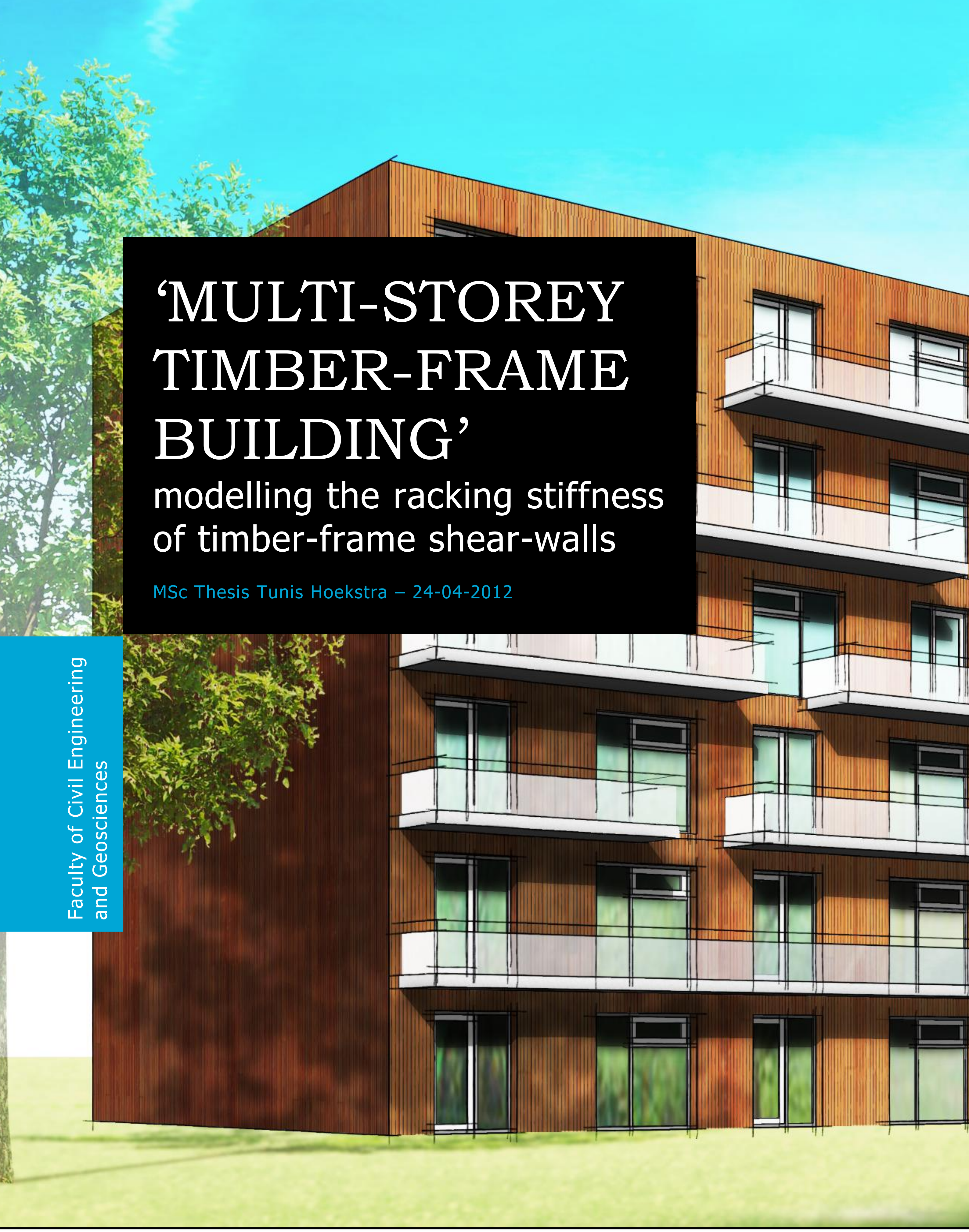

‘MULTI-STOREY TIMBER-FRAME BUILDING’

modelling the racking stiffness of timber-frame shear-walls

MSc Thesis Tunis Hoekstra – 24-04-2012

Faculty of Civil Engineering and Geosciences

TU Delft Delft University of Technology

ingenieursbureau boorsma b.v.

General information:

This document is the final report of the MSc thesis 'Multi-storey timber-frame building, in which the racking stiffness of timber-frame shear-walls is analyzed. The thesis is written by Tunis Hoekstra as a final proof before receiving his Master of Science degree in Civil Engineering at the faculty of Civil Engineering and Geosciences at TUDelft.

Additionally to this document, a report was made containing the results of a literature and knowledge review.

The cover displays a concept for a six-storey timber-frame apartment building. The image is used with credits to Han van Zwieten Architecten.

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Preface and acknowledgements

Today, timber structures are receiving attention more and more. Because of the increasing interest in sustainable construction as well as for other reasons, building industry in Europe is (re)-discovering multi-storey timber structures for construction of mid-rise buildings.

In this master's thesis, an analytical calculation method and modelling approach are presented, to calculate the timber-frame shear-wall racking stiffness. With the research I did, a specific gap in engineers knowledge is completed, and a contribution is made to the development of multi-storey timber-frame structures in the Dutch context of building engineering.

This thesis' report is written as a final proof before receiving the Master of Science (MSc) degree in Civil Engineering at TUDelft. After finishing the bachelor part of the study and making the choice for the master track of building engineering, I became specialized in structural design, in which the building structure is focused on. The study is completed with this graduation thesis.

Although the research is done to proof the academic capabilities of the student on first hand, its value to engineering practice is highly influenced by the generous support of Ingenieursbureau Boorsma B.V. This engineering firm provided me the tools and services I need to do the work in an efficient manner. I like to thank my supervisor Ir. Jan Banga for the clear opinions, judgment, and support he gave in the many conversations we have had.

Finally, I like to thank the people not mentioned above. Thanks for your contribution in providing test-data, giving permission for the use of images, and sharing your opinion with me. Last but not least I like to mention the patience of my girlfriend. Study and graduation took up a lot of company. Thank you Marie.

Drachten, April 2012

Tunis Hoekstra

Summary

The calculation of the timber-frame shear-wall racking stiffness is an issue that is currently not dealt with in the European standard for structural design of timber structures: Eurocode 5. Although important for the verification of deformations and force-distribution within the timber-frame structures, the calculation of the timber-frame shear-wall racking stiffness can be regarded as a gap in engineer's knowledge. At least in the Dutch situation. With use of the strength-related code directives, a lot of experience in timber-frame construction is gained in the Netherlands already. However, this did not result in the realisation of multi-storey timber-frame buildings exceeding five storeys yet. Because of the growing interest for sustainable timber construction, among clients such as housing associations, it might be expected that multi-storey timber-frame buildings exceeding five storeys will become realised, as can be seen from a number of examples abroad. Before this reality can take place, prediction of deformation and force distribution of the timber-frame structure have to be possible. Therefore, a calculation method and modelling approach for the calculation of timber-frame shear-wall racking stiffness are derived in this master's thesis.

In the literature and knowledge review a lot of information was found from literature. Most important issues with respect to the stiffness of the timber-frame element became clear. Additionally to this, a large amount of test-results was found. These test-results appeared to be very useful for verification of the proposed calculation and modelling methods.

In the analysis part of the master's thesis first the slip of the fasteners was studied. From the literature review was known that the fasteners are the most important parts in the timber-frame element, determining the racking stiffness of the shear-wall. Making a comparison between the Eurocode 5 design rules, and the results determined from load-displacement test-data, it became clear that the code-based fastener-slip modulus K_{ser} agrees very well with the slip-modulus determined from test-data. The equations for K_{ser} given in Eurocode 5 can be used for the calculation of the timber-frame shear-wall stiffness.

A second issue studied in the analysis part of this master's thesis was the stiffness of the hold-down anchorage. From the literature review became clear that sufficient hold-down capacity is required to guarantee the development of the intended force-distribution in the timber-frame shear-wall panel. It turned out that the hold-down anchorage stiffness can be calculated taking into account the number of fasteners, the stiffness of the steel and timber cross-section, and the possibly weakening effect of hole-clearance.

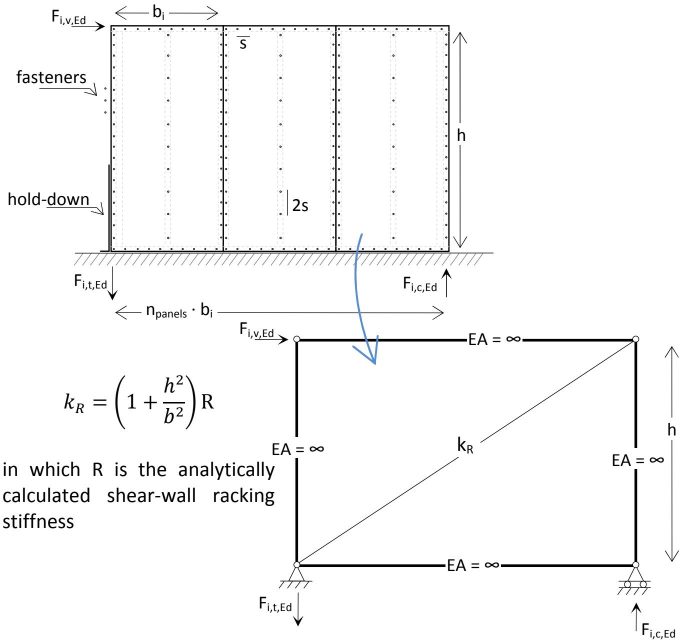
In the third part of this master's thesis an analytical calculation method was derived. The most important parameters that determine the timber-frame shear-wall racking stiffness are taken into account in this calculation method. The calculation can be made by hand, which was aimed at. In addition to the analytical calculation method, a modelling approach was suggested. This modelling approach is suitable for use with generally available software for structural analysis and makes use of the analytically calculated racking stiffness parameter. From the analysis became clear that the following components in the timber-frame shear-wall racking stiffness have to be taken into account:

- slip of fasteners along the panel perimeter : 48%
- shear of the sheathing board material : 12%
- strain in leading & trailing stud : 8%
- strain in the hold-down anchorage : 13%
- compression perpendicular to grain in the bottom rail : 15%

The average contribution (%) of these components to the total deformation could be calculated from the analytical calculation that was made using the geometry of 31 tests. From comparison with the test-based determined racking stiffness values, it turned out that the proposed analytical calculation method gives useful results. The analytically calculated value for the racking stiffness, for which use is made of the fastener-slip-

modulus K_{ser}, can be used as a value for the calculation of the timber-frame shear-wall racking stiffness and deformation in the serviceability limit state (SLS).

The timber-frame shear-wall racking stiffness determined with use of the analytical calculation method can be used in the modelling approach, which was developed in the next part of the master's thesis. A truss-type model was proven suitable, for use in a framework program for structural analysis. A calculation and modelling example is given which showed excellent agreement between calculated and test-based determined racking stiffness. In the suggested modelling approach, a truss-type model is made consisting of hinge-connected elements of infinite stiffness. The timber-frame shear-wall racking stiffness is inherited by the axial stiffness of a diagonal brace.



$$k_R = \left(1 + \frac{h^2}{b^2}\right) \mathrm{R}$$

in which R is the analytically calculated shear-wall racking stiffness

figure 1: Model for the structural analysis of the timber-frame shear-wall in a framework program.

In the last part of the master's thesis attention was given to the analysis and modelling of perforated timber-frame shear-walls. These are shear-walls provided with an opening such as a window or door. Two methods for modelling were derived, to calculate the racking stiffness of the perforated shear-walls. In the literature review load-displacement data were of tests on perforated shear-walls were found. The test-based determined racking stiffness values were used for verification with the results of analytical calculation and modelling. In the modelling of the perforated timber-frame elements it was chosen to divide the shear-wall element in a number of panels, depending on the geometry of the window. For each of these panels a diagonal brace was added, which resulted in a truss model with multiple braces. The stiffness of these diagonal braces was determined with use of the analytical calculation method. Two different methods of employing the analytical calculation method were derived. Both methods, the multi-panel method as well as the equivalent-brace method, showed reasonable agreement with the test-based results.

Keywords

racking, stiffness, timber, frame, shear, wall, modelling, calculation, analytical, method, structural, framework, fastener, hold-down, slip, modulus, K_{ser}, truss, model, brace, perforated, sheathing, panel, board

Table of contents

Literature and knowledge review

The literature review (23-04-2012) is the first part of the MSc thesis ‘Multi-storey timber-frame building’. In this report load-displacement data from several sources in literature were brought together. Findings about the most important parts in the timber-frame shear-wall, and the characteristics of the behaviour, were reported as well.

1 Introduction

In this chapter an introduction is given into the topic of multi-storey timber-frame construction. First the load-bearing system of buildings is explained, after which the timber-frame method, and cross-laminated construction type are explained. Furthermore, some projects in the Netherlands and abroad, are shown to illustrate the state-of-the art in multi-storey timber-frame building. Next to this the benefits of timber structures to building engineering are explained, also in relation with the current state of affairs in building industry. These benefits show a bright perspective for multi-storey timber-frame buildings. The chapter will be finished by introducing the subject of this master's thesis, and objectives of the research. In the last paragraph an outline of this report will be shown.

1.1 Typical timber construction methods

In this paragraph the timber building structure will be explained. How does the load-bearing structure of a building look like, and which construction types are used in multi-storey timber buildings?

1.1.1 Load-bearing system of buildings

A building structure will be loaded by vertical forces because of the dead weight and imposed loads. Next to this there are the horizontal forces coming from wind. Horizontal forces can also be present in case of a seismic event. However, the presence of earthquakes is a local variable, and in Dutch situation no seismic design is required from code-perspective. Horizontal forces because of wind are a result of the wind-pressure on the facade. The building facade will bring the wind-forces into the floor and roof. By diaphragm action of these horizontal elements, the wind-forces are transferred to the shear-walls in the structure. These shear-walls transfer the resulting forces to the foundation. See figure 2.

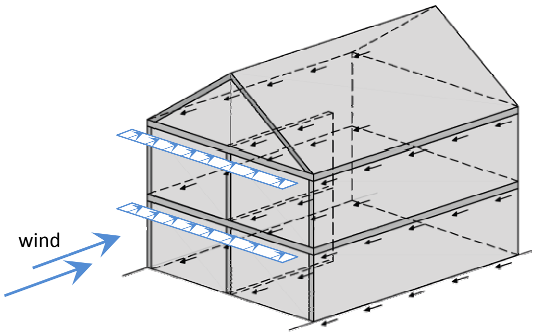


figure 2: Building loaded by wind on the facade, giving a load on the floor-diaphragm (Banga & Graaf, 2012).

Diaphragm action

An important requirement to the performance of floors is that they have to fulfil the diaphragm function. Therefore, next to their load-bearing function, the floors need to have in-plane stiffness. Depending on the ratio between diaphragm stiffness of the floor and racking-stiffness of the shear-wall, two possible load-distributions are shown in figure 3.

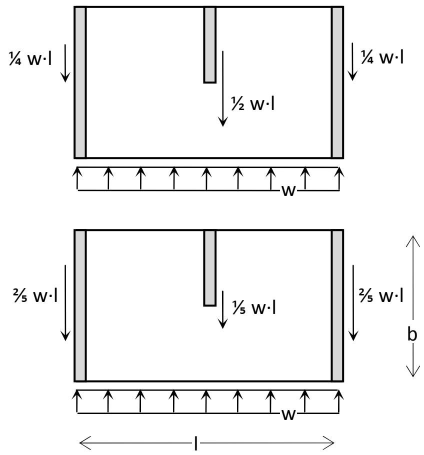


figure 3: Different distribution of load over the shear-walls (Banga & Graaf, 2012).

Shear-walls

The shear-walls are subjected to a shear-force. This shear force will result in a sliding, tilting, and racking mechanism on the shear-wall. See figure 4. The shear-walls will deform under the action of wind loading. This deformation may not exceed the limit values, otherwise the user will be confronted with diminished serviceability. To estimate the deformations of the structure, the racking-stiffness of the shear-wall has to be known. Determining the shear-wall stiffness, especially the timber-frame shear-wall racking-stiffness will be of special interest in this masters' thesis.

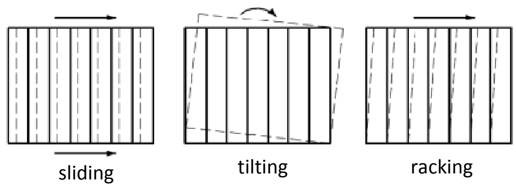


figure 4: Sliding, tilting, and racking of shear-walls (Banga & Graaf, 2012).

1.1.2 Structural lay-out of a timber-frame building

In general the load-bearing system of a building will be like explained in the paragraph before. Aspects that make buildings different from each other are the specific building type, and construction method. In this introduction two different types of timber building-structure will be shown. First, standard timber-frame construction is explained extensively. The racking-stiffness of the timber-frame shear-wall element is subject of research in this masters' thesis. Next to this, limited attention will be paid to the cross-laminated type of building structures, this method will not be subject of the thesis.

Platform-type structure

Timber-frame buildings are most often built according to the platform method. After the concrete foundation and ground floor are made, the sill-plate will be mounted. This sill-plate guarantees that the wall is placed in levelled position. If the walls are made according to the geometrical and dimensional tolerances, the first floor can be positioned directly on top of the walls. In this way a level platform is created for the erection of the walls on the first floor. In this way a structure can be stacked storey on storey, see figure 5.

