible part in the atrium / staircase area.

Based on the findings from literature as presented above the following conclusion can be drawn. For the dimensional proportions of the diaphragms treated above, the assumption of rigid diaphragm behaviour can be justified. The assumption should be critically treated for more slender floor plan proportions, and floor diaphragms with perforation such as atrium and staircases. For rigid diaphragms, modelling can consist of rigid bracing, or plate elements. Flexible floor diaphragms can be modelled as if shear-wall diaphragms would be modelled.
2.2 Fastener and hold-down level

In this paragraph shortly will be discussed what the code design rules on determination of the fastener characteristics consist of. Moreover, the intended approach to determine the characteristics of hold-down anchorage will be explained.

2.2.1 Fasteners

As will be presented in chapter 3, test-data of fasteners are result of the literature review. In a later phase of the masters’ project, these data will be used to determine stiffness values according to NEN-EN 26891. The determined stiffness values will then be compared with stiffness values derived from Eurocode 5. This process will result in more insight into the applicability of the Eurocode 5 rules for the fastener slip-modulus ($K_{ser}$) to timber-frame shear-wall analysis.

Calculation of $K_{ser}$ according to Eurocode 5

In NEN-EN 1995 equations to determine the fastener-slip-modulus $K_{ser}$ are given for several fastener types. Dowels, bolts, screws, nails, and staples are included in the design rules. The equations identify the timber mean density $\rho_m$ and fastener diameter $d$ as parameters influencing the stiffness of the connection. Timber-to-timber connections and timber-to-wood-based panel connections are aimed at. Regarding the steel-to-timber connections, only the density of the timber member is involved in the equation, and $K_{ser}$ may be multiplied by 2.

In Eurocode 5 gypsum-to-timber connections are not mentioned. About this type of connection, some information was found in the German code DIN 1052 (2008-12, table G.1). There it is stated that the fastener-slip modulus $K_{ser}$ has to be reduced with 40% for connections of timber to gypsum paperboard panels. In the German code, the characteristic timber density $\rho_k$ needs to be used instead of $\rho_m$.

An example for the equations given can be the formula to calculate the slip-modulus for nail-to-timber connections with nail fasteners:

$$K_{ser} = \frac{\rho_m^{1.5} \cdot d^{0.8}}{30}$$

In which:

$$\rho_m = \sqrt{\rho_{m,1} \cdot \rho_{m,2}}$$

The applicability of the design rules on deriving $K_{ser}$ to timber-frame shear-wall analysis, will be studied by comparing the Eurocode derived stiffness values with the stiffness values determined using NEN-EN 26891 and the test-data from literature.

Determination of $K_{ser}$ ($k_s$) by experimental testing according to NEN-EN 26891

$k_s$ can also be determined through experimental testing. In Eurocode 5 reference is made to NEN-EN 26891 for calculation rules on how to do this. In the test standard, $K_{ser}$ is denoted as $k_s$. The test standard provides uniform rules for conditioning of the test specimens, loading procedure, measurement of the slip and load, format of the test report, and calculation of the slip modulus. $k_s$ is in fact a modified stiffness value between 10% and 40% of the estimated maximum load $F_{est}$ of the connection. Furthermore, equations are given to calculate the stiffness at higher load levels (60% and 80%).

Influence of moisture content on fastener stiffness $K_{ser}$

The stiffness of the sheathing-to-timber joint is dependent on the moisture content. However, the magnitude of influence is not known exactly. Analogous to the relation between the modulus of elasticity and the moisture content, a change of 1.5% per 1% change in moisture content could be a reasonable assumption. (Kuilen, 2008) Additionally, some experimental information can be found in publications of Nakajima. (Nakajima, 2001), (Nakajima & Okabe, 2004) Tests on plywood shear-walls under conditions with relative humidity (RH) of 65% and 90% revealed a 21% decrease in initial stiffness of the wall panel in the ‘wet’ (RH 90%) condition, compared with the wall panel in the ‘dry’ (RH 65%) condition.
Considering European spruce, the increase of the moisture content is about 12%, when changing the RH from 65% to 95% (Banga & Graaf, 2002). Assuming the plywood from Nakjima’s tests to behave the same, this will result in 2.1% change in initial stiffness per 1% change in moisture content.