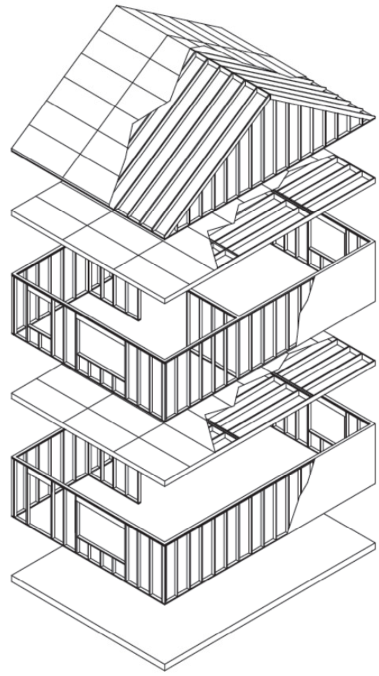


figure 5: Platform-type of timber-frame structure (Banga & Graaf, 2012).

Differential movement

In figure 7 and figure 8, the layout of the timber-frame wall- and floor-element is shown. In the platform method, the floor-elements are protruding beyond the load-bearing walls. Shrinkage can be caused by release of moisture after construction. Shrinkage will be highest, perpendicular to the grain of the wood fibre. The shrinkage is concentrated in the joists of the floors, head-binder, sill plate, and top- and bottom rail of the load-bearing walls. This deformation can increase up to a 5 - 10 mm uniform shortening per storey.

figure 6: Solution in the structural detail between platform- and balloon type (Bouwwereld, 2009).

Balloon- and mixed-type

Next to the platform-method there is the balloon-type of timber-frame construction. The balloon-type structures are characterized by load-bearing walls being extended to the full building height. The floors do not interfere in the wall structure, but are suspended between the load-bearing walls. By means of a steel angle for example. For multi-storey timber-frame building a solution to differential shrinkage is made, which in fact is a mix of the platform- and balloon-type of timber-frame structure. See figure 6. The load-bearing walls can protrude beyond the floors, while the floors still can be stacked on top of the walls. In this solution differential shrinkage is minimised.

Timber-frame shear-wall layout

The shear-walls have a double function. First of all, the walls can have a load-bearing and stabilizing function. Secondly walls have a separating function between spaces. To fulfil the requirements of these different functions, the wall-elements consist of a number of materials. The wall is made by a frame of timber studs and rails. The timber is most often taken from a European or North-American source of sawn softwood. An insulating material is placed between the framing elements. The timber-frame wall is finished with a board material. This board material can have a structural as well as a fire-safety or acoustic function. Most often the material on the inner face of the wall will be made of a mineral-type of board. This can be a gypsum-paper, or gypsum-fibre board. The board-material on the cavity face of the shear-wall can be sheathed with a wood-based panel. This can be plywood, particleboard, or oriented strands board (OSB) for example. To control the moisture content in the structure, vapour barriers are added to the element by means of vapour tight foil on the interior face, and a watertight membrane (breather-membrane) on the exterior face of the wall. See figure 7. The correct detailing of the vapour-control and insulation layers is very important. Both will have a big influence on the user comfort, interior climate, energy performance of the design, and durability of the structure. Bad detailing can cause serious problems. If the moisture content in the wall cross-section becomes too high, the durability of the timber structure can be affected by fungi.

Timber-frame floor-layout

In figure 8 the typical layout of the timber-frame floor element is shown. The floor will transfer its own weight, and the load of people and furniture, to the load-bearing walls. For reasons of fire protection and sound-insulation, a gypsum-board ceiling is attached to the floor joists by means of a resilient metal rail. This mass-spring-mass system, together with the insulation material between the ceiling and floor, can effectively absorb the sound. For floors that separate different dwellings from each other, more severe requirements will lead to a different layout. Experience proved that well-performing solutions are available to provide an acoustic separation between the dwellings in an apartment building for instance.

Prefabrication of timber-frame elements

A very high level of prefabrication can be reached with use of the timber-frame building structure. The facade elements can be made in the factory, including cladding, doors, windows, and window-frames. This offers a lot of advantages such as high quality level, better quality control, and, quick and clean construction. Prefabrication of timber-frame elements is standard business in the Netherlands for years already. In North America and Canada however, the tradition of carpentry on the building site still can be seen.

1.1.3 Cross-laminated timber

Alternatively to the timber-frame method, a timber building can also be constructed with use of cross-laminated timber. This material is becoming increasingly admired among architects and clients, and is made in massive plates measuring up to 24 x 3 meter. Timber laths are glued together cross-wise, to form the panel. There are a number of manufacturers supplying the product. Plates are available with different thickness and order of the layers. Multi-storey timber buildings up to nine storeys completely in timber are already made with use of this material. These projects however will not be treated in this introduction, the focus of this report is on the timber-frame method. In figure 4 sliding, tilting, and racking mechanism of shear-walls is shown. These effects are also present in CLT shear-walls. Deformation due to sliding and tilting need to be limited by sufficient stiff

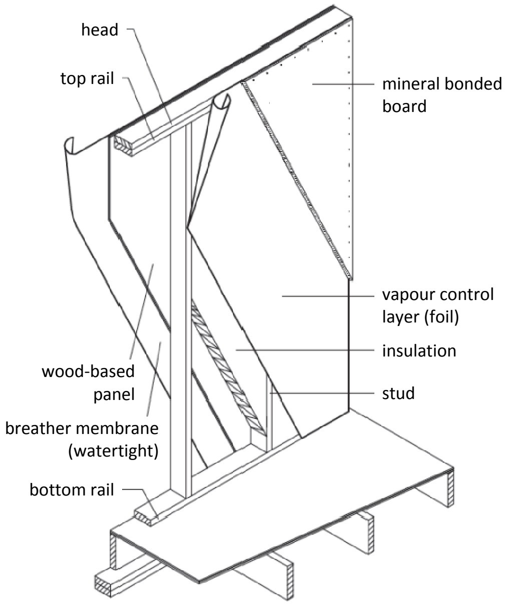


figure 7: Layout of a typical timber-frame wall-element (Banga & Graaf, 2012).

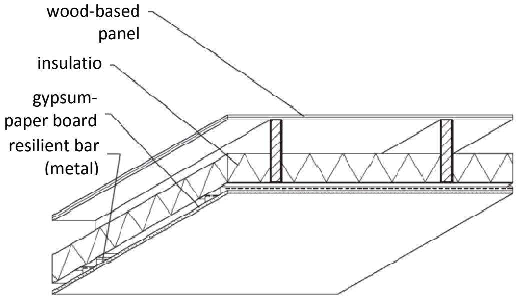


figure 8: Layout of a typical timber-frame floor-element (Banga & Graaf, 2012).

figure 9: Dutch way of timber-frame construction: prefabricated facade elements (Centrum Hout Almere, 2000).

figure 10: North-American way of timber-frame construction: stick-built on site.

connections with the substructure. The racking mechanism in timber-frame shear-walls consist of two contributions: rotation of the sheathing because of slip of the fasteners, and shear deformation of the sheathing material. In cross-laminated timber, the racking mechanism merely consists of shear deformation of the panel.

figure 11: Construction using cross-laminated timber: sawn plate elements, structure to be finished at the building site.

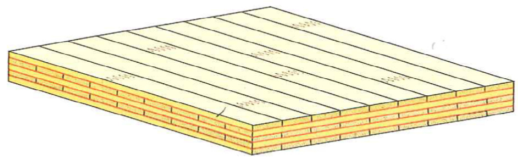


figure 12: Lay-out of a CLT-panel

1.2 Dutch situation

On the cover of this report an illustration can be seen of the concept "Stapelen met Houtskeletbouw" This concept is made by a Dutch architect, a company in timber-frame buildings, and an engineer of Ingenieursbureau Boorsma. The concept is made in relation to the increasing interest for the timber-frame method by clients such as housing associations. In this paragraph the development and state-of-the-art of the timber-frame method in the Netherlands will be presented.

History

For a long time, Dutch building industry was used to apply only mineral materials in the load-bearing structure of the building. Masonry, calcium-silicate blocks, and concrete were the materials most often used for purpose of load-bearing walls in buildings. Timber was only used in the structure of roofs. Although the timber-frame method was introduced simultaneously with calcium-silicate (kalkzandsteen) already round about 1950, calcium-silicate became the dominant construction material (Mahapatra & Gustavsson, 2009). Real-estate developing agents and contractors did not prefer timber as a building material. Because of negative experiences with respect to acoustics, floor vibrations, and fire safety, timber dwellings were not popular at all (Jorissen & Leijten, 2008). It was common opinion that timber buildings would not be able to fulfil the requirement of the users, while construction methods using concrete floors and walls of calcium-silicate performed well. Loss of knowledge and experience were result of these developments, consequently the Dutch people still are less used to the phenomenon of timber buildings (Cobouw, 2009).

Modern timber-frame construction

In the seventies, the Canadian alternative for timber-frame construction has been introduced in the Netherlands. This method of modular structures and standardised dimensions was aimed to be suitable for the dense populated Dutch situation. Round about 1980 some favourable developments took place. In 1984 the "Vereniging van Houtskelet Bouwers" (VHSB) was initiated. This association aims at sharing knowledge and experience, development of standards and regulations, stimulating research, and generating public attention for the timber-frame construction and timber-frame buildings. Together with Stichting Keuringsbureau Hout (SKH), all VHSB-members were equipped with product- and process certification. The timber-frame method was evaluated, and fulfilled all requirements and conditions of "Het Bouwbesluit" (The Dutch building decree). The promotional organisation "Centrum Hout" promotes the use of timber to the public with use of websites, excursions, symposia, social media, and other activities. In the project "Meerlaagse houtskeletbouw" (multi-storey timber-frame building), scientific research has been done, and a guide and magazines were issued. This cooperative effort resulted in still growing interest among users and clients for the timber-frame construction method (Mahapatra & Gustavsson, 2009).

Realised projects in the Netherlands

The developments presented in the paragraph before resulted in a number of cases in which the timber-frame method has been applied successfully. Especially in renovation and low-rise residential buildings (dwellings) the timber-frame method is an often-used construction type. Multi-storey timber-frame buildings however, are very rare in the Netherlands, and buildings exceeding five storeys, have not been build yet. One of the earliest examples of a larger residential building still is the highest timber-frame building in the Netherlands. The building is situated in Delft, and became ready in 1992. Today it is called the “Ecodus building” (figure 13). In plan the building has the shape of six hexagonal figures. Four of these hexagonals consist of five levels, of which ground floor as well as the four storeys on top are stabilised completely by the timber-frame structure. Only a few steel columns are used to transfer heavy vertical loads.

figure 13: “Ecodus” in Delft, The Netherlands, 1992

figure 14: Three-storey residential building in Aalsmeer, The Netherlands, 2000 (Centrum Hout Almere, 2000).

In 2009 in Buren a four-storey project was build, which is known as “Lage Korn” (figure 15). The design of the residential building is a realisation of the so-called “Maskerade®” concept, designed by architect Maarten van der Breggen. The structural detail shown in figure 6 was applied in this project. Maarten van der Breggen is also the architect of the timber-frame project in Aalsmeer that became ready in 2000. See figure 14. Although this is a building with three storeys, Centrum Hout denoted it to be a demonstration project for multi-storey timber frame building (Centrum Hout Almere, 2000).

figure 15: “Lage Korn” Three-storey residential and utility building in Buren, The Netherlands, 2009 (Breggen, 2009).

In 2007 in Almere the “Malmö Hus” became ready. This is a project of four storeys. The floor construction is a typical example of a timber-frame floor. The walls however are made of cross-laminated timber (Bouwwereld, 2009).

VDM is a company in the Netherlands active in timber-frame building and development. The company is characterised by the integration of all activities into one chain. Design, construction, as well as sales, are done by the company, often in cooperation with housing associations. In 2011 “Koninginnehof” in Zuidwolde was realised by VDM. See figure 16. Part of the project is a three-storey residential building. Together with Han van Zwieten architects and Ingenieursbureau Boorsma, VDM developed a concept for a six-storey timber-frame building. An artist’s impression of this concept is used on the cover of this report. In figure 17 the plan is shown. In the concept the

figure 16: Three-storey timber-frame residential building in Zuidwolde, The Netherlands, 2011.

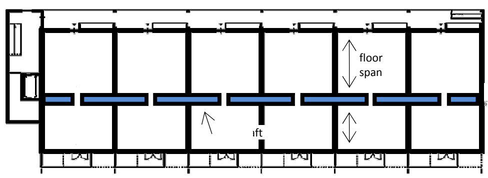


figure 17: Concept plan for six-storey timber-frame building, 2010.

floors span from the facade to the wall-shafts. "Koninginnehof" in Zuidwolde can be seen as a three-storey realisation of the concept.

figure 18: Residential building at Geert Grootestraat in Zwolle, The Netherlands, 2011.

In figure 18 a residential building is shown which became finished recently. It is located at the Geert Grootestraat in Zwolle. Although the main load-bearing structure was made in steel, the facade, walls and floors are timber-frame elements. This building of six storeys can be seen as an example of the perspective for multi-storey timber-frame building. The quick construction of the building is remarkable. In April 2011 the people moved out of their outdated apartments, in December 2011 they could re-enter the new apartments that are constructed at the foundations of the former building. (Bouwwereld, 2012)

1.3 Situation abroad

In the paragraph before the Dutch state-of-the art was presented. Much more experience with multi-storey timber-frame building is present outside the Netherlands. In North America and Canada a lot of buildings are made. These buildings typically comprise five storeys of timber-frame structure on top of a basement and ground-floor build in concrete. Reference is made to Kevin Cheung who describes a number of buildings (Cheung, 2010). Remarkable is also the amount of multi-storey timber buildings constructed in Scandinavia, and particularly Sweden. Publications and information about these projects often is written in the national language, and was not easy accessible. The website of the Swedish timber building council, Sveriges Träbyggnadskansli, can be browsed for reference to these projects. In this paragraph two research-projects will be shown as well as four commercially realised multi-storey timber-frame residences, which can be found in English language.

figure 19: "Neeswood Capstone" building at E-defence in Miki, Japan, 2009.

In figure 19 the Neeswood Capstone building is shown. This six-storey building was made in 2009 at the E-defence seismic test facility in Miki (Japan). This full-scale seismic test was part of a research program in which also smaller buildings, shear-wall elements, and sheathing-to-timber fasteners are tested. Goal of the research program was to make multi-storey timber-frame construction possible in regions with high seismic intensity. Contribution of experts and researchers from all over the world took place. Engineers were invited to predict the behaviour of the building during the test. The test made it possible to validate the different methods for analysis and modelling developed in different areas around the world (Follesa et

figure 20: "TF2000" research building at BRE Airship Hangar in Cardington, United Kingdom, 1999.

al., 2010). It was shown that the six-storey building could withstand the most severe earthquakes without failure. Information and load-displacement data of experiments performed in the initial phase of the project are used in this master's thesis.

Another interesting research project is shown in figure 20. This is the TF2000 research carried out in the United Kingdom from 1997 until 1999. In the BRE Airship Hangar a number of buildings were build at full scale. In addition, a timber-frame building was made, and subjected to tests. Useful knowledge was gained about the stiffness of the building by dynamic testing. Furthermore, insight was gained concerning fire-safety of the building and resistance to progressive collapse (Ellis & Bougard, 2001).

One of the commercial projects that was realised as a result of the knowledge gained in the TF2000 project, is shown in figure 21. "The Reflection Building" is realised on a spot with unfavourable soil conditions. A lightweight timber building was the only way to make the project feasible. The residential building measures six storey's and was commissioned by the housing association "St. James Homes" is located in North Woolwich, close to London, and became finished in 2002 (THFC, 2003).

In figure 22 the residential building "Casa Montarina" is shown. This building became finished in 2007, and measures five storeys. The project is located in Lugano (Switzerland). The timber-frame structure is made with OSB and gypsum-paper-board sheathing which is a common choice in timber-frame construction. A remarkable issue with respect to this building is the more slender shape. This can have to do with national building regulations. For example: regulation in the Netherlands is given with respect to daylight entrance in the building. The requirements in the building decree result in a slender shape of the building, because the depth of rooms in the plan of the building is limited. The buildings in North America and New Zealand (figure 23) are less slender. Their shape will more resemble a solid square. With respect to this issue, figure 22 shows a nice example that can also be applicable in the Dutch situation.


figure 21: Residential building "The Reflection Building" in North Woolwich, London area, United Kingdom, 2002.


figure 22: Residential building "Casa Montarina" in Lugano, Switzerland, 2007.


figure 23: Residential building "Martin Square" in Wellington, New Zealand, 2004 (Milburn & Banks, 2004).

1.4 Benefits of timber-frame construction

As shown in the paragraphs before, there is a reasonable amount of experience with multi-storey timber-frame building abroad. Presently, only one example of a building with five storeys exists in the Netherlands. Today, the building industry is experiencing tough times. The effects of the global credit crunch are shown to be disastrous for building industry in the Netherlands. From 2008 on, the Dutch building industry is confronted with a decreasing volume of new-to-build buildings. Can it be concluded that the present and future situation look very difficult for building industry in general and multi-storey timber construction in particular? No, the present situation in building industry, and the perspectives in future do not have to be that bad. A number of phenomena's will drive multi-storey timber-frame building to use in the Dutch construction trade. These issues are related with the advantages of multi-storey timber-frame building and will be explained below.

Quick and clean construction

In densely populated areas, such as the city centre, the effect of nuisance to the environment because of construction activities can be of considerable influence on a building project. Use of prefabricated elements is almost inevitable in the inner urban environment. Building with timber is such a method. The time of construction can be very short, while it is one of the most important parameters to have minimal trouble for the neighbourhood. Even medium-rise projects with 5 to 8 storey's can be realised very fast with a minimum of nuisance. Less intensive transport movements are required. The aspect of quick construction is especially important for the renovation market. Urban renewal will become a more important activity for building industry in the years to come. As a result of the credit crunch, the new-to-build market will probably never become that booming again as before 2008. Urban renewal however, will become a frequent activity. In the period of rebuild after world war II, many residential buildings were constructed. New requirements with respect to sustainability and the increasing costs for energy will induce housing associations to renew, or refurbish, their outdated housing stock. This potentially will become a large market.

figure 24: Construction of a building in the city-centre.