According to NEN-EN 26891, 20°C and 65%RH is the standard climate condition for testing when comparison of joints under similar conditions is aimed at. In some of the test reports and papers, reference is made to these circumstances as boundary condition for the shown load-displacement figures. From other sources of load-displacement graphs no information about the relative humidity is known.

One might say that for shear-walls applied in Serviceclass 2 a reduction in stiffness has to be calculated with. And, if the situation in timber-frame structures is analyzed, it becomes clear that the circumstances inside a cavity wall (Serviceclass 2) are different from the atmosphere inside the building (Serviceclass 1). However, the differences observed in testing at 90%RH and 65%RH influence the stiffness of the sheeting-to-timber connections much more, than the smaller differences between the interior and cavity side of the shear-wall will do. The moisture content of the timber on the inside of the building and inside the cavity wall will be always between 10% and 20%. Additionally the stiffness values of the timber based panel applied on the inner side of the building (Serviceclass 1) would have to be increased compared with values determined from testing at 20°C and 65%RH atmosphere.

Therefore the conclusion can be drawn that both effects could counterbalance each other. Hence, no effect of moisture content will be considered in the next phases of the project.

**Conclusion**

In conclusion can be said that the factors influencing the fastener stiffness $K_{sef}$, of the fastener are the following:

- Density of the timber (taken into account in Eurocode 5)
- Diameter of the fastener (taken into account in Eurocode 5)
- Type of sheeting material
- Moisture content of the wood (not taken into account by Eurocode 5, will not be considered)
- Thickness of the sheeting material
  
  Although the thickness is important for the ultimate capacity (strength) of the connections, it is not taken into account in Eurocode 5. Confirmation of this can also be seen in test-data. Compare curve 6.2 and 9.1 in figure 45, and curve 2.1 and 2.2 in figure 46.

- Yield strength of the fastener (Plastic moment capacity)
  
  Although the yield-moment capacity of the fastener is influencing the ultimate capacity (strength) of the connections, the influence on the stiffness of timber-to-framing connection is negligible. (Leichti et al., 2006)

The influencing factors stated above will determine the fastener-slip characteristics and failure mechanism, as long as enough edge distance is applied. If the distance to the edge is too short, other effects will become
important. For example, the way in which the edge is finished can be of influence on the characteristics of the fastener (manufacturing issue). For gypsum-based panels this can be seen from the test-data found from literature. Differences between sawn edges and edges finished with a paper cover become visible in the test results when not enough edge distance is applied. Compare test results 7.3 and 7.4 in figure 47. The shortest edge of a standard gypsum paperboard is the sawn edge. No finishing is applied on this edge. The longest edge of a standard gypsum paperboard can be finished in different ways. The reinforcing paper will make this edge a little bit stronger than the short edge.

![Sawn edge on the left, finished edge on the right. (Knauf product information)](figure 32)
2.2.2 Hold-down anchors

For the determination of hold-down stiffness, no standard method is available. However, in this paragraph will be presented which parts of Eurocode 5 on connections and fasteners will be used when a method to derive hold-down stiffness will be explored in the later phases of the masters’ project. The method can be verified using the hold-down test-data presented in chapter 3 and NEN-EN 26891.

The design rules to derive $K_{ser}$ according to Eurocode 5 are already discussed above. The fastener-slip modulus will again be used to derive a stiffness value for the hold-down anchorage.

When the designer is calculating the design strength for hold-down anchorage, the calculation starts with the design strength for a single fastener $(F_{f,Rd})$. A number of $n$ fasteners are used. When calculating the combined action of all fasteners, the number of fasteners has to be reduced. The effective number of fasteners can be calculated as following:

$$n_{ef} = n^{k_{ef}}$$  \hspace{1cm} (9)

In which $k_{ef}$ is a reduction factor bringing in account the fastener spacing and the option of predrilled holes. The capacity of the hold-down can now be calculated to be:

$$F_{t,Rd} = n_{ef} \cdot F_{f,Rd}$$  \hspace{1cm} (10)

The reduction of the number of fasteners is introduced because of the splitting behaviour visible near failure of a parallel to grain positioned and loaded row of fasteners. The accumulation of perpendicular to the grain tensile stresses, gives reason to reduce the capacity of the connection (Jorissen, 1998). However, considering the serviceability limit state, this reduction is no issue because it is the stress-level in ultimate limit state causing the splitting of the timber grains. Therefore, the stiffness of the individual fastener does not have to be reduced when contributing to the total stiffness of the hold-down anchor.

$$K_{hold-down} = n \cdot K_{ser}$$  \hspace{1cm} (11)

This is only valid when the fasteners act as a set of parallel springs, which is the fact when the deformation of the steel section can be neglected. A comparison can be made with the stiffness value determined on basis of the test-data. For the standard anchorage solutions, manufactured by Rotho Blaas and Simpson, test-data were provided, and found in literature. Stiffness values for custom made configurations of hold-down anchors can be estimated when it is possible to derive the stiffness of hold-down anchorage solutions in the way as it is described above.
3 Experimental results

As was mentioned in §1.2, the test-data of timber-frame shear-walls, shear-walls with opening, sheeting-to-timber fasteners, and hold-down anchorage, will be presented. How the information will be used is already discussed in §1.2 and 1.3. The information will be the experimental basis to determine stiffness values for the wall assemblies and connections (§2.2 & 4.3). These stiffness values will be compared to the present code directives in a later phase of the project. Validation of the modelling approach with the experimental information will be another purpose of the test-data.

3.1 Full-scale shear-wall tests

From several sources, reports on experimental testing were found. In this section, tests on full-scale shear-walls are presented. The information found in literature is studied and individual curves are combined into one graph for wall widths 600, 1200, 1800, 2400, 3600 and 4800 mm. Comparison of one curve with another is made possible by displaying all relevant information next to the graphs. If it is not stated differently, the standard parameters are:

- Dimensions of the wall (b x h) as displayed in the title of the graph.
- Shear-wall sheeted on only one side of the shear-wall panel are considered to be standard. If both sides of the shear-wall are sheeted, or when different fastener or sheeting layouts are applied on both sides it will be explicitly mentioned.
- Walls were tested fully anchored, using hold-down provisions or applying a vertical load.

From a wide range of shear-wall configurations test results were found, this can be seen in table 2. References displayed in the table can be found on page 31 in the reference list.

<table>
<thead>
<tr>
<th>Wall dimensions (b x h): [mm]</th>
<th>600 x 2400</th>
<th>1200 x 2400</th>
<th>1800 x 2400</th>
<th>2400 x 2400</th>
<th>3600 x 2400</th>
<th>4800 x 2400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastener spacing (s): [mm]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OSB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(3.4)</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td></td>
<td></td>
<td>(1.3)</td>
<td>(1.2)</td>
<td>(7.2)</td>
<td>(8.3)</td>
</tr>
<tr>
<td>Gypsum paper board (GKB)</td>
<td></td>
<td>(10.1)</td>
<td>(10.3)</td>
<td>(10.4)</td>
<td>(14.2)</td>
<td>(14.9)</td>
</tr>
<tr>
<td>Gypsum fibre board</td>
<td>(11.1)</td>
<td>(12.1)</td>
<td>(11.3,5,7)</td>
<td>(12,2,3,4)</td>
<td>(11.4,6,8)</td>
<td></td>
</tr>
<tr>
<td>Particleboard</td>
<td></td>
<td></td>
<td></td>
<td>(9.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardboard fibre board</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(9.2)</td>
</tr>
<tr>
<td>MDF</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(14.17)</td>
</tr>
</tbody>
</table>

*table 2: Overview available test-data of shear-wall diaphragms as found in literature.*
The information in the legend of the graphs can be read as following: *Ply t = 9.5 nail Ø2.87 x 50, s = 100, b x h = 1820 x 2440, (1.3)* means that the tested panel was made using plywood sheeting with thickness 12.0 mm. The fastener type is stated, as well as the fastener spacing (s) in mm. Thereafter the dimensions of the shear-wall panel are stated (b x h). For reference (1.3) is added, in which 1 refer to the source in literature: reference 1, and 3 means that this is the third test in the publication referred to. Reference to literature is made on page 31.