Energy performance and sustainability

Realized projects demonstrated that with use of timber(-frame) construction excellent results can be achieved in the renovation market. Here it is where the advantages of timber construction become most obvious. Modern and comfortable buildings can be realised, which use a minimum of energy because of their superior thermal qualities. The increasingly important role of sustainability aspects in the structural design can be illustrated with the development of several labels and certificates for sustainability purposes such as Breeam and GreenCalc+. More and more clients ask for buildings with these types of labels. Also national and European governments increase their demands with respect to energy performance. Regulation is becoming more strict in future. These developments will result in a more advantageous market position for the timber material. Timber is a renewable material, if the timber is harvested from sustainably managed forests, and durability is guaranteed for a considerable lifespan in design. This in contrast to most of the alternative materials and building methods.

figure 25: Timber coming from sustainably managed sources can be recognised by the FSC and PEFC label.

Quality control and prefabrication

In recent years, a number of accidents made clear that structural safety is at risk. It became clear that the safety of structures, during construction and in service, is negatively affected by the increased number of parties involved in the construction process. Because of the increased phenomenon of outsourcing, money and time is cut on tasks and responsibilities at the interfaces. The consequences are decaying quality, and increasing costs for repair of failures. In 2008 these costs measured approximately 11% of the building volume, which is 6,1 billion euro's (ANP, 2010). The high level of prefabrication in Dutch timber-frame construction is beneficial for the quality of the timber-frame system. So far, timber-frame building is the sole construction method in the

Netherlands holding the KOMO attest with production certificate for the entire system. This ensures the quality to clients and users.

1.5 Masters' thesis research

In the paragraphs before, it was shown to be likely that actual realization of multi-storey timber-frame buildings in the Netherlands will be seen in the present future. Outside the Netherlands a lot of experience and knowledge is gained in analysis, modelling, and construction of multi-storey timber-frame buildings. Part of this knowledge and experience was reported about in the literature review. However, analysis and modelling of timber-frame shear-walls is treated with different objectives and methods in research and engineering. There are calculation methods that make use of an analytical approach, as well as methods that make use of finite-element analysis. Aimed is at determination of the dynamic, or static, characteristics of the structure. Often the seismic behaviour of the timber-frame structure is of interest in the analysis. The analytical methods are developed for strength-related issues in most cases. Not all of these methods and approaches will fit to the Dutch situation of timber-frame engineering. Goal of this master's thesis is to determine an analytical calculation method that is in agreement with the Dutch situation, and with NEN-EN 1995, the Eurocode 5. This code for structural design of timber structures, is governing for all member states of the European Union, and is based on the limit states approach.

Providing knowledge for structural design

In limit states design, a structural design needs to fulfil the demands in ultimate limit state (ULS) and serviceability limit state (SLS). Prevention of undesirable deformation or vibration is important because these can affect the appearance or effective use of the building, this is meant with serviceability. Therefore, the stiffness of the structure is analysed in 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. In Eurocode 5 (NEN-EN 1995), a calculation method is given to check the timber-frame shear-wall and floor panels for the strength-related requirements in the ultimate limit state. In the calculation method, the forces on the wall- or floor-diaphragm are assumed to be known, however, the force distribution in ULS can be dependent of the stiffness properties of the structural elements. In contrast with the strength-related requirements, the way in which the stiffness of timber-frame diaphragms has to be determined is not explicit prescribed in the current Eurocode 5. The only information given in the code are, the assumed force distribution within the timber-frame element in ULS, as well as a slip-modulus for wood-based panel-to-timber fasteners in SLS (K_{ser}), and ULS (K_u). It is not stated if this slip-modulus can be used to calculate the racking-stiffness of timber-frame shear-walls, and if so, how this has to be done. In conclusion can be said that engineer's knowledge and the code the information contains a gap with respect to this topic. In this master's thesis relevant knowledge is gained about the stiffness aspects in timber-frame construction, both in SLS and ULS. Insight is provided in analysis and modelling of the timber-frame shear-wall racking stiffness.

Efficiency of structural design

With use of the knowledge presented in this thesis, the deformations (SLS) of, and force-distribution in the building structure (ULS) can be calculated. The stiffness of the timber-frame structure can be estimated, which enables prediction of the dynamic behaviour of a building subjected to wind loading. In the paragraphs before, a number of reference examples of multi-storey timber-frame building were shown. In addition to these examples, there are many hybrid buildings in which a timber structure is stabilised with use of concrete or steel elements. These can be elevator shafts, staircases, steel frames, and steel braces for instance. In these projects the timber-frame elements are used only for separating functions. This is a pity, because the timber-frame element can also be used for structural purposes. Because of the structural and economical advantages a more efficient design can be made when the contribution of the timber-frame elements in the stabilizing system is quantified. Especially for the structural design of multi-storey buildings being higher than 3 storeys, the racking-stiffness of the timber-frame element will be of significant interest.

Research question

The objective of the thesis can be formulated with use of the research questions below. An answer to the research questions will be provided in the chapters hereafter. The central question of research is the following:

> *"Which parts or elements are of importance, and have to be included in the structural analysis and modelling of the timber-frame structure, in such a way that a useful prediction of deformations in serviceability limit state (SLS), and force distribution in ultimate limit state (ULS) can be made?"*

This research question can be divided into the following sub questions:

1. Which information can be found in literature about analyses and modelling of timber-frame shear-wall racking stiffness?
   a. Which parts of the timber-frame shear-wall determine the racking stiffness of the element?
   b. Are results available of experimental testing on these parts, and shear-wall elements as a whole?

The first research question was already answered in the literature and knowledge review. From the literature study could be concluded that the fastener and hold-down conditions are the most influential parameters for the racking stiffness of the shear-wall element. Therefore, the fasteners and hold-down stiffness will be analysed first. This will be done by answering the following questions:

2. Is it possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for the purpose of calculating the racking stiffness of timber-frame elements?
   a. How do the code-based stiffness slip-modulus K_{ser} and test-based slip-modulus k_s agree with each other?
   b. What can be said about the applicability of the slip-modulus, K_{ser}, K_u, k_s, and $k_{s(ULS)}$, to serviceability, and ultimate limit state?

3. Is it possible to predict the hold-down stiffness with use of an analytical calculation method?
   a. How do the results of this calculation method agree with results of experimental testing?

4. Is it possible to derive an analytical calculation method to calculate the timber-frame shear-wall racking stiffness?
   a. How does the calculation method look like?
      - Which contributions in the deformation have to be taken into account?
      - What is the share of each contribution in the total resulting racking deformation?
      - Which parameters and equations have to be used?
   b. Are the calculated racking stiffness values useful in structural design?
      - Do the calculation results agree with test-results?
      - For which purpose is the resulting racking stiffness to be used?

5. Is it possible to suggest a modelling approach for structural analysis of timber-frame shear-wall racking stiffness, using a regular framework program?
   a. How does the model look like?
   b. For which purposes is the modelling approach to be used?

6. Is it possible to use the determined calculation and modelling methods to analyse perforated shear-walls?

In the following chapters an answer to the research questions will be prepared. In chapter 7, the conclusions will be presented in answer to the research questions.

2 Mechanical fasteners in timber-frame structures

In the introduction of this master's thesis, a research question with respect to the fastener slip-modulus (fastener stiffness) was postulated:

> 2. *Is it possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for the purpose of calculating the racking stiffness of timber-frame elements?*
>   a. *How do the code-based stiffness slip-modulus K_{ser} and test-based slip-modulus k_s agree with each other?*
>   b. *What can be said about the applicability of the slip-modulus, K_{ser}, K_u, k_s, and $k_{s(ULS)}$, to serviceability, and ultimate limit state?*

In this chapter, an answer to the research question will be given comparing the results of 27 tests with the results of calculation according to Eurocode 5. These 27 tests comprise 157 specimens. It is necessary to explain the backgrounds of K_{ser} and k_s before being able to make a comparison, or, to say something about applicability to limit states design of the determined and calculated fastener stiffness,

The slip-modulus of the fastener is one the most important factor influencing the stiffness of the timber-frame shear-wall. In Eurocode 5 equations are given for the calculation of the fastener stiffness in serviceability limit state K_{ser}. To get more insight in the Eurocode 5 design rules about the fastener slip-modulus, and the applicability of this stiffness value to the structural analysis of timber-frame structures, a comparison will be made between the experimentally determined slip-modulus k_s and the calculated slip-modulus K_{ser}.

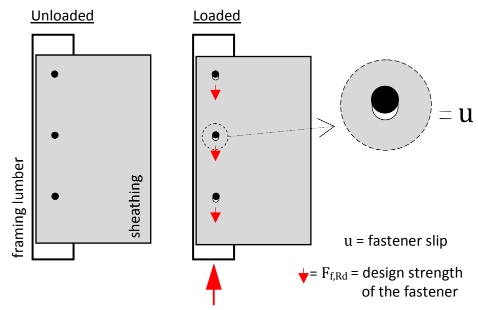


figure 27: Slip (u) of the fasteners when loaded

figure 26: Experimental test set-up to derive fastener slip-modulus. (Christovasilis et al., 2007)

The general principles for the experimental determination of strength and stiffness of dowel type fasteners are given in NEN-EN 26891. Reference is made to this standard in Eurocode 5. In chapter 7, *'Serviceability limit states'*, a remark is made which is stating that in NEN-EN 26891 the symbol of k_s is used instead of K_{ser}. Consequently, fastener slip-modulus either can be determined experimentally, or calculated using the Eurocode design rules.

2.1 Determining fastener slip-modulus k_s

From different sources in literature, results of experimental testing on sheathing-to-timber fasteners were found. The load-displacement graphs of these tests were presented in the literature review. Sheathing materials used in these tests are: oriented strand board (OSB), plywood, gypsum paper board (GPB), gypsum fibreboard, and particleboard. Screws, nails, and staples, are used to attach the sheathing to a timber member. Different test procedures, and test set-ups are used. In some of the reports a fastener slip-modulus is calculated, however, different approaches were used among the tests. As already mentioned earlier, in Eurocode 5 a remark is made about NEN-EN 26891. Therefore this standard is used to determine new stiffness values for all fastener tests.

2.1.1 Contents of NEN-EN 26891

In the standard NEN-EN 26891 is stated how to perform, and analyse, experimental testing of joints with mechanical fasteners. Prescriptions are given regarding conditioning of test specimens, form and dimension of test specimens, measurement apparatus, loading procedure, adjustment of the results, and, contents of the test report. In this paragraph only the procedure to derive the fastener-slip-modulus will be elaborated on.

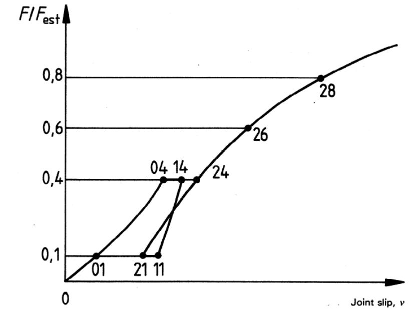


figure 28: Idealized load-deformation curve and measurements. (EN 26891)

Before doing the test series, the maximum load is estimated by a single test. This value is denoted as F_{est}.
The estimated maximum load is used to determine the loading procedure. When the fastener tests are done, the exact value for F_{est} is known, namely: F_{max}. In this manner the maximum load for a certain fastener is obtained. F_{max} will then be used to compute the slip-modulus k_s. To receive characteristic and design values for the strength of the fastener connection, F_{max} must be adjusted statistically.

The slip-modulus k_s is defined as following.

$$k_s = \frac{0{,}4 \cdot F_{est}}{v_{i,mod}} \quad 2.1$$

$$v_{i,mod} = \frac{4}{3}(v_{04} - v_{01}) \quad 2.2$$

$$F_{est} = F_{max}$$

v_{04} is the deformation on the load-level of $0{,}4 \cdot F_{max}$. v_{01} is the deformation on $0{,}1 \cdot F_{max}$. From the procedure can be seen that k_s does not contain possible initial slip occurring before the $0{,}1 \cdot F_{max}$ load level. Also the convex / concave curvature between 0,1 and $0{,}4 \cdot F_{max}$ is not contained in the value for k_s.

2.1.2 Limitations of available test-results

When testing of fasteners is performed according to NEN-EN 26891, a loop will be visible in the load-displacement graph (figure 28). The loop is a consequence of the applied loading procedure.

> *'The load shall be applied up to $0{,}4 \cdot F_{max}$ and maintained for 30 seconds. The load shall then be reduced to $0{,}1 \cdot F_{max}$ and maintained for 30 seconds. Thereafter the load shall be increased until the ultimate load or slip of 15 mm is reached.'* (NEN-EN 26891)

Eventually, the slip-modulus k_s is calculated based on the values in points 01 and 04 (figure 28). However, in none of the graphs found in literature, a loop is visible. As mentioned earlier, this is because of the different test methods.

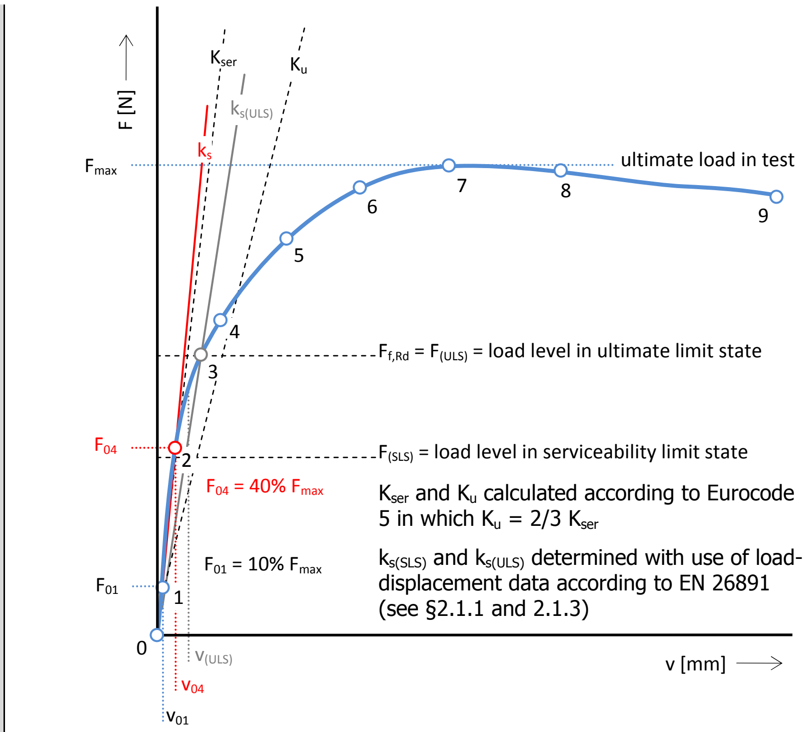


figure 29: Fictitious load-displacement graph with parameters.

In figure 29, a fictitious load-displacement graph is shown as an example. This graph is copied from literature using 9 measurements, which are the data points 0..9. These measurements are done by hand using a triangle (geodriehoek). Enough points were measured to receive the whole load-displacement curve, from start to failure. Due to this method, only a limited number of data-points is known. If not determined by the data points, the values for 0,4 and 0,1·F_{max} can be calculated by linear interpolation between adjacent data points. On average the load-displacement graphs were based on 19 measurements. However, as explained, only the initial part of the trajectory is of importance for the purpose of calculating the fastener-slip modulus.

2.1.3 Test-based slip-modulus at the ultimate limit state load level

'K_{ser} is the slip-modulus per fastener under service load' (Eurocode 5). Therefore it can be found in chapter 7 of the Eurocode 5, about *'Serviceability limit states'*. In a remark in §7.1, the possibility of experimental determination of k_s is mentioned, in the code is written that K_{ser} is denoted as k_s in NEN-EN 26891. Obviously, K_{ser} and k_s are only intended for use of serviceability limit state purposes. In Eurocode 5, the slip-modulus for the ultimate limit is given as K_u = ⅔ · K_{ser}. In this chapter will be discussed if K_u gives useful values for the fastener stiffness in ultimate limit state. In the NEN-EN 26891 standard is prescribed how to determine a fastener slip-modulus at the load levels of 0,6·F_{max} and 0,8·F_{max}. However, no mention is made for what purpose these values can be used. Limit state design issues are not mentioned at all.

To determine a fastener slip-modulus for ULS purposes on basis of the test-results, the following approach was followed in this master's thesis. First, the design strength ($F_{f,Rd}$) of the fastener in ultimate limit state (ULS) was calculated using the Johansen equations in Eurocode 5. This design strength was taken as a load for which a certain deformation ($v_{F_{f,Rd}}$) could be read from the load-displacement graphs. Finally a test-based stiffness parameter ($k_{s(ULS)}$) for the ULS was determined as following:

$$k_{s(ULS)} = \frac{F_{f,Rd}}{v_{i,mod}} \tag{2.3}$$

$$v_{i,mod} = \boxed{\frac{F_{f,Rd}}{F_{f,Rd} - F_{01}}}(v_{F_{f,Rd}} - v_{01}) \tag{2.4}$$

analogous to $\frac{4}{3}$ in equation 2.2

As a result, new information about the load levels in ULS was gained using the calculated values for the fastener design strength $F_{f,Rd}$. Information on the properties of the used sheathing-material, framing timber and fastener, often was incomplete. When it was not mentioned, the common used properties were taken for the calculation of $F_{f,Rd}$. Only in the Netherlands strength class C18 is used for framing timber. All tests found in literature were performed outside the Netherlands. Therefore, $F_{f,Rd}$ is calculated for framing timber C24, service class I, and the load-duration-class for short-term action. γ_M = 1,3 for connections. k_{mod} = 0,9 for timber, plywood, and OSB. k_{mod} = 0,85 for particleboard, and k_{mod} = 0,8 for gypsum-paper-board and gypsum-fibre-board. According to NEN-EN 1995-1-1:2005 §9.2.4.2 (4) and (5), the design strength $F_{f,Rd}$ is multiplied with $c_i \cdot 1{,}2$. This is taken into account in the calculation of $F_{f,Rd}$. The utmost value for $c_i \cdot 1{,}2$ is 1,12, when sheathing with dimensions 1220 x 2600 mm is used. The design strength $F_{f,Rd}$ was calculated using a standard spreadsheet for sheathing-to-timber connections for wood-based sheathing materials. These calculations are not appended to this master’s thesis. Reference is made to Appendix I (page 121).