**Figure 35:** Load displacement characteristics for timber-frame shear-wall panels 600 x 2400 (b x h).

**Figure 36:** Load displacement characteristics for timber-frame shear-wall panels 1200 x 2400 (b x h).
Figure 37: Load displacement characteristics for timber-frame shear-wall panels 1200 x 2400 (b x h).

Figure 38: Load displacement characteristics for timber-frame shear-wall panels 1800 x 2400 (b x h).
**Figure 39:** Load displacement characteristics for timber-frame shear-wall panels 2400 x 2400 (b x h).

**Figure 40:** Load displacement characteristics for timber-frame shear-wall panels 3600 x 2400 (b x h).
Load-displacement characteristics for timber-frame shear-wall panels 4800 x 2400 (b x h)

Figure 41: Load displacement characteristics for timber-frame shear-wall panels 4800 x 2400 (b x h).

References to literature

In the legend of the graphs, reference is made to the following numbers. These references can be found in the back of the report on page 46.

(1) (Yasumura & Kawai, 1997)
(2) (Andreasen, 2000)
(3) (Salenikovich, 2000)
(4) (Ni et al., 2010)
(5) (NAHB Research Center Inc., 2005)
(6) (Yasumura & Karacabeyli, 2007)
(7) (He et al., 2001)
(8) (Yasumura, 1991)
(9) (Vessby et al., 2008)
(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002)
(11) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 16-02-2000)
(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 04-10-2001)
(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 29-05-2002)
(14) (Källsner, 1984)
(15) (Richard et al., 2002)
3.2 Full-scale shear-wall tests with opening

From three sources, reports on experimental testing of perforated shear-walls were found. In this section, the results of these tests will be presented. Perforated shear-walls are shear-wall diaphragms, provided with openings (perforation). These can be either windows, doors, or other type of openings. Because the range of ultimate capacity and deformation capacity of the test series were different, the load-displacement graphs are presented according to their source. In this way, comparison of the individual curves is easier. All relevant parameters are presented in the legend, as well as the figures indicating the individual wall layout. Reference to literature is made in the subtitles, and below the third graph.

![Load-displacement characteristics for perforated timber-frame shear-walls](image1)

**Figure 42**: Load displacement characteristics for timber-frame shear-wall diaphragms with opening. (Källsner, 1984)

![Load-displacement characteristics for perforated timber-frame shear-walls](image2)

**Figure 43**: Load displacement characteristics for timber-frame shear-wall diaphragms with opening. (Dolan & Heine, 1997), (Dolan & Johnson, 1996)
References to literature

(14) (Källsner, 1984)
(15) (Richard et al., 2002)
(16) (Dolan & Johnson, 1996) & (Dolan & Heine, 1997)
3.3 Sheeting-to-timber fastener tests

From several sources, reports on experimental testing of fastener connections were found. These tests are often used to derive stiffness values to be implemented in structural analysis and modelling. In this section, the test-data on sheeting-to-timber fasteners are presented. The information found in literature is studied and individual curves are combined into graphs for the different sheeting materials. Comparison of one curve with another is made possible by displaying all relevant information next to the graphs. All curves display the behaviour for one single fastener.

From table 3 it can be concluded that from a wide range of fastener and sheeting combinations, test results were found. Reference displayed in the table can be found on page 34 in the reference list.

<table>
<thead>
<tr>
<th>Board material</th>
<th>Staple</th>
<th>Nail</th>
<th>Screw</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ø1,50 (1,53) x 50</td>
<td>Ø1,8 x 65</td>
<td>Ø1,34 / 1,62 x 60</td>
</tr>
<tr>
<td>OSB 9,5</td>
<td>[11.1] (1.2)</td>
<td>(6.3)</td>
<td>[6.1] [6.2]</td>
</tr>
<tr>
<td>OSB 11</td>
<td>(2.1)</td>
<td>[9] (1.4)</td>
<td>[3.6] [3.5]</td>
</tr>
<tr>
<td>Plywood 9</td>
<td>[1.3]</td>
<td>(4)</td>
<td>[1.6] [1.5]</td>
</tr>
<tr>
<td>Plywood 9.5</td>
<td>(2.2)</td>
<td>(4)</td>
<td>[1.6] [1.5]</td>
</tr>
<tr>
<td>Gypsum paper board (GKB) 12</td>
<td>(1.1)</td>
<td>(4)</td>
<td>[1.6] [1.5]</td>
</tr>
<tr>
<td>Particle board 10</td>
<td>(15)</td>
<td>(13.3)</td>
<td>(13.3)</td>
</tr>
<tr>
<td>Cement splinter board 16</td>
<td>(4.2)</td>
<td>(10.1)</td>
<td></td>
</tr>
<tr>
<td>Hardboard 8</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