2.2 Calculation of fastener slip-modulus K_{ser}

In Eurocode 5 (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 ρ_m, and fastener diameter d as parameters influencing the stiffness of the connection. Timber-to-timber connections and sheathing-to-timber connections are aimed at. Regarding the steel-to-timber connections, only the density of the timber member is involved in the equation, and the slip-modulus K_{ser} is multiplied by 2.

In Eurocode 5 gypsum-based sheathing-to-timber connections are not mentioned. About this type of connection, some information was found in the German code DIN 1052 (2008-12, tabelle 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. However, in the German national annex to Eurocode 5, no attention is paid anymore to the calculation of K_{ser} for gypsum paperboard panels, neither a 40% reduction is mentioned. As a consequence, no equations are available for the calculation of K_{ser} for gypsum paperboard or gypsum fibreboard sheathing.

2.2.1 Equations from Eurocode 5

For the screw, nail and staple different formula’s have to be used.

Nails:
$$K_{ser} = \frac{{\rho_m}^{1,5} \cdot d^{0,8}}{30} \tag{2.5}$$

Screws:
$$K_{ser} = \frac{{\rho_m}^{1,5} \cdot d}{23} \tag{2.6}$$

Staples:
$$K_{ser} = \frac{{\rho_m}^{1,5} \cdot d^{0,8}}{80} \tag{2.7}$$

In which:
$$\rho_m = \sqrt{\rho_{m,1} \cdot \rho_{m,2}} \tag{2.8}$$

For a print of the calculation sheet reference is made to Appendix I (page 121). Below some explanation is given about the properties used in the calculation of K_{ser}.

Density of the materials

As can be seen above, the density ρ_m is required as a parameter in the equations. It would have been nice when this parameter was given in all the test reports found in literature. However, often no information is given about the density of the materials that were used. On one hand, because they were not required, in such a case no attempt was made to calculate the stiffness values according to the code. On the other hand it could be the case that researchers did not notice the importance of the mean density, as a main parameter influencing the stiffness of the fastener connection.

Nevertheless, the calculations require information on the mean density of framing timber and sheathing material. In the calculation sheet in Appendix I (page 121) is stated which values are taken for the calculation. For the homogeneous materials, the mean density is the same as the characteristic density (used in the calculation of $F_{f,Rd}$). For plywood some different values were taken for the separate tests, these values are based on the information stated in the test reports. Plywood is an inhomogeneous material; therefore, the mean density is different from the characteristic density.

See the next page for an overview of the density-values that were used in the calculations.

Material:[1)]	Timber C24	OSB	Finish Spruce Plywood	Douglas-fir Plywood	CSP Plywood	Particleboard (t = 10 mm)	Gypsum paper Board (GPB)	Gypsum fibre board
ρ_m [kg/m³]	420	550 [1)]	460 [2)]	520 [3)]	450 [5)]	650 [1)]	900 [1)]	1210 [6)]
ρ_k [kg/m³]	350	550 [1)]	400 [2)]	460 [4)]	420 [5)]	650 [1)]	900 [1)]	1210 [6)]

[1)] If not stated different, values for the density are taken from 'Handboek houtskeletbouw' (Banga & Graaf, 2012)
[2)] Value for ρ_k according to the handbook of Finish Plywood
[3)] Calculated using the ratio ρ_k/ρ_m according to the handbook of Finish Plywood, and value for ρ_k from [4)]
[4)] Values for ρ_k taken from the may 1996 version of prEN 12369 Wood-base panels - Characteristic values for established products.
[5)] According to the test information, Canadian Softwood Plywood CSP grade CSA0151-M was used. A publication of CAN PLY gives values for ρ_k & ρ_m
[6)] Density of the gypsum fibre board specimen was mentioned in the test report (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 16-12-2002)

table 1: Mean and characteristic density of the materials used in calculation.

Effective diameter of the screw fastener

As can be seen from equation 2.5, 2.6, and 2.7, the diameter of the fastener is also a factor influencing the stiffness K_{ser}. The question arose which diameter needs to be taken for screws. For strength considerations, the Johansen equations are based on the embedment strength $f_{h,0,k}$. In the equations for the embedment strength, the diameter of the screws has to be taken as an affective diameter d_{ef}. When the diameter of the threaded part is not equal to the smooth shank, the effective diameter (d_{ef}) is taken as 1,1 times the inner thread diameter. According to NEN-EN 14592, this inner thread diameter is at least 0,6 times the thread diameter. Because it is assumed that stiffness parameter K_{ser} and embedment strength $f_{h,0,k}$ are equivalent parameters with respect to the characteristics of the timber, this approach is also followed in the calculation of K_{ser}. Consequently $d_{ef} = 1{,}1 \cdot 0{,}6 \cdot d$ gives $d_{ef} = 0{,}66 \cdot d$. In the calculations for K_{ser}, d_{ef} will be used.

figure 30: Standard drywall screws Ø3,8x45. $d_{ef} = 0{,}66 \cdot 3{,}8 = 2{,}51$ mm

2.3 Comparison slip-modulus k_s with K_{ser} for wood-based panels

Before taking notice of this paragraph it is important to know that the information on the density of the materials used in the tests not always was mentioned in the literature sources. Therefore, reasonable values are taken for the calculation of K_{ser}. These are reported in the table above. Furthermore, reference is made to the column charts on page 127. In this paragraph an answer to the research question will be given. For reason of clarity the research question is recalled below:

5. *Is it possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for the purpose of calculating the racking stiffness of timber-frame elements?*
   a. *How do the code-based stiffness slip-modulus K_{ser} and test-based slip-modulus k_s agree with each other?*
   b. *What can be said about the applicability of the slip-modulus, K_{ser}, K_u, k_s, and $k_{s(ULS)}$, to serviceability, and ultimate limit state?*

An answer to the research question will initially be formulated for each board material. In paragraph 2.8, the conclusions will be summed up.

2.3.1 OSB sheathing

Oriented strand board as sheathing material was used in 8 of 27 test series in total. These 8 tests, comprised 67 specimens. In each case nails were used. In this paragraph will be observed how the test-based slip-modulus k_s is related to the code-based slip-modulus K_{ser} and K_u for the OSB tests. From the column charts can be observed:

1 out of 8 tests: $k_s \geq K_{ser} \geq K_u$
2 out of 8 tests: $k_s \approx K_{ser}$
3 out of 8 tests: $K_{ser} \geq k_s \geq K_u$
2 out of 8 tests: $k_s \approx K_u$

On average $K_{ser} = 1{,}16 \cdot k_s$ for OSB sheathing.

A remarkable value for k_s was result of test (3.5) concerning a nail fastener Ø3,1x80 and OSB with thickness t = 11 mm. The average of 3 specimen resulted in a value for k_s of 1300 N/mm. The reason for this relatively high value is unclear. From the same source, also the test for particleboard was found (3.3). Also this particleboard test comprised 3 specimens and led to a higher value for k_s than expected from the calculation of K_{ser}. This can be an indication for a significant influence of the test-setup in the test series (3.x).

The fastener stiffness determined using the load and displacement at load-level 10% $\cdot$ F_{max} and the design strength $F_{f,Rd}$, will be denoted as $k_{s(ULS)}$. If $k_{s(ULS)}$ is related to K_{ser} and K_u, it can be seen that:

1 out of 8 tests: $k_{s(ULS)} \geq K_{ser} \geq K_u$
2 out of 8 tests: $k_{s(ULS)} \approx K_{ser}$
3 out of 8 tests: $K_{ser} \geq k_{s(ULS)} \geq K_u$
2 out of 8 tests: $k_{s(ULS)} \approx K_u$

On average $K_u = 0{,}78 \cdot k_{s(ULS)}$ for OSB sheathing.

There are minor differences between k_s and $k_{s(ULS)}$.

figure 31: Example of OSB (Oriented Strand Board)

F_{max} is the ultimate load reached in the tests, F_{04} is 40% of F_{max}. The deformation and fastener capacity on this load level is used for the determination of k_s in NEN-EN 26891. When the difference between F_{04} and, the design strength $F_{f,Rd}$ according to Eurocode 5, is observed, it can be seen that F_{04} and $F_{f,Rd}$ are almost the same for all OSB fastener tests, on average $F_{04} = 0{,}98 \cdot F_{f,Rd}$. This is also the reason for the minor differences between k_s and $k_{s(ULS)}$.

2.3.2 Plywood sheathing

Especially with respect to the plywood sheathing, the results of the comparison need to be treated carefully. A wide range of densities is produced by manufacturers of plywood. In most cases no information was available about the densities of the plywood used in the tests. Consequently an assumption had to be made, keeping in mind the variety within the plywood sheathing products. See also page 26. Assuming a density different from the density applied in the tests, has an influence on the comparison between k_s and K_{ser}, as well as the calculation of $F_{f,Rd}$. This will be explained below. 7 test series were found in literature, these 7 tests comprised 26 specimens.

In test (4) is stated that Canadian Softwood Plywood (CSP) Grade CSA 0151-M was used. From the CSP-plywood documentation on the internet a mean density (ρ_m) of 450 kg/m^{3}was found. From the papers, presenting tests (2.1), (2.2), and (4), information was given about the type of plywood that was used. In tests (2.1) and (2.2), Douglas-fir plywood is mentioned. Information about Douglas fir plywood was found in the may 1996 version of prEN 12369 Wood-based panels: '*Characteristic values for established products*'. The reports on tests (1.3), (8.3), (8.4), (8.5), did not contain information about the type of plywood. Therefore, it was chosen to take the density values of Finish-Spruce plywood, which are known from the Handbook of Finish Plywood. Consequently, the calculated values for K_{ser}, and $F_{f,Rd}$ are based on a density that can be questionable.

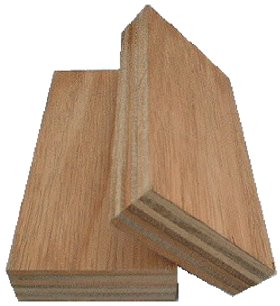


figure 32: Example of plywood

From the column charts can be observed:

1 out of 7 tests: $k_s \geq K_{ser} \geq K_u$
1 out of 7 tests: $k_s \approx K_{ser}$
4 out of 7 tests: $K_{ser} \geq k_s \geq K_u$
1 out of 7 tests: $k_s \approx K_u$

On average $K_{ser} = 1{,}08 \cdot k_s$ for Plywood sheathing.

Attention have to be paid to the result of test (1.3). From the column charts can be seen that k_s is larger than K_{ser}. This can be explained by means of the earlier mentioned difference between the assumed density of the plywood in the calculation of K_{ser}, and the density actually applied in the test. It is unsure however, if this is the right explanation for the difference. An error in the measurements does not seem likely because the other tests in this series do not display such a relatively large stiffness in the test. For reference, see the plywood and gypsum connections from series (1.x).

If $k_{s(ULS)}$ is related to K_{ser} and K_u, it can be seen that:

1 out of 7 tests: $k_{s(ULS)} \geq K_{ser} \geq K_u$
1 out of 7 tests: $k_{s(ULS)} \approx K_{ser}$
4 out of 7 tests: $K_{ser} \geq k_{s(ULS)} \geq K_u$
1 out of 7 tests: $k_{s(ULS)} \approx K_u$

On average $K_u = 0{,}72 \cdot k_{s(ULS)}$ for plywood sheathing.

The initial trajectory of the individual load-displacement graphs below 40% F_{max} could not be measured with the same accuracy for all tests. Also the ratio between 40% F_{max} and $F_{f,Rd}$ was different for the individual tests. These issues can give reason for a difference between k_s and $k_{s(ULS)}$, and the result of test (1.3). As can be observed from the test results, the design strength $F_{f,Rd}$ always stayed below 40% F_{max} except for test (4). On average, F_{04} is 19,4% higher compared with $F_{f,Rd}$.

Remark

In the Eurocode 5 equations for the calculation of $F_{f,Rd}$ a distinction is made between the different sheathing materials by means of the embedment strength $f_{h,k}$, this in contrast with the equations for K_{ser}. Only in case of plywood, the characteristic density of the sheathing (ρ_k), is used in the Eurocode equation for $f_{h,k}$. As a consequence, the calculated connection strength $F_{f,Rd}$ will be lower than the fastener in reality, if in the calculation a value for ρ_k is used lower than the density applied in reality.

2.3.3 Particleboard sheathing

For 27 sheathing-to-timber connections, load-displacement data were collected in the literature review. Only two of these tests (9 specimens) concerned particleboard sheathing. A nail fastener was used. Particleboard is a homogeneous wood-based material, therefore no defects or special additives influence the performance of the material. Based on the tests can be concluded:

2 out of 2 tests: $k_s \gg K_{ser}$ On average: $K_{ser} = 0{,}60 \cdot k_s$.

1 out of 2 tests: $k_{s(ULS)} \approx K_{ser}$
1 out of 2 tests: $k_{s(ULS)} \approx K_u$

Next to this: 40% F_{max} is lower compared with the calculated design strength $F_{f,Rd}$: $F_{f,Rd} = 1{,}49 \cdot F_{04}$.

figure 33: Example of particleboard

2.4 Slip-modulus k_s for gypsum-paper board and gypsum-fibre board

Because of their different characteristics, the gypsum-based sheathing materials are treated separately in this paragraph. Important to notice is that the gypsum paper boards are homogeneous products. All manufacturers deliver the standard gypsum paperboard with the same properties. The properties of the gypsum fibre boards however, are characterized by the variety of different fibres added to the gypsum.

figure 34: Example of Gypsum paper board

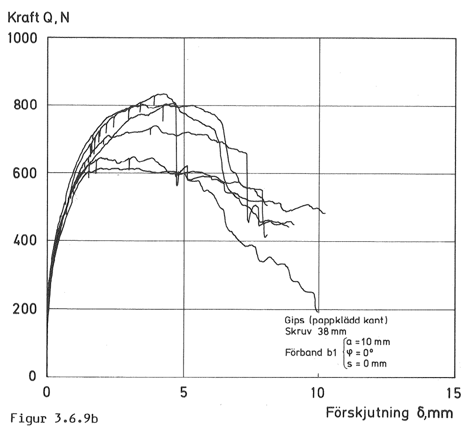
In Eurocode 5 (EN-1995-1-1) no design rules are given for the application of gypsum-paperboard or gypsum-fibreboard sheathing. However, in DIN-EN-1995-1-1/NA rules are given for the calculation of the fastener strength in case of gypsum paperboard sheathing. These design rules, for timber-frame shear-walls sheeted with gypsum paperboard or gypsum fibreboard, are nowadays also approved for implementation in NEN-EN-1995-1-1/NB, the Dutch national annex to the Eurocode 5.

Based on the equations for gypsum paperboard, given in the German national annex, a value can be calculated for the embedment strength $f_{h,k}$, and fastener strength $F_{f,Rd}$. Although rules for the calculation of $f_{h,k}$ are given in DIN-EN 1995-1-1/NA, no attention is paid to the slip-modulus K_{ser}. The applicability of the equations for K_{ser} to the gypsum sheathing-to-timber connections is not mentioned. Therefore, no value for K_{ser} could be calculated for the gypsum-based sheathing materials. Consequently no comparison could be made between a calculated slip-modulus K_{ser}, and experimentally determined slip-modulus k_s. The determined values for k_s can be found on page 32.

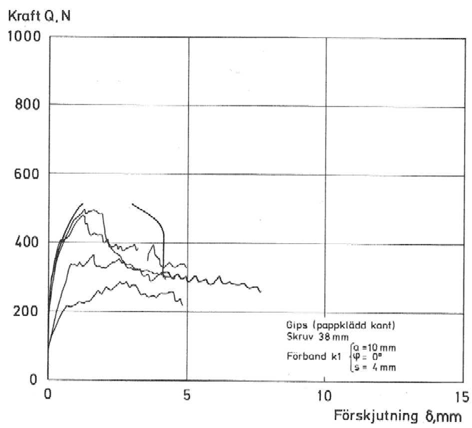
2.4.1 Gypsum-paper board (GPB)

For the gypsum paperboard sheathing, results were found from 4 tests series (52 specimens). When the values for k_s are observed (Appendix II), a strange value for the fastener-slip-modulus k_s is immediately visible from test (7.4). The load-displacement data for this test is the average load-displacement behaviour of 10 test specimen. The column-chart in Appendix II shows a magnitude for k_s of $25 \cdot 10^2$ N/mm. If the effective diameter of the screw is observed (0,66 · 3,3 = 2,2 mm) the test-based stiffness of the connection, k_s, should be comparable with the value for the fastener stiffness k_s received from test (7.2). In test (7.4), a screw Ø3,3 x 45 is used, in test (7.2), a nail fastener Ø2,2 x 45 is used. Test (7.2) is performed in the same experiment, is based on of 10 specimens, and reported in the same publication[1]. The tests were done according to the same procedure, only the fastener that was used is different. Assuming an error in the measurements cannot be the explanation for the extraordinary high stiffness for test (7.4). A similar high value can also be seen for test (15.4), in this test k_s is measured to be $20 \cdot 10^2$ N/mm. In this case also an error in the test set-up has to be excluded from explanation. Both cases give reason to conclude that the stiffness of a screw fastener in gypsum-paper board can display high variety.