table 3: Overview of available test-data for sheeting-to-timber fasteners.

References to literature:
(1) (Yasumura & Kawai, 1997)
(2) (Komatsu et al., 1999)
(3) (Dujic & Zarnic, 2003)
(4) (Karakabeyli & Ceccotti, 1996)
(5) (Christovasilis et al., 2007)
(6) (Richard et al., 2002)
(7) (Bouw- en woningtoezicht Rotterdam, 1989)
(8) (Andreasoss, 2000)
(9) (Salenikovich, 2000)
(10) (Vessby et al., 2008)
(11) (He et al., 2001)
(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)
(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 10-06-2002)
(14) (Conte et al., 2011)
(15) (Källsner, 1984)
Multi-storey timber frame building – literature review

**Figure 45:** Load-displacement characteristics for fastener tests with OSB sheeting.

**Figure 46:** Load-displacement characteristics for fastener tests with Plywood sheeting.
**Figure 47: Load-displacement characteristics for fastener tests with Gypsum-based sheeting.**

**Figure 48: Load-displacement characteristics for fastener tests with different types of sheeting.**
3.4 Hold-down anchorage tests

From two sources, reports on experimental testing of hold-down connections were found. The tests, which were (on request) provided by Rotho Blaas GmbH, were originally performed to receive European Technical Approval certificate for the hold-down solutions manufactured by Rotho Blaas. Another purpose is to receive characteristic values for the ultimate load capacity of the tie-down anchors. Although the test-data can be used to determine stiffness values for the hold-down, the stiffness aspects are not mentioned in product information. Another well-known manufacturer of hold-down solutions is Simpson Strong-Tie. Load-displacement information on Simpson’s HTT16 hold-down was found in literature.

The meaning of the information on the hold-down stiffness aspect, can be explained as following. Often hold-down anchorage is prescribed by engineers based on a ULS approach. These hold-downs can be bought from stock, or can be designed individually for each single project as metal straps with certain dimensions, fastener, and fastener spacing. The hold-down solutions can be very stiff, as can be seen from the curves. When the deformations of the hold-down contribute to the deflections of the shear-wall panel as a whole, there can be reason to include the hold-down stiffness in the modelling. In such a situation, Eurocode 5 can be used to calculate a stiffness value for the hold-down anchor. (As explained in §2.2.2.) However, verification of this approach is only possible by comparison with the stiffness values determined from the load-displacement graphs.

figure 49: Hold-down test-data from testing, mean curves.

figure 50: Impression of situation after failure. (NAHB Research Center, 2003)

figure 51: Hold-down test-data from testing, nail Ø4x40.

figure 52: Hold-down test-data from testing, nail Ø4x75.
figure 53: Hold-down test-data from testing, screw Ø5x50.

References to literature:

1. Rotho Blaas Rothofixing WHT 80/620 hold-down (Rotho Blaas GmbH Rothofixing, 07-04-2010)
2. Simpson Strong-Tie HTT16 hold-down (NAHB Research Center, 2003)
3.5 Compression perpendicular to grain tests

In the literature review, also information came across about compression perpendicular to the grain. In standard walls, the vertical studs distribute the vertical forces to the foundation through the horizontal rails. Subsequently, a compressive stress perpendicular to grain will develop in the horizontal rail, resulting in additional horizontal deflections. The information that was found is displayed below, but will not studied further. In common Dutch practice deformations due to compression perpendicular to grain are prevented by increasing the cross section of the compressive stud. The contact area will increase, therefore the stress perpendicular to grain will decrease and deformation is prevented.