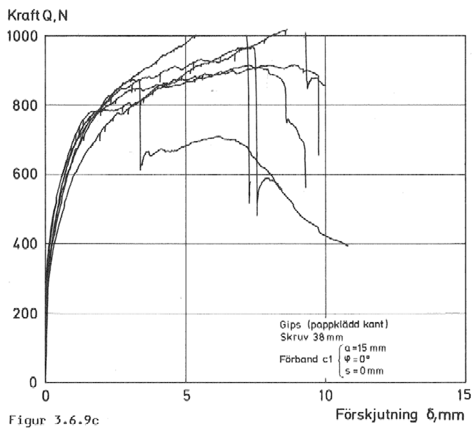
Although similar fasteners were applied, the high values for the screwed gypsum-to-timber connections in tests (7.4) and (15.4) do not agree with the stiffness values found in tests (8.1) and (15.5). A reason for this can be damage to the paper reinforcing envelop in case of tests (7.4) and (15.4). As can be seen in the figure below, perforation of the paper can have a large influence on the ultimate strength of the fastener in gypsum-paper board. This will also affect the initial stiffness, because in the way it is calculated, use is made of the 40% F_{max} load-level. In the initial trajectory of the load-displacement graphs, the worsening effect is not present, however because of the lower ultimate strengths, the stiffness values are based on different load-levels. This is a reason for weakening becoming visible in the initial stiffness.



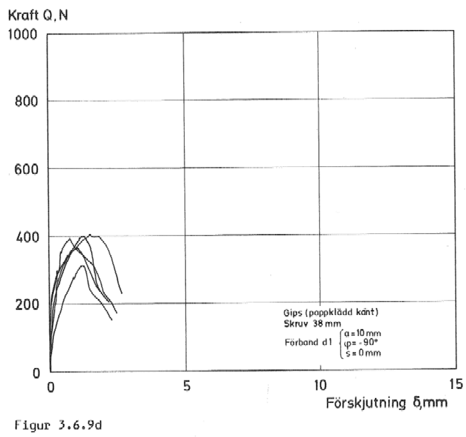
Edge distance a = 10 mm / Fastener loaded parallel to lined sheathing edge



Edge distance a = 10 mm / Paper perforated by the screw, screw head 4 mm below panel surface / Fastener loaded parallel to lined sheathing edge



Edge distance a = 15 mm / Fastener loaded parallel to lined sheathing edge



Edge distance a = 10 mm / Fastener loaded perpendicular to sheathing edge

figure 35: Gypsum paper-board fastener-tests, Gyproc t = 12,5 mm, screw Ø3,0x37,6 (Källsner, 1984)

An average value for k_s can be based on the 36 specimen from test (1.1), (7.2), (8.1), (15.3), (15.2), and (15.5) $k_s = 52 \cdot 10^1$ [N/mm]. In the table below the test-based values for the fastener-slip k_s are shown:

[1] (Bouw- en woningtoezicht Rotterdam, 1989)

Fastener	**Material** (thickness t [mm])	**Slip-modulus** [N/mm]	**Test**
N Ø2,34x40 (GN40)	GPB (12,0)	$32 \cdot 10^1$	(1.1)
N Ø2,2x45	GPB (12,5) Gyproc	$56 \cdot 10^1$	(7.2)
Scr Ø3,9 x 41	GPB (13,0) Danogips Normal	$36 \cdot 10^1$	(8.1)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (lined edge)	$64 \cdot 10^1$	(15.3)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (sawn edge)	$55 \cdot 10^1$	(15.2)
Scr Ø3,0 x 38	GPB (12,5) Gyproc Normal (sawn edge)	$67 \cdot 10^1$	(15.5)

table 2: Experimentally determined (mean) values for fastener slip-modulus k_s for gypsum-paper board.

Gypsum paper-board (GPB)
Nail (N)
Screw (Scr)

When the design strength according to Eurocode 5, $F_{f,Rd}$, is compared with F_{04} for all tests, one can see that $F_{04} \approx F_{f,Rd}$. If test (7.4) and (15.4) are not considered, the average ratio between F_{04} and $F_{f,Rd}$ will be: $F_{04} = 1{,}10 \cdot F_{f,Rd}$.

2.4.2 Gypsum-fibre board

The gypsum fibre boards are different from the gypsum paper boards. No reinforcing paper envelop is present. Reinforcing additives like paper, glass or sisal fibres, are added to the gypsum mixture. Consequently, the *'gypsum fibre boards'* are a group of boards with varying properties. Every manufacturer delivers a different type of gypsum fibre board. Therefore it is not surprising that the determined values for k_s do not give reason for assuming a clear relationship between stiffness of the fastener, and its type of fastener, diameter, or length.

figure 36: Material components of the Fermacell® gypsum fibre board. (On the left: cellulose fibres made from recycled paper)

The values for k_s shown in Appendix II differ very much. The slip-modulus determined from test (3.1) shows a relatively high value if compared with the other staple fastener tests in gypsum-fibre board. A reasonable explanation could be that the results of the tests from the series (3.x) were influenced by the test-setup, or the processing of the measurements, as was already concluded earlier on page 27. Also the tests with OSB and particleboard specimens, test (3.4) and (3.3) showed relatively high values for the fastener slip-modulus k_s. When test (3.1) is dismissed, an average value for k_s can be calculated based on 5 tests (22 specimens): $k_s = 84 \cdot 10^1$ [N/mm] for the tests with nail fasteners, and $k_s = 50 \cdot 10^1$ [N/mm] for the tests with staple fasteners. In the table below the test-based values for the fastener-slip k_s are shown:

Fastener	**Material** (thickness t [mm])	**Slip-modulus** [N/mm]	**Test**
St Ø1,4 x 1,6 x 60	GFB (12,5) (1-layer homogeneous) Fermacell	$40 \cdot 10^1$	(14)
St Ø1,53 x 50	GFB (15,0) (3-layer homogeneous) Rigidur RH	$46 \cdot 10^1$	(12.1)
N Ø2,5 x 65 (Rillenagel)		$27 \cdot 10^1$	(12.2)
St Ø1,8 x 65		$65 \cdot 10^1$	(13.1)
N Ø2,5 x 55		$140 \cdot 10^1$	(13.2)

table 3: Experimentally determined (mean) values for fastener slip-modulus k_s for gypsum-fibre board.

Gypsum fibre-board (GFB)
Nail (N)
Staple (St)

When the determined strength values are considered, reasonable agreement exists between the tests. From a comparison between the design strength ($F_{f,Rd}$)[2] and the observed ultimate load (F_{max}), the following conclusion can be drawn. $F_{f,Rd}$ is of the same magnitude as 40% F_{max}, $F_{f,Rd} = 1{,}05 \cdot F_{04}$ on average. (6 tests, 25 specimens)

2.5 Applicability to serviceability limit state

From the OSB, plywood, and particleboard tests can be concluded that the code-based fastener slip-modulus K_{ser} is almost equal to the test-based slip-modulus k_s. Therefore it can be concluded that it is suitable to use the code-based fastener slip-modulus K_{ser} for design purposes in the serviceability limit state (SLS). K_{ser} gives a good estimation of the fastener stiffness in the initial trajectory of the load-displacement graph, which determines the behaviour of the connection in SLS. A more elaborate explanation is given below using figure 37.

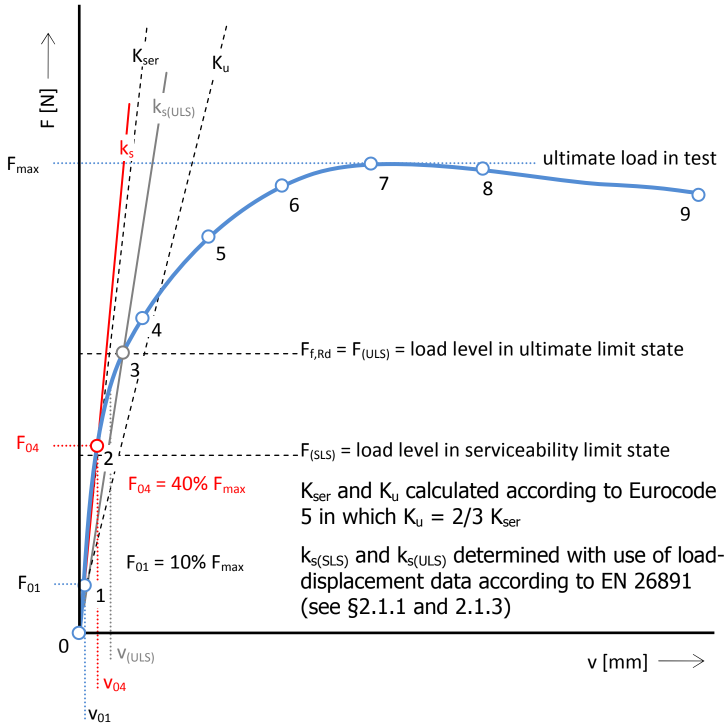


figure 37: Fictitious load-displacement graph with parameters.

10% and 40% F_{max} are the load-levels used to determine the fastener slip-modulus k_s ($\approx K_{ser}$). From the observations discussed in the paragraphs before can be concluded that $F_{f,Rd} \approx$ 40% F_{max}. If $F_{f,Rd}$ is the ultimate capacity of the sheathing-to-timber connection in ULS, the load-level in SLS can be estimated to be $F_{f,Rd} / \gamma_Q$ which lead to 40% F_{max} / 1,35 = 30% F_{max} in case of standard dwellings, and 40% F_{max} / 1,50 = 27% F_{max} for apartment buildings. Consequently, k_s ($\approx K_{ser}$) can be used for serviceability limit state purposes such as calculating the deflections of the timber-frame shear-wall.

In case of particleboard sheathing it is questionable if the assumption $F_{f,Rd} \approx$ 40% F_{max} is valid. Because $F_{f,Rd} = 1{,}49 \cdot$ 40% F_{max} on average. If particleboard sheathing is applied in a dwelling, $1{,}49 \cdot$ 30% F_{max} will become 45% F_{max}. This load-level is a little bit higher compared with 40% F_{max} and will lead to a smaller stiffness. However, for the same tests it was shown that $K_{ser} = 0{,}60 \cdot k_s$ on average. Now it can be concluded that K_{ser} still remains a good estimation for the fastener stiffness in SLS in case of particleboard sheathing-to-timber connections.

2.6 Applicability to ultimate limit state

As already explained in the literature review, the stiffness of the timber-frame shear-wall has its influence on the force distribution between the shear-walls in plan of the building. To determine the stiffness of the wall in the

[2] Determined using the equations for the embedment strength $f_{h,k}$ provided in the Handboek Houtskeletbouw (Banga & Graaf, 2002) The equation is based on a European Technical Approval (ETA-03/0050).

ultimate limit state (ULS), a slip-modulus for the fastener in ULS is required. K_{ser} is the slip-modulus per fastener under service load (Eurocode 5), this is already discussed in §2.1.3. Design rules for the calculation of K_{ser}, can be found in chapter 7 of the Eurocode 5. In Eurocode 5, §2.2.2 *‘design requirements for ultimate limit states’*, is stated that the slip-modulus for the ultimate limit state (ULS), K_u, should be taken as K_u = ⅔ K_{ser}.

When K_u is compared with $k_{s(ULS)}$, it is shown that K_u from Eurocode may result in too low values. K_{ser} however, may result in values too high. This can be seen from the calculation sheet in Appendix I (page 121)., and from the column charts in Appendix II. $k_{s(ULS)}$ is the test-based slip-modulus determined on the ULS load level $F_{f,Rd}$. From the tests on OSB, plywood, and particleboard can be concluded that the stiffness of the fastener at the design strength ($F_{f,Rd}$) load level, $k_{s(ULS)}$, can vary between K_{ser} and K_u. Therefore the following conclusion will be proposed. For ULS purposes, such as determining the force distribution within the structure, the limit values K_{ser} and K_u for the stiffness of the sheathing-to-timber connection have to be evaluated both. The most unbeneficial value have to be taken.

2.7 Applicability of average slip-modulus

Because the presented load-displacement data gives average load-displacement curves, one may argue that the conclusion above is conservative for 50% of the test specimen. Each test consist of 6 specimens on average. (the 27 tests comprise 157 specimens) The mean stiffness k_s will be representative for the behaviour of the fastener if enough ‘specimen’ are used in a connection. When the timber-frame shear wall is considered, this requirement is fulfilled. Along the perimeter of the wall, a significant number of fasteners is employed. Therefore, it is justified to use the average value for the fastener-slip modulus instead of a characteristic value.

2.8 Conclusion

In the paragraphs before, the fastener slip-modulus has been analysed. Both the Eurocode 5 design rules, and experimental results were observed. An answer to the research question can be given.

> *2. Is it possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for the purpose of calculating the racking stiffness of timber-frame elements?*
> *a. How do the code-based slip-modulus K_{ser} and the test-based slip-modulus k_s agree with each other?*
> *b. What can be said about the applicability of the slip-modulus, K_{ser}, K_u, and k_s, to serviceability, and ultimate limit state?*

In answer to the research question, the following conclusions can be drawn.

Ratio between the calculated and experimentally determined slip modulus

Conclusion 2.:

> *It is possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for calculating the racking stiffness of the timber-frame elements.*

Conclusion 2.a.:

> *Reasonable agreement can be seen between the code-based slip-modulus K_{ser}, and the test-based slip-modulus k_s. For gypsum-fibre board and gypsum-paper board the test-based slip-modulus k_s can be used.*

For the wood-based sheathing materials OSB, plywood and particleboard, a slip-modulus K_{ser} can be calculated with use of the equations in Eurocode 5. This slip-modulus K_{ser} is almost equal to the experimentally determined slip-modulus k_s:

- OSB: K_{ser}/k_s = 1,16 ± 0,18
- Plywood: K_{ser} / k_s = 1,08 ± 0,14
- Particleboard: K_{ser} / k_s = 0,60 ± 0,12 (2 tests, 9 specimen)

For the sheathing-to-timber connections with gypsum paper board (GPB) and gypsum fibre board (GFB) no slip-modulus K_{ser} can be calculated. An experimentally based slip-modulus k_s [N/mm] is determined:

Fastener	**Material** (thickness t [mm])	**Slip-modulus** [N/mm]	**Test**
N Ø2,34x40 (GN40)	GPB (12,0)	$32 \cdot 10^1$	(1.1)
N Ø2,2x45	GPB (12,5) Gyproc	$57 \cdot 10^1$	(7.2)
Scr Ø3,9 x 41	GPB (13,0) Danogips Normal	$36 \cdot 10^1$	(8.1)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (lined edge)	$64 \cdot 10^1$	(15.3)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (sawn edge)	$55 \cdot 10^1$	(15.2)
Scr Ø3,0 x 38	GPB (12,5) Gyproc Normal (sawn edge)	$67 \cdot 10^1$	(15.5)
St Ø1,4 x 1,6 x 60	GFB (12,5) (1-layer homogeneous) Fermacell	$40 \cdot 10^1$	(14)
St Ø1,53 x 50	GFB (15,0) (3-layer homogeneous) Rigidur RH	$46 \cdot 10^1$	(12.1)
N Ø2,5 x 65 (Rillenagel)		$27 \cdot 10^1$	(12.2)
St Ø1,8 x 65		$65 \cdot 10^1$	(13.1)
N Ø2,5 x 55		$141 \cdot 10^1$	(13.2)

table 4: Experimentally determined (mean) values for fastener slip-modulus k_s (K_{ser})

Gypsum paper-board (GPB)
Gypsum fibre-board (GFB)
Nail (N)
Screw (Scr)
Staple (St)

Limit state design with respect to the fastener-stiffness

Conclusion 2.b.:

> *In the structural analysis of stiffness of timber-frame shear-wall, the calculated fastener-slip-modulus K_{ser} and the experimentally determined fastener-slip-modulus k_s may be used for serviceability limit state purposes such as calculating the deflections of the timber-frame shear-wall. For ultimate limit state purposes, such as determining the force distribution within the structure, both K_u and K_{ser} have to be evaluated, the governing value will be the least advantageous one. $K_u = 2/3 \cdot K_{ser}$ (or $2/3 \cdot k_s$)*

2.9 Recommendations

As a result of the analysis of the fastener stiffness a few recommendations can be given.

- Only two tests were found in the research for the literature review, which is a basis to small for drawing a conclusion. It is likely that more tests on sheathing-to-timber connections with particleboard can be found in literature. More searching for experimental test-results on sheathing-to-timber connections would be useful, in the literature review a limited amount of time could be invested in searching for the results of these tests. (This can be relevant for the other sheathing materials too.)

- The big spread in the results of the tests on gypsum-based sheathing-to-timber connections suggest that the stiffness of these type of connections not only depends on density, and fastener diameter. Other effects, like edge distances and penetration of the paper reinforcement, can play an important role. Perhaps more severe criteria have to be applied to the gypsum-based sheathing-to-timber connections compared with the wood-based sheathing-to-timber connections. The formulation of these criteria and additional (experimental) research could lead to the formulation of an equation to calculate a fastener-slip-modulus K_{ser} for connections executed with gypsum-paper board, and gypsum-fibre board.