![Figure 54: When vertical forces are becoming higher, multiple vertical studs can be used, to reduce compression perpendicular to grain.](image)

![Figure 55: Load-displacement test results from several compression perpendicular to grain tests.](image)

![Figure 56: Compression perpendicular to grain deformation. (NAHB Research Center, 2003)](image)

References to literature:
(1) (Andreasson, 2000) (also information available concerning tension & shear in framing member connections)
(2) (Vessby et al., 2008) (also information available concerning tension & shear in framing member connections)
4 Issues concerning the modelling approach and code directives

In this part of the report, four topics on racking strength and stiffness aspects will be discussed. First, the requirements to a modelling suggestion will be explained. As stated in chapter 1, the modelling suggestion shall be treated more in detail, in a later phase of the project. Secondly, a modelling approach found in literature will be presented. Next, the analytical method according to Eurocode 5, for verification of strengths in SLS and ULS, will be treated. The final issue that will be discussed is the test method NEN-EN 594, about the experimental determination of racking strength and stiffness of timber-frame shear-walls.

4.1 Modelling aspects

Possibilities for modelling

A suggestion for a modelling approach to support the structural analysis, should fit to the possibilities of the software available within the company of Boorsma Consultants. As described in chapter 1, the goal of the project is to derive or determine the required linear-elastic stiffness properties, which are needed to model the timber-frame structure in a common used structural analysis framework program (like Technosoft Raamwerken). No plate or shell elements will be used, but cross bracing will contain the equivalent stiffness parameter to represent plane elements in 2D.

Source for the stiffness values, which will be used as input in the framework modelling, can be either, the Eurocode 5 design rules, test-data, a combination of both, or could be derived using more detailed modelling. To perform more detailed modelling, a software package will be used, more advanced than the common framework structural analysis software. This will be a commercially available FEM structural analysis program with user-friendly interface (Scia Engineer). No academic methods will be used because in these methods the possibilities are too extensive (Ansys / Diana). This choice will limit the possibilities of the FE method to the most used options. However, 3D analysis, plate and shell elements, and non-linear spring elements are among the possibilities.

![Image of intended thesis result: modelling a timber-frame structure using experimental input. Figure composed from several sources: Rothofixing WHT hold-down device picture & graph (Rothoblaas GmbH), photo test set-up (NAHB Research Center Inc., 2005), graph test result (Salenikovich, 2000).](image)

Modelling approach as found in literature

As is made clear in the paragraph above, and shown in figure 57, an idea what the modelling approach will consist of, already exists. The feasibility of this type of approach was shown when studying literature. Different approaches, but also similar ways of modelling were found.
Pang and Rosowsky developed a modelling approach to take part in a benchmark study concerning the NEESWood project. (Some other findings from this project are discussed in §2.1.8) In the picture below the physical existing shear-wall and floor diaphragms are drawn in grey colour. The representation of the shear-walls in longitudinal direction (x-axis) is omitted for clarity. As can be seen, the diaphragms are represented by spring elements, which are connected rigidly to, master and slave, nodes. The springs contain all the stiffness information needed to get a sufficient accurate model. In fact, the model only consists of a set of equations and programming, because it is a mathematical numerical model.

![Numerical modelling approach. (Pang & Rosowsky, 2010)](image)

Pang and Rosowsky applied the principle of sub-modelling to get correct stiffness values as input for the spring elements. Therefore, more detailed modelling was done on shear-wall level:

![Detailed FEM modelling of a timber-frame shear-wall. (Pang & Rosowsky, 2010)](image)

The horizontal floor diaphragm in figure 58 was modelled using the so-called two-node-beam element. Therefore, first an analysis was performed using a detailed FEM approach. The information on stiffness of the diaphragms could thereafter be implemented in the two-node-beam element approach. See figure 60, and figure 61.
Using different methods of modelling for different levels of scale is a usual approach. Dujic and Zarnic followed this modelling procedure as well. In a detailed FEM model, the fasteners and hold-down anchorage were modelled using non-linear springs. The framing elements were represented by the normal dimensions, and were connected with plastic hinges. The sheeting and glulam beams were modelled using linear elastic panel elements.