3 Hold-down connectors

To prevent uplift of the timber-frame shear-wall, hold-down anchorage is applied. The need for sufficient anchorage was already discussed in the literature review. It was shown, that the stiffness of the anchorage (k_{hd}) can have disproportionate effects on the stiffness of the timber-frame shear-wall as a whole. Therefore, the following research question was postulated:

> 3. *Is it possible to predict the hold-down stiffness with use of an analytical calculation method?*
>   a. *How do the results of this calculation method agree with results of experimental testing?*

In answer to the research question, an analytical approach is proposed to calculate the stiffness for a certain hold-down solution. This approach will be based on the fastener-slip modulus (K_{ser}). To discuss the applicability of the analytical approach, a comparison will be made between test-based ($k_{hd;test}$) and analytically derived stiffness values ($k_{hd;Kser}$). Load-displacement data concerning the destructive testing of hold-down anchorage, was generously provided by RothoBlaas GmbH[3], and was found in literature. Before presenting the calculation method, and the results of comparison with test results, the structural application of hold-down anchorage will be explained.

3.1 Structural application of hold-down anchorage

In this paragraph, the use of hold-down provisions will be shortly discussed first. Secondly, the magnitude of load for a certain hold-down provision will be derived. Information about the magnitude of load on a tie-down anchor is required to enable proper judgment of the test-results.

Need for hold-down anchorage

In experimental testing of timber-frame shear-walls as well as in practice, there can be a need for hold-down anchorage. Reason for this is the absence of sufficient vertical loading to counteract the tensile forces in the leading edge-stud of the shear-wall. The tensile load in the edge-stud is caused by horizontal loading on the top-rail. In experimental testing a number of measures can be taken to prevent uplift of the tensile stud. These can be seen in figure 38. In (a) a roller is mounted to keep the top rail in horizontal position. In (b) vertical load is applied to prevent a tension force in the leading stud to originate. In (c) the racking load is combined with a vertical load on the leading stud, this can be achieved by applying a diagonal load. In (d) the application of a hold-down anchor can be seen.

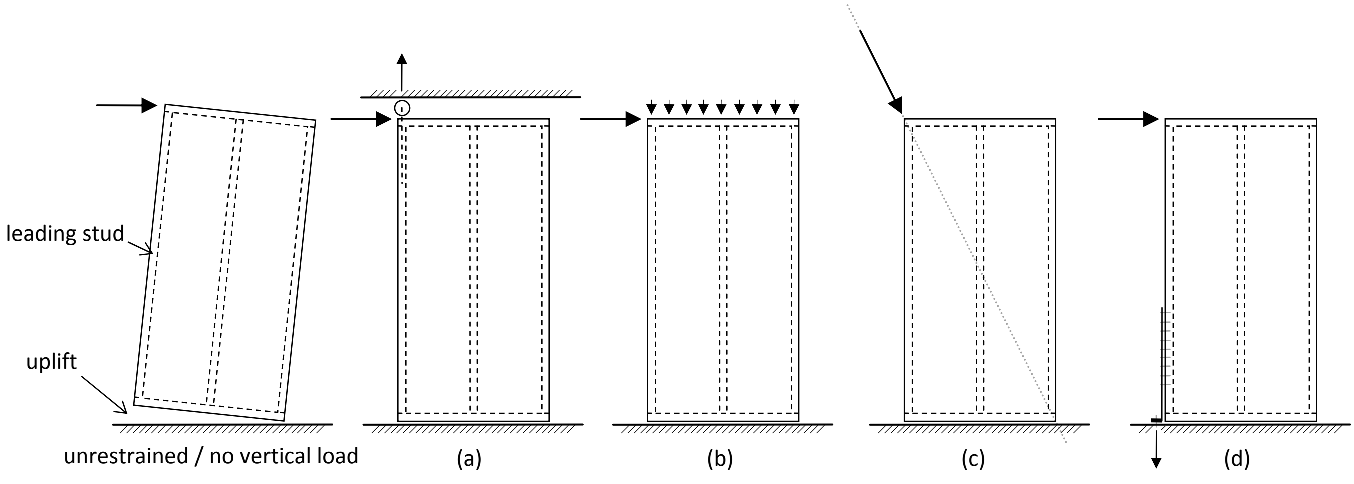


figure 38: Several test set-ups to prevent uplift of the leading stud. (Fasteners along the perimeter, and shear connectors in the bottom rail are not shown in the drawing.)

[3] RothoBlaas GmbH is a manufacturer of a variety of materials for purpose of timber carpentry. These materials include all kinds of necessities, such as tools, fasteners, fixings, anchorage solutions, etc. Load-displacement data from experimental testing on hold-down anchorage is presented in the literature review. Website: http://www.rothoblaas.com

In the full-scale shear-wall tests that were found in literature, the test methods of testing are approximately divided as following: in ⅓ of all cases, uplift was not prevented. Apparently, the researchers were not aware of the disadvantageous effects, or they accepted lower racking strengths. In ⅓ of the tests, uplift was prevented using a roller mounted to the test-rig. In the remaining ⅓ of the tests, uplift was prevented using a hold-down anchor. Another option, which can also be chosen, is the application of vertical load. In practice there will always be a certain vertical load coming from dead weight of the building levels above. The tensile forces in the vertical studs can be cancelled or reduced by this vertical load. In such a case, no, or a less strong hold-down will be applied. Because the hold-down anchorage solution never will be of a finite stiffness, the deformation in the anchorage will cause a certain part of the horizontal wall deflection. To quantify this contribution in the racking deformation, more insight into the contribution of the hold-down anchor will be gained in this chapter.

Magnitude of the design strength

Based on the tests, the elastic stiffness of the hold-down will be determined ($K_{hd;test}$). Similar to the evaluation of the fastener tests, the displacement at 10% and 40% of the ultimate load (F_{max}) will be used to determine this experimental stiffness value. However, it is questionable if the load on the hold-down in practical application ($F_{t,Ed}$) will be below the 40% load-level in SLS and ULS. Next to this, it is questionable if the load will remain in the elastic trajectory of the load-displacement graph. Will the stiffness of the hold-down remain elastic if the design strength ($F_{t,Rd}$) is reached?

The ultimate loads that were reached in the hold-down tests, are not related to the design strength of a shear-wall. The only purpose of the tests was, to determine the ultimate capacity of the hold-down devices. Therefore, it is unsure to what extend the tested hold-down configurations will be utilized in the real situation of application. Although unsure, an estimation of the usual load level has to be made in order to check the validity of a linear-elastic modelling approach. In engineering practice, a hold-down anchor is dimensioned based on the demands with respect to the tensile strength to be taken up by the hold-down. Therefore the determined elastic stiffness will be judged on basis of the ultimate design strength of the hold-down anchor or connector ($F_{hd;Rd}$).

The design strength of the hold-down can be calculated analytically according to Eurocode 5. In this calculation use is made of the fastener strength $F_{v,Rd}$, which can be calculated using the Johansen equations for the steel-to-timber connections. The formulas contain the so-called Johansen part, bringing in account the effect of embedment, and the development of plastic hinges. Secondly, there is the rope-effect, which includes the contribution of the withdrawal strength to the transverse resistance. Annular nails have a much higher withdrawal strength compared with standard smooth nails. Therefore, the rope effect and withdrawal strength are treated with caution in Eurocode 5. The rope effect in general is reduced to 25%, and for fasteners different from screws, only a part of the withdrawal strength can be calculated with. Consequently, the design rules lead to a low value for the fastener strength in case of steel-to-timber fasteners. From tests, much higher values are retrieved. Therefore, the values for $F_{v,Rk}$ as given in the documentation of Simpson are very different from the values for $F_{v,Rk}$ as provided by Rothoblaas (see table 5).

figure 39: Example of an annular nail: the ringed profile provides additional withdrawal strength.

For the fasteners used in the tests the following values for the fastener strength can be obtained:

hold-down test:	Simpson Strong-Tie HTT16			RothoBlaas Rothofixing WHT80/620								
used fasteners:	nail Ø4,11 x 89 (smooth)			nail Ø4 x 40 (annular)			nail Ø4 x 75 (annular)			screw Ø5 x 50		
fastener strength $F_{v,Rk}$ [kN]:	1,48 [1]	--	--	1,32 [1]	1,83 [2]	1,31 [3]	1,47 [1]	2,50 [2]	1,75 [3]	1,34 [1]	2,51 [2]	1,97 [3]

[1] Calculated using Eurocode 5 NEN-EN 1995-1-1 (steel thickness: 3,0 mm) see Appendix III.
[2] Obtained from Simpson Strong-Tie documentation: CNA-Kammnägel & CSA Schrauben
[3] Obtained from RothoBlaas documentation: Anchor nails & Screws for plates

table 5: Characteristic values for the steel-to-timber fastener strength

For the calculation of the design strength of the hold-down anchor, the values provided in the documentation of Simpson Strong-Tie will be used. Although these are relatively high values, they are reliable. European Technical Approval (ETA) has been obtained by experimental verification of the transverse strength, as well as the withdrawal strength. From the characteristic fasteners strength ($F_{v,Rk}$), presented in the table above, the design strength ($F_{v,Rd}$) can be calculated as following: (Service class I, short term loading)

$$F_{v,Rd} = F_{v,Rk} \cdot \frac{k_{mod}}{\gamma_M} = F_{v,Rk} \cdot \frac{0,9}{1,3} \tag{3.1}$$

The design strength of the fasteners ($F_{v,Rd}$) will be multiplied by the applied number of fasteners (n) to receive the design strength ($F_{hd,Rd}$) of the hold-down:

$$F_{hd,Rd} = n \cdot F_{v,Rd} \tag{3.2}$$

These calculations can be found in Appendix IV, page 135.

3.2 Analytical calculation of hold-down stiffness

In this paragraph will be explained how the stiffness ($K_{hd;Kser}$) of the hold-down can be estimated analytically. If the steel and timber cross sections are considered to be inextensible, all fasteners will act as a parallel system. Therefore the total stiffness of the system can be computed by multiplying the number of fasteners (n) with the stiffness of a single fastener. The stiffness of a single fastener is assumed to be equal to the slip-modulus of the fastener K_{ser}. From figure 41 the result can be seen. The hold-down test (1.1), a Rothofixing WHT 80/620 hold-down with 50 annular nail fasteners Ø4 x 40 will be treated as an example below.

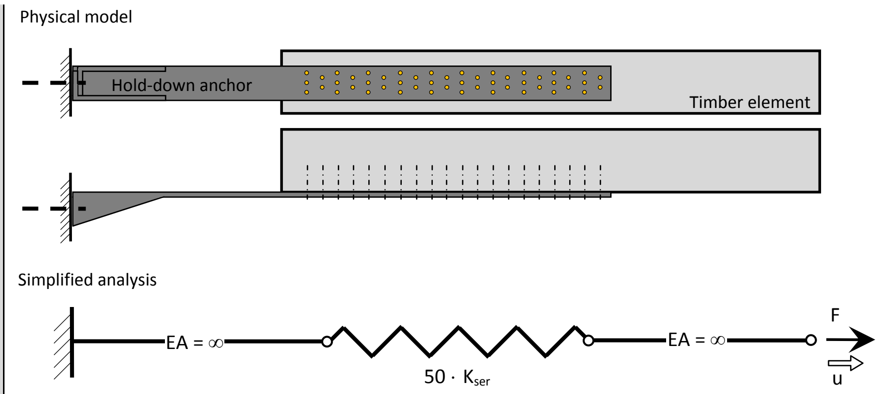


figure 40: Simplified calculation of the hold-down stiffness for Rothofixing WHT80/620 hold-down & 50 nails Ø4 x 40

For an annular nail Ø4 x 40 the slip-modulus K_{ser} can be calculated to be 870 N/mm (C24) in case of a steel-to-timber joint, K_{ser} has to be multiplied by 2. This results in: $2 \cdot K_{ser}$ = 1740 N/mm. The Rothofixing WHT 80/620 hold-down can be fixed to the timber with a maximum number (n) of 50 nails. The analytically derived stiffness of this hold-down will be:

$$K_{hd;Kser} = n \cdot K_{ser} = 50 \cdot 1{,}74 \cdot 10^3 = 86{,}98 \cdot 10^3\ N/mm \qquad 3.3$$

This calculation is made for all available tests. For the results, reference is made to Appendix IV, page 135.

3.3 Test-based hold-down stiffness

Based on the tests, the elastic stiffness of the hold-down will be determined ($K_{hd;test}$). Similar to the evaluation of the fastener tests according to NEN-EN 26891 (§2.1, page 23), the displacement at 10% and 40% of the ultimate load (F_{max}) will be used to determine this experimental stiffness value. For the determined stiffness values, reference is made to Appendix IV, page 135.

3.4 Comparison between test-based and calculated results

In order to validate the proposed method, a comparison is made between the values for the hold-down stiffness derived with an analytical approach, and the stiffness values determined from the load-displacement data. A significant difference was observed between the calculated, and experimentally determined values. An explanation for this difference will be searched for in this paragraph, using a more detailed analysis of three tests. As a conclusion an improved method to calculate the hold-down stiffness will be proposed.

3.4.1 Observations

When the analytically determined hold-down stiffness ($n \cdot K_{ser}$) is compared with the test-based hold-down stiffness ($k_{hd;test}$) it can be seen that the experimentally determined stiffness of the hold-down anchor is much lower compared with the calculated stiffness on basis of the fastener-slip modulus K_{ser}. In figure 41 are shown the calculated values for the hold-down stiffness: $n \cdot K_{ser}$, and the experimentally determined values for the hold-down stiffness: $k_{hd;test}$.

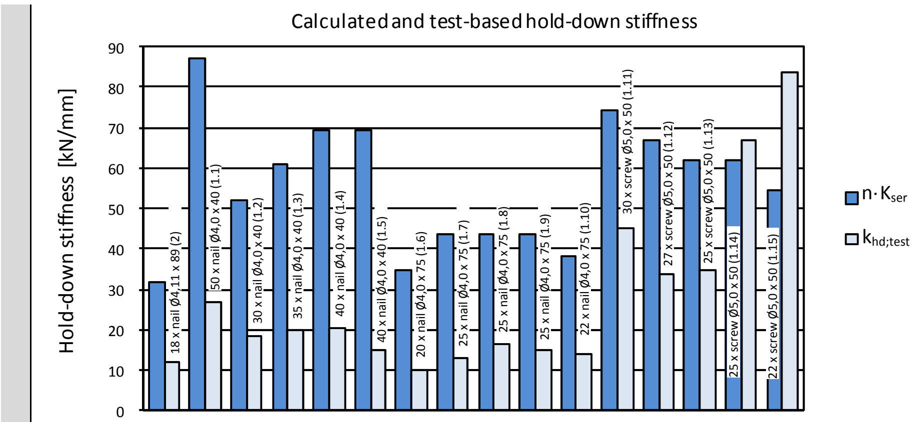


figure 41: Comparison between $n \cdot K_{ser}$ and $k_{hd;test}$.

From figure 41 can be concluded that no agreement exists between the calculated stiffness value and the test-based stiffness value for the hold-down stiffness in the case of nail-type fasteners. On average, $n \cdot K_{ser} = 2{,}62 \cdot k_{hd;test}$, Factors contributing to the difference between $n \cdot K_{ser}$ and $k_{hd;test}$, will be quantified in the next paragraph. A detailed analysis will be carried out to calculate a more accurate estimation of the hold-down stiffness.

Test:		(2)	(1.1)	(1.8)
hold-down & configuration :		Simpson Strong-Tie HTT16 18 x nail Ø4,1 x 89	Rothofixing 80/620 50 x nail Ø4,0 x 40	Rothofixing 80/620 25 x nail Ø4,0 x 75
$k_{hd;test}$	[kN/mm]	12	27	16
$n \cdot K_{ser}$	[kN/mm]	32,00	86,98	43,49
$n \cdot K_{ser} / k_{hd;test}$	[-]	**2,62**	**3,24**	**2,65**

table 6: Results of simplified calculation of the hold-down stiffness value.

3.4.2 Explanation of deviation

The difference between the test-based values for the hold-down stiffness, and the calculated stiffness values, as shown in table 6, is the result of an overestimation of the hold-down stiffness. The issues that contribute to a less stiff behaviour of the hold-down in reality, will be presented in this paragraph.

Elastic strain in the steel and timber cross section

The first and most obvious reason for the overestimation of the stiffness is the assumption of an inextensible steel and timber cross-section. In figure 40, the simplified hold-down system is shown. The test of Rothofixing WHT 80/620 hold-down with 50 nails Ø5,0 x 50, was used to calculate n · K_{ser} = 86,98 · 10^3 N/mm. The cross-sectional area of the steel hold-down anchor measures 80,0 · 3,0 = 240,0 mm^2 in general, and (80,0 + 83,0) · 3,0 = 489 mm^2 for the foot of the hold-down. The length of the steel strip is (½ · 410,0) + 60 = 265 mm for section l_1 and 150 mm for section l_2.

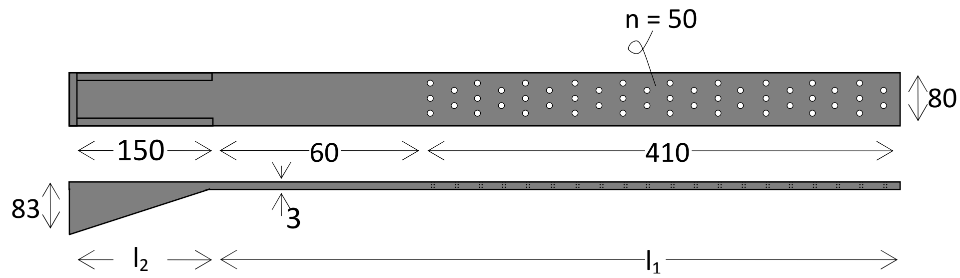


figure 42: Lay-out of the Rothofixing WHT 80/620 (dimensions are out of scale)

The stiffness of the steel hold-down can be calculated to be:

$$\frac{1}{k_{steel}} = \frac{1}{\frac{E \cdot A}{l_1}} + \frac{1}{\frac{E \cdot A}{l_2}} = \frac{1}{\frac{210.000 \cdot 240}{265}} + \frac{1}{\frac{210.000 \cdot 489}{150}}$$

$$k_{steel} = \frac{1}{\frac{1}{\frac{210.000 \cdot 240}{265}} + \frac{1}{\frac{210.000 \cdot 489}{150}}} = 148{,}84 \cdot 10^3 \; N/mm$$

The stiffness of the steel-cross section is 1,7 times as stiff, as all fasteners together. (Remind: n · K_{ser} = 86,98 · 10^3 N/mm.) In the same manner, the stiffness of the timber cross-section can be calculated. This stiffness measures 879,14 · 10^3 N/mm, which is 10,2 times higher compared with the fasteners stiffness. These calculations state that the steel cannot be assumed to be inextensible, compared to the combined stiffness of the fasteners together. The elongation of the steel parts is reason for an error in the calculation of the hold-down stiffness.