Using this type of modelling also perforated shear-walls can be analysed. The results of the detailed modelling can thereafter be used in the less complex models using equivalent bracing.
The simplified bracing approach can be used to construct a larger 3D model of the whole building.

How the modelling aspect of the master’s thesis will exactly be dealt with will be determined in a later phase of the project. The information stated in this paragraph only presented the findings from the literature research.
4.2 Eurocode 5 methods for strength verification

In Eurocode 5 (NEN-EN 1995) can be chosen from two analytical methods to calculate shear-wall diaphragms. The methods are denoted as Method A and Method B. In this paragraph, Method A will be highlighted because this is the recommended method. The Eurocode approach can be used to derive the design value for the strength of a wall loaded in plane \( F_{v,Rd} \). The method is based on the design strength of a single fastener \( F_{f,Rd} \). This design strength can be calculated using the Johansen’s equations which are given in the code. The strength of a single fastener can be used to calculate the design strength of a single wall panel \( F_{w,Rd} \), taking into account the width \( b_1 \) of the panel, and fastener spacing \( s \) and panel \( h/b \) ratio. (For a drawing of a standard shear-wall panel, see figure 8 on page 10.)

\[
F_{v,Rd} = \sum F_{l,v,Rd} \tag{12}
\]

\[
F_{w,Rd} = \frac{F_{f,Rd} b_1 c_i}{s} \tag{13}
\]

In \( c_i \), the height-to-width ratio \( h/b \) is included. The fastener strength may be increased with a factor 1,2 because of system-action. A reduction of the weakest side has to be calculated with when different sheeting is applied on both sides of the wall panel, this reduction is 50% or 75% (§2.1.6). Perforated panels with windows or door openings do not contribute to the strength of the shear-wall assembly (§2.1.7). Additionally is stated that the external tensile and compression forces on the edges of the panel need to be taken up by the supporting structure. Sufficient hold-down capacity has to be provided using tie-down anchorage or vertical load. Vertical load however is not considered in the Eurocode method. No rules or guidelines are given on stiffness aspects of wall and floor diaphragms. In practice, the limit state approach will be limited to strength considerations only. This is one of the reasons to perform this master’s study (See §1.1).

4.3 NEN-EN 594 Racking test method

The racking strength and stiffness of timber-frame wall panels can also be determined with experimental testing. Different monotonic and cyclic testing standards are available around the world. These tests can be different not only because of the boundary conditions (fixing the panel to the test rig), but also the procedure of applying the load is often different. Although the tests may be performed according to different standards, for purpose of calculating stiffness values from the test-data (§3.1 & §3.2) use will be made of NEN-EN 594.

Experimental tests of timber-frame shear-wall panels are often assigned to research institutes by manufacturers of sheeting materials such as: Oriented Strand Board, Plywood, Particleboard, Medium Density Fibreboard, Gypsum Paper Board (GKB) or Gypsum Fibre Board. The manufacturers use the tests to gain European Technical Approval (ETA) to sell their products for structural purposes on the European market. Strength values taken up in the product information are often based on these ETA’s. Although the tests are done to acquire characteristic strength values in most cases, the load-displacement graphs can also be used to derive stiffness values. This information however, is almost never given in the product information. From literature and the archive of Boorsma Consultants, load-displacement graphs were found for a number of shear-wall configurations. Using NEN-EN 594, stiffness values can be determined from this test-data.

According to NEN-EN 594 the racking stiffness of the panel can be calculated from the following equation:

\[
R = \frac{F_4 - F_2}{v_4 - v_2} \tag{14}
\]

Where:
- \( F_2 \) is the racking load of 0,2 \( F_{\text{max}} \)
- \( F_4 \) is the racking load of 0,4 \( F_{\text{max}} \)
- \( v_2 \) and \( v_4 \) is the deformation on these load levels
\( F_{\text{max}} \) is reached when either the panel collapses, or the panel attains a deformation \( v \) of 100 mm, whichever occurs first.

*figure 65: Test procedure. (NEN-EN 594)*
5 References


Rotho Blaas GmbH, n.d. *Information on Rothoblaas WHT angle bracket hold-down device*. Generously provided by Ing. Fabio Verber from Rotho Blaas GmbH.