An estimation of the error is made for three of the tests. In the figure below is explained how the hold-down anchor is schematized in the more advanced analysis. In this way the stiffness of the steel and timber cross-section can be taken into account in the calculation of the hold-down stiffness. Except for using matrices, the calculation could not be made in an analytical way. Therefore, the configurations of three tests were modelled in a structural analysis program, this program was capable to calculate with spring elements. The fasteners were modelled as spring elements with the transverse stiffness being similar to the fastener slip-modulus K_{ser}.

Using the more detailed model (figure 43, next page) an estimation of the stiffness of the hold-down could be made, including the elongation of steel and timber. This new estimation will be denoted as $k_{hd;FEM}$. For F, and u, see figure 43.

$$k_{hd;FEM} = \frac{F}{u} \tag{3.4}$$

Eccentricities in the load on the bar elements were prevented by applying a lateral roll-support to all nodes. The nodes were only free to move in the direction of the deflection u. As mentioned above, three tests were analysed using the more detailed approach. The results are given in the table below.

Test:	(2)	(1.1)	(1.8)
hold-down & configuration :	Simpson Strong-Tie HTT16 18 x nail Ø4,1 x 89	Rothofixing 80/620 50 x nail Ø4,0 x 40	Rothofixing 80/620 25 x nail Ø4,0 x 75
$k_{hd;test}$ [kN/mm]	12	27	16
$k_{hd;FEM}$ [kN/mm]	27,58	57,60	33,84
$k_{hd;FEM}$ / $k_{hd;test}$ [-]	**2,26**	**2,15**	**2,06**

table 7: Results of a more detailed analysis, estimation of the error in stiffness value.

Remember from table 6 :

$n \cdot K_{ser}$ / $k_{hd;test}$ [-]	2,62	3,24	2,65

The result of the more detailed approach is a smaller difference between the calculated stiffness value and the stiffness value determined from tests. Elongation of the steel and timber cross-section is taken into account.

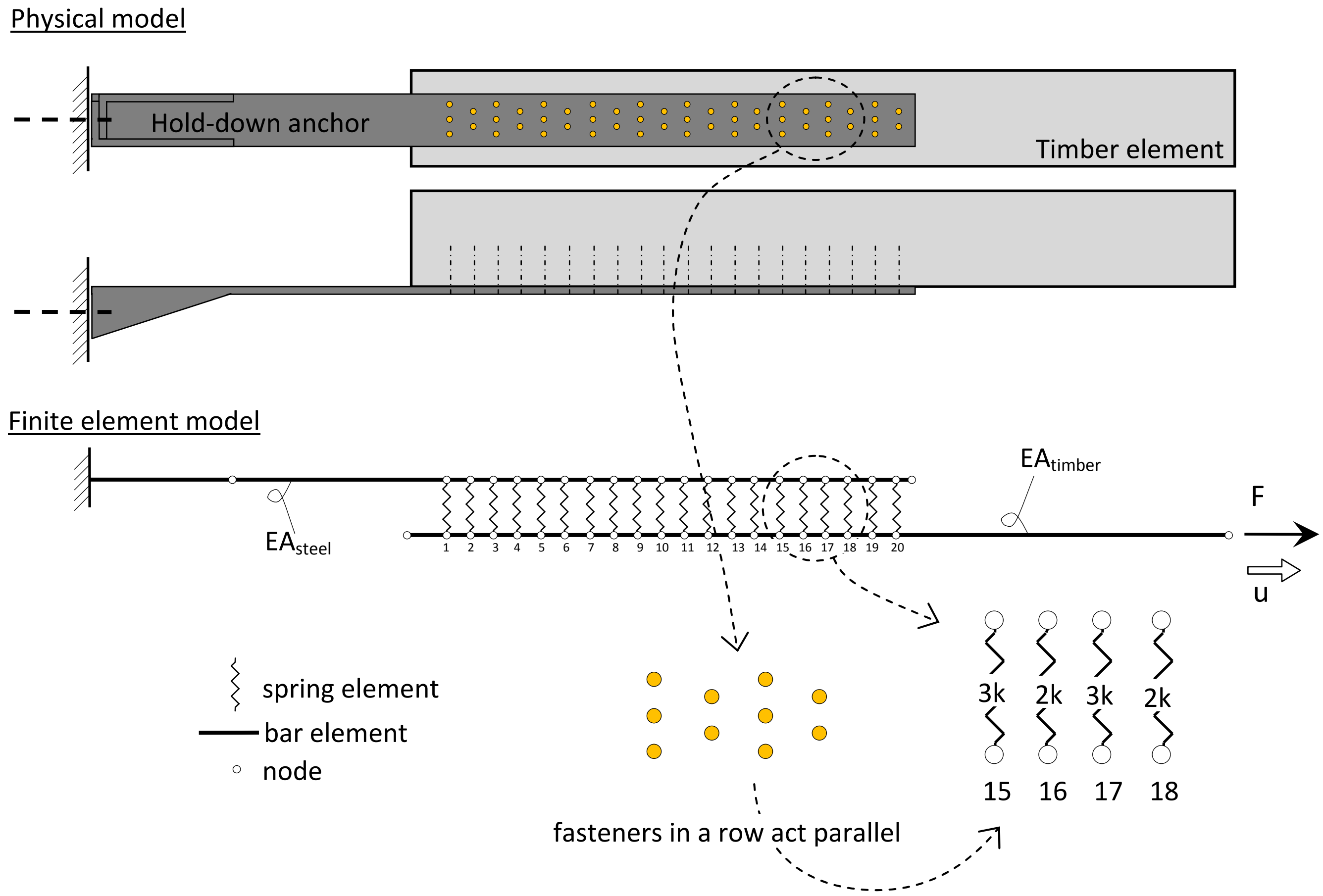


figure 43: Representation of the hold-down anchorage for purpose of structural analysis (configuration of test (1.1)).

The analysis of the detailed modelling approach showed some additional results. These will be discussed in the next part of this paragraph. Thereafter will be discussed how the observed difference between calculations and tests can be limited even more.

Fastener force distribution

The results of the more detailed analysis provided insight into the force distribution between the fasteners. For three tests, the ratio between the actual load taken by the fastener ($F_{f,Ed}$), and the average load per fastener

($\bar{F}_{f,Ed}$) is calculated. In figure 44 this ratio is shown for each separate fastener row (next page). In test (1.1), 50 fasteners were divided over 20 rows. In test (1.8), 25 fasteners were divided over 9 rows. In test (2), 18 fasteners were divided over 9 rows. In figure 44 the position of the fasteners is shown on the horizontal axis.

The shown force distribution is valid in the elastic part of the load-displacement diaphragm of the hold-down. In the detailed hold-down model, the load was increased from 10,00 kN up to the load where the design strength $F_{f,Rd}$, of one of the fasteners was exceeded.

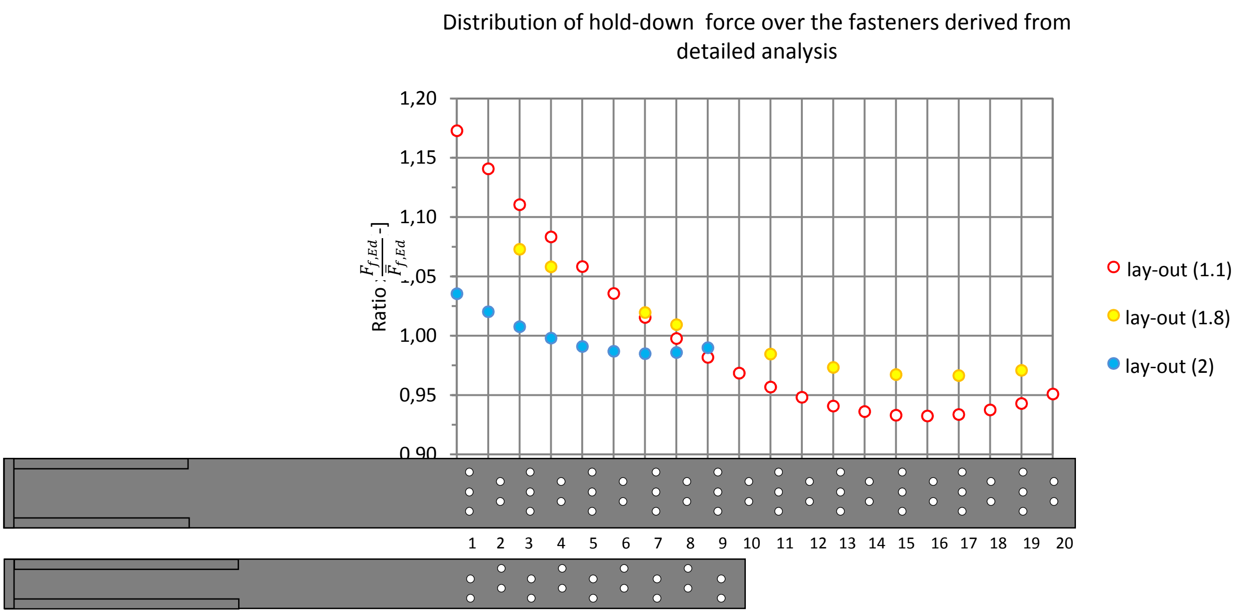


figure 44: Elastic force distribution between the fasteners. Between brackets is indicated from which test the layout was used in the detailed analysis. A row of 2 or 3 fasteners, is modelled together as one spring, and is indicated using the numbers 1..20.(Rothofixing 80/620 on top, Simpson HTT16 below)

Elastic limit for the hold-down strength

By limiting the fastener strength to the code-based design strength $F_{f,Rd}$, it was possible to approximate the hold-down load-level, which would force the fasteners to exceed the design strength. When applying a higher load to the hold-down, it can be assumed that the fasteners will start to deform plastically. As was shown in chapter 0, a fastener will display elastic behaviour until the design strength. The elastic limit for the hold-down strength will be denoted as $F_{hd;Rd}$.

Test:		(2)	(1.1)	(1.8)
hold-down & configuration :		Simpson Strong-Tie HTT16 18 x nail Ø4,1 x 89	Rothofixing 80/620 50 x nail Ø4,0 x 40	Rothofixing 80/620 25 x nail Ø4,0 x 75
F_{04} (40% F_{max} in test)	[kN]	13	41	39
$F_{f,Rd}$	[kN]	1,48 · 0,9 / 1,3 = 1,02	1,83 · 0,9 / 1,3 = 1,27	2,50 · 0,9 / 1,3 = 1,73
n · $F_{f,Rd}$	[kN]	18 · 1,02 = 18,44	50 · 1,27 = 63,35	25 · 1,73 = 43,27
$F_{hd;Rd}$ (elastic limit in detailed analysis)	[kN]	**17,80**	**54,00**	**40,00**

table 8: Results of a more detailed analysis.

From the table above can be concluded that the test-based stiffness is determined on a load-level (0,4·F_{max}) which according to the detailed analysis will be below design capacity ($F_{hd,Rd}$). However, this does not have to imply that the load-displacement relation can be characterized by elastic behaviour. Taking into account, the withdrawal effect in the design strength of the fastener, requires a certain amount of (plastic) deformation. When the design strength $F_{f,Rd}$ of the fasteners is utilized for every fastener, the hold-down will be loaded above the elastic limit (n · $F_{f,Rd}$). For stiffness purposes, it can be advised to apply a disproportionate strong hold-down anchor.

Hole clearance[4]

The second effect that will contribute to a less stiff behaviour of the hold-down anchorage in the tests, is slip of the fasteners in the drilled holes. When the diameter of the fastener is smaller than the hole diameter the hold-down will move a little before being loaded significantly. In reality, a fastener will not be placed exactly in the centre of the hole. When the hold-down is loaded, eventually all fasteners will touch the steel. Therefore, an extra reduction in the stiffness of the hold-down has to be calculated with.

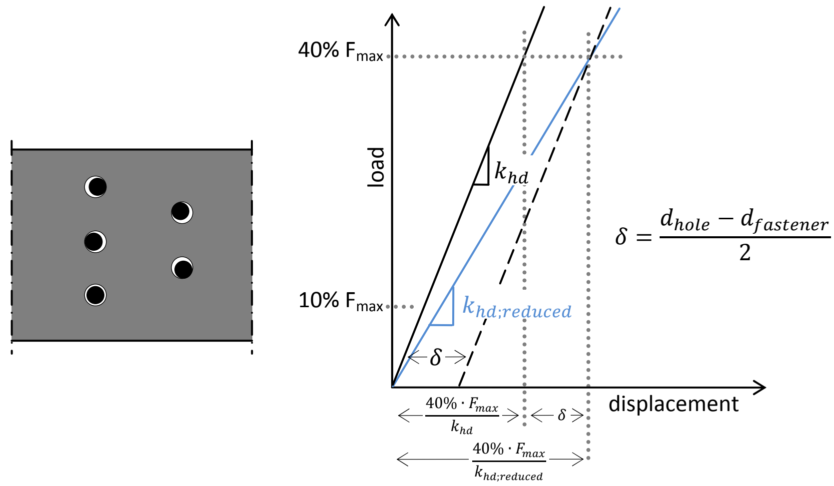


figure 45: Hole clearance, and calculation of reduced stiffness

From the Simpson HTT16 and the Rothofixing 80/620 documentation could be found that the hole diameter equals 5,00 mm. Because in tests (1.1) to (1.10) fasteners were used with a diameter of 4,00 mm, an initial slip 0,50 mm has to be distracted from the stiffness as an average slip for all fasteners. k_{hd} is calculated with units: kN/mm. The reduced stiffness value will be:

$$k_{hd;reduced} = \frac{0{,}4 \cdot F_{max}}{\left(\frac{0{,}4 \cdot F_{max}}{k_{hd}}\right) + \left(\frac{d_{hole} - d_{fastener}}{2}\right)} \qquad 3.5$$

This will lower the difference between the results from tests and calculation as following:

Test:	(2)	(1.1)	(1.8)
hold-down & configuration :	Simpson Strong-Tie HTT16 18 x nail Ø4,1 x 89	Rothofixing 80/620 50 x nail Ø4,0 x 40	Rothofixing 80/620 25 x nail Ø4,0 x 75
$k_{hd;test}$ [kN/mm]	12	27	16
$k_{hd;FEM}$ [kN/mm]	27,58	57,60	33,84
$0{,}4 \cdot F_{max}$ [kN]	13	41	39
d_{hole} [mm]	5,0	5,0	5,0
$d_{fastener}$ [mm]	4,1	4,0	4,0
$k_{hd;FEM;reduced}$ [kN/mm]	14,13	33,83	23,62
$k_{hd;FEM;reduced}$ / $k_{hd;test}$ [-]	**1,16**	**1,26**	**1,44**

table 9: Results of a more detailed analysis, estimation of the error in stiffness value.

Remember from table 7:

$k_{hd;FEM}$ / $k_{hd;test}$ [-]	**2,26**	**2,15**	**2,06**

For tests (1.11) to (1.15) no reduction because of hole-clearance need to be applied. The screws (Ø5,0) fitted exactly to the holes of the hold-down. The agreement between the calculated value and test result for the hold-

[4] hole clearance = gatspeling

down stiffness in these tests proves that the reduction because of hole-clearance is a real issue, and is taken into account with satisfying accuracy in the proposed analytical method.

figure 46: Screw head with underhead to fill the hole clearance are delivered by hold-down manufacturers.

Origins of the equations for K_{ser}

A difference between the test-based value for the hold-down stiffness, and the calculation-based value for the hold-down stiffness will always remain. Most important reason is the use of K_{ser} in the calculations. Originally the slip-modulus K_{ser} is determined based on timber-to-timber connections. Assumptions in the derivation of the equation for K_{ser} are the governing failure mode, and the ratio between the embedment strengths of both elements (Blaß & Ehlbeck, 1993). These assumptions are valid for two identical timber elements in a connection with failure mode (f) as shown in figure 47. In the figure, immediately can be seen how the assumptions differ from the actual lay-out of the steel-to-timber connection.

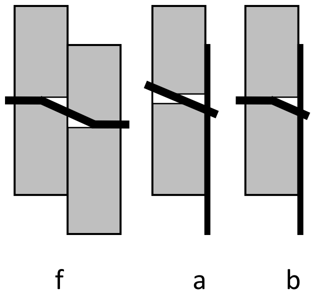


figure 47: Failure mode assumed in equations for K_{ser} (f), and actual failure modes in a (thin) steel-to-timber connection (a) & (b). (EC 5)

Because the hold-downs can be classified as thin steel-plates, no clamping of the fastener into the steel can take place. Although a plastic hinge can possibly develop as shown for failure mode (b), the development of two plastic hinges will be hardly possible. Additionally, the steel will have an infinite embedment strength compared with the timber. For this type of connections, Eurocode 5 simply states to multiply the slip-modulus K_{ser} by 2, and to take the mean density ρ_m as the density of the timber element only. In this way a correction is made. See also page 26. However due to the difference between failure mode f, and a or b, the stiffness of the sheathing-to-timber connection K_{ser} is estimated a little bit too high.

3.5 Improved calculation method

On page 41 an approximation of the hold-down stiffness was made to show the influence of the timber and steel on the system as a whole. An illustration is shown below to explain the calculation that was only explained by text on page 41. In this paragraph an improved method to estimate the hold-down stiffness will be proposed.

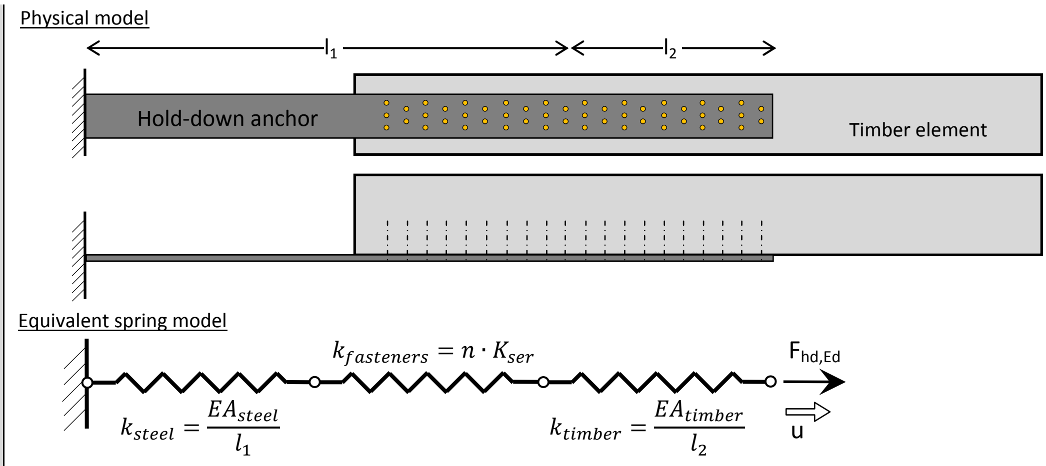


figure 48: Improved method to determine the hold-down stiffness,

In which $F_{hd,Ed}$ is the actual force on the hold-down. In the calculation of the hold-down stiffness for comparison with the test-results, $F_{hd,Ed}$ was equal to 40% F_{max}. The equivalent stiffness of the system can be calculated taking into account three components. The stiffness of the steel cross-section, the stiffness of the timber cross-section, and the combined stiffness of all fasteners together. These have to be summed up as can be seen from equation 3.6.

$$k_{equivalent} = \frac{1}{\frac{1}{k_{steel}} + \frac{1}{k_{fasteners}} + \frac{1}{k_{timber}}} \qquad 3.6$$

$$k_{equivalent} = \frac{F_{hd,Ed}}{u} = k_{hd} \qquad 3.7$$

If the fastener holes are larger compared with the fastener diameter, the stiffness of the hold-down has to be reduced, relative to the level of load, as following:

$$k_{hd;reduced} = \frac{\mathrm{F_{hd,Ed}}}{\left(\frac{\mathrm{F_{hd,Ed}}}{k_{hd}}\right) + \left(\frac{d_{hole} - d_{fastener}}{2}\right)} \qquad 3.8$$

A comparison was made between the results of this improved method and the test results. This comparison can be found in Appendix IV, page 135. For a more easy way of comparison, see the figure below.

Results

In the bar-chart below the difference between the calculated and test-based value for the hold-down stiffness is shown for each test.

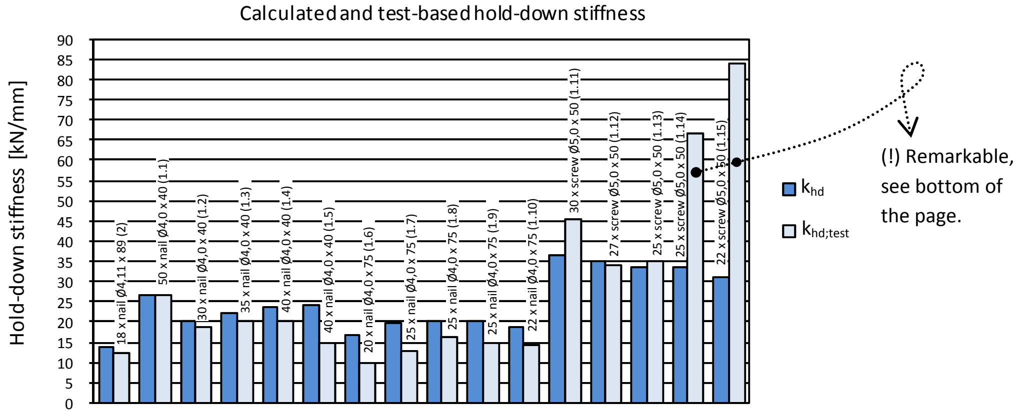


figure 49: Results of the improved method, comparison between calculated and test-based value.

From the calculations can be concluded that on average $k_{hd} = 1{,}12 \cdot k_{hd;test}$. For the three tests, treated in the paragraphs before, comparison is made in the table below.

Test:	(2)	(1.1)	(1.8)
hold-down & configuration :	Simpson Strong-Tie HTT16 18 x nail Ø4,1 x 89	Rothofixing 80/620 50 x nail Ø4,0 x 40	Rothofixing 80/620 25 x nail Ø4,0 x 75
$k_{hd;test}$ [kN/mm]	12	27	16
k_{hd} [kN/mm]	13,99	26,70	20,19
k_{hd} / $k_{hd;test}$ [-]	**1,14**	**1,00**	**1,23**

table 10: Results of the improved method, comparison between calculated and test-based value for three tests.

In figure 49 remarkable results are visible for tests (1.14) and (1.15). From the test report the following could be seen. In comparison with tests (1.12) and (1.13) relatively low values for the ultimate load F_{max} were reached in tests (1.14) and (1.15). Consequently 0,4 · F_{max} was proportionally lower. The deformation however, was disproportionately lower. What the explanation is for the lower ultimate load in tests (1.14) and (1.15) is unknown. When the stiffness value $k_{hd;test}$ for tests (1.14) and (1.15) is calculated based on 0,4 · F_{max} and 0,1 · F_{max} belonging to test (1.13), values, for $k_{hd;test}$, of similar magnitude would have been found as are displayed for tests (1.12) and (1.13). No definitive conclusion can be drawn, possibly an unknown issue in the test-setup caused the higher stiffness values.

3.6 Conclusion

In this chapter the stiffness of the hold-down anchorage has been studied. An answer to the research question is drawn in this paragraph.

Research question:

> 3. *Is it possible to predict the hold-down stiffness with use of an analytical calculation method?*
>   a. *How do the results of this calculation method agree with results of experimental testing?*

A comparison was made between analytical calculations and load-displacement data found from literature. Based on this comparison, the following answer to the research question can be made:

Conclusion 3.:

> *It is possible to predict the hold-down stiffness with use of an analytical calculation method. This calculation method takes into account the stiffness of the steel and timber section, the stiffness of the fasteners, hole-clearance, and the actual load-level. For further guidance reference is made to page 46.*

Conclusion 3.a.:

> *Reasonable agreement was proven between the results of experimental testing found in literature, and the result of analytical calculation. The difference between the calculated values for the hold-down stiffness, using the fastener-slip-modulus K_{ser}, and the stiffness values as determined from the load-displacement data of 16 tests, on average, can be expressed with use of the following relation: $k_{hd}/k_{hd;test} = 1{,}12 \pm 23\%$.*

An additional conclusion can be given:

> *From the detailed analysis can be concluded that the test-based stiffness is determined on a load-level that is characterized by elastic behaviour. However, when the design strength of the fasteners as given from the manufacturers’ documentation is utilized, the hold-down will be loaded above the elastic limit. This can be reason to design the hold-down anchor for stiffness purposes. In such a case the hold-down is more strong than required from strength perspective.*

3.7 Recommendations

As a result of this chapter, a recommendation can be given:

- It can be useful to perform experimental testing on hold-downs with different lay-out. Take for instance hold-down solutions with a larger cross-sectional area of the steel, or hold-down configurations with less but strong fasteners. The magnitude of the tensile stresses in the steel, or the number of fasteners can be of influence on the difference that was found.

4 Racking stiffness of the timber-frame element

The stiffness of the timber-frame shear-wall is the main subject of research in this masters' thesis, as explained in the introduction in chapter 0. Insight in the calculation of the timber-frame shear-wall racking stiffness, was shown to be essentially to predict the deformations (SLS) and force distribution (ULS) in multi-storey timber-frame structures. Therefore, the following research question was proposed:

> 4. *Is it possible to derive an analytical calculation method to calculate the timber-frame shear-wall racking stiffness?*
>   a. *How does the calculation method look like?*
>   - *Which contributions in the deformation have to be taken into account?*
>   - *What is the share of each contribution in the total resulting racking deformation?*
>   - *Which parameters and equations have to be used?*
>
>   b. *Are the calculated racking stiffness values useful in structural design?*
>   - *Do the calculation results agree with test-results?*
>   - *For which purpose is the resulting racking stiffness to be used?*

An answer to these questions will be prepared in this chapter. First, some general explanation is given about the calculation methods in Eurocode 5, and the standard for racking tests on timber-frame shear-walls. Thereafter a test-based value for the racking stiffness will be determined with use of the load-displacement data found from literature, in paragraph 4.1. In paragraph 4.2 explanation will be given about the several contributions to the racking deformation of the timber-frame shear-wall, equations will be derived to take these contributions into account in the analytical calculation method. In paragraph 4.3 will be explained which deformation limits have to be regarded in the design of a timber-frame shear-wall. In paragraph 4.4 a comparison will be made between the analytically calculated, and test-based values for the racking stiffness. The work in the first three paragraphs will be the basis for the conclusions in paragraph 4.5. These conclusions will be the answer to the research question shown above. After all some recommendations are given in paragraph 4.7. The chapter is finished with 4.8, in which the proposed analytical calculation method is shown.

4.1 Test-based racking stiffness

Although in Eurocode 5 no method is given to determine the stiffness of a timber-frame diaphragm with use of a calculation, reference is made to NEN-EN 594: *'Timber structures - Test methods - Racking strength and stiffness of timber frame wall panels'*:

> *'(4)P The racking resistance of a wall shall be determined either by test according to EN 594 or calculations, employing appropriate analytical methods or design models.'*
>
> (*NEN-EN 1995-1-1 §9.2.4.1*)

In this standard the general principles for the experimental determination of racking-strength as well as racking-stiffness of the timber-frame shear-walls are given. Consequently, NEN-EN 594 can be used to determine the racking stiffness of the timber-frame shear-wall with use of test results. From different sources in literature, results of experimental testing on timber-frame shear-walls were found. The load-displacement graphs of these tests were presented in the literature review. Different test procedures, and test set-ups are used. In some of the reports, values for the shear-wall stiffness are calculated, however, different approaches were used among the tests. Therefore NEN-EN 594 is used to determine a new racking stiffness value for all shear-wall tests based on the load-displacement data.

4.1.1 Contents of NEN-EN 594

In the standard NEN-EN 594 is stated how to perform, analyse, and report the experimental testing of timber-frame shear-walls for the determination of the racking strength and stiffness. Relevant information for the determination of racking stiffness is shown below:

The test results shall contain:

a) racking stiffness of the panel, calculated from the equation

$$R = \left[\frac{F_4 - F_2}{v_4 - v_2}\right] \text{ [N/mm]} \qquad 4.1$$

where: F_2 *is the racking load of* $0{,}2 \cdot F_{max}$ *[N]*
F_4 *is the racking load of* $0{,}4 \cdot F_{max}$ *[N]*
v_2 *and* v_4 *is the deformation [mm]*

(NEN-EN 594:2011)

No reference is made in Eurocode 5 to the shear-wall racking stiffness value *R*, as calculated according to NEN-EN 594. In paragraph 4.4 will be explained what the meaning is of this stiffness value.

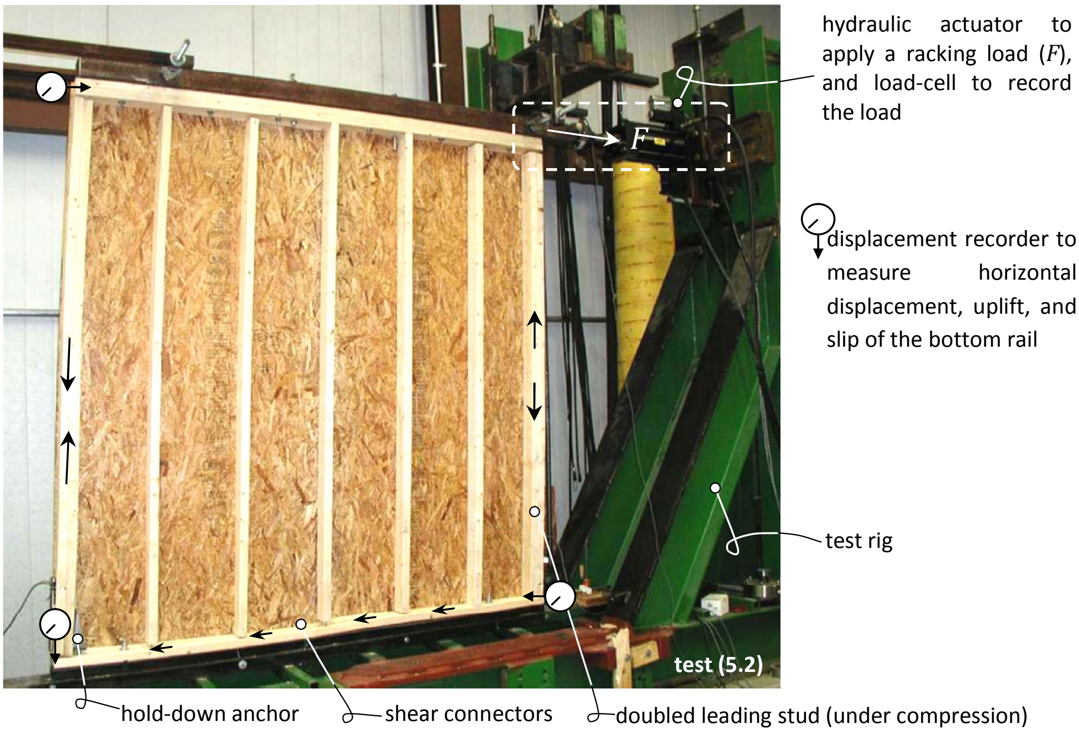


figure 50: Illustrative test-setup for timber-frame shear-wall racking tests. (NAHB Research Center Inc., 2005)

4.1.2 Selection of tests

In the literature review a lot of shear-wall tests were presented. The load-displacement graphs of 51 tests were presented. These data were received from 15 test-series, comprising 113 specimen. However, not all tests are useful for analysis. Some of the tests were reported with a minimum of information regarding the test-setup and the boundary conditions. The presence or absence of certain connectors was not mentioned for example. An analytical calculation to estimate the shear-wall stiffness was not possible for tests without clear information in the test-report, consequently these tests are omitted in this analysis. The load-displacement data for the tests which will be treated in this chapter are appended in Appendix V, page 137.

4.1.3 Determining the shear-wall racking-stiffness

With use of equation 4.1 a value for the racking stiffness R is calculated. The required input for the calculation, and the results are shown in the calculation sheet in Appendix VI, on page 139159. F_{max}, F_{04}, F_{02}, v_{04}, and v_{02} are determined based on the load-displacement data. The experimentally determined racking stiffness R will be compared with the analytically determined shear-wall stiffness. Furthermore, the use and applicability of the stiffness values with respect to the serviceability and ultimate limit state, as well as the choice of the analysed tests will be commented on.

4.2 Calculated racking stiffness

In this paragraph an analytical calculation of the racking stiffness for the given timber-frame shear-wall tests will be shown. The racking stiffness of the shear-wall is the ratio between the load on the shear-wall ($F_{v,Ed}$), and the horizontal deformation (u) of the timber-frame wall element. A number of different contributions to this deformation can be distinguished. The contributions on element level and on the single panel scale are shown below, including a description of the mechanism causing the deformation.

Panel level:
u_f: rotation of the sheathing due to slip of the fasteners along the perimeter of the panel
u_G: shear deformation of the panel sheathing

Element level:
u_{hd}: deformation in the hold-down anchorage cause uplift of the tensile loaded stud
u_c: compressive stresses perpendicular to grain of the bottom rail crush the bottom rail cross-section
u_v: slip of the shear connectors fixing the bottom rail to the test rig (or substructure)
u_{str}: compressive and tensile strain in the vertical studs on the element edges

The total deformation of the timber-frame shear-wall panel is the summation of all individual contributions:

$$u = \sum u_i = u_f + u_G + u_{hd} + u_c + u_v + u_{str} \quad (4.2)$$

The racking stiffness of the timber-frame shear-wall panel can be calculated to be:

$$R = \frac{F_{v,Ed}}{u} \quad (4.3)$$

For the analytical calculation of R for the tested wall lay-outs, which were presented in the paragraph before, use will be made of the 40% F_{max} load-level F_{04}: $F_{v,Ed} = F_{04}$. This is the same load-level as is used to determine R according to NEN-EN 594.

Information given in de test reports and papers, is often specific to the individual test. This information is taken into account in the calculation. For example: with use of the method presented in paragraph 3.5, it was possible to estimate realistic values for the hold-down anchorage stiffness, based on the information found in the reports.

In the analytical determination of the timber-frame shear-wall racking stiffness R, a force distribution in the panel will be assumed as was presented in the literature review. A plastic force distribution, method A from Eurocode 5, was proven to give the best agreement with respect to the determination of racking strength. This was shown from experimental research as presented by Källsner and Girhammar in 2009 (Källsner & Girhammar, 2009).

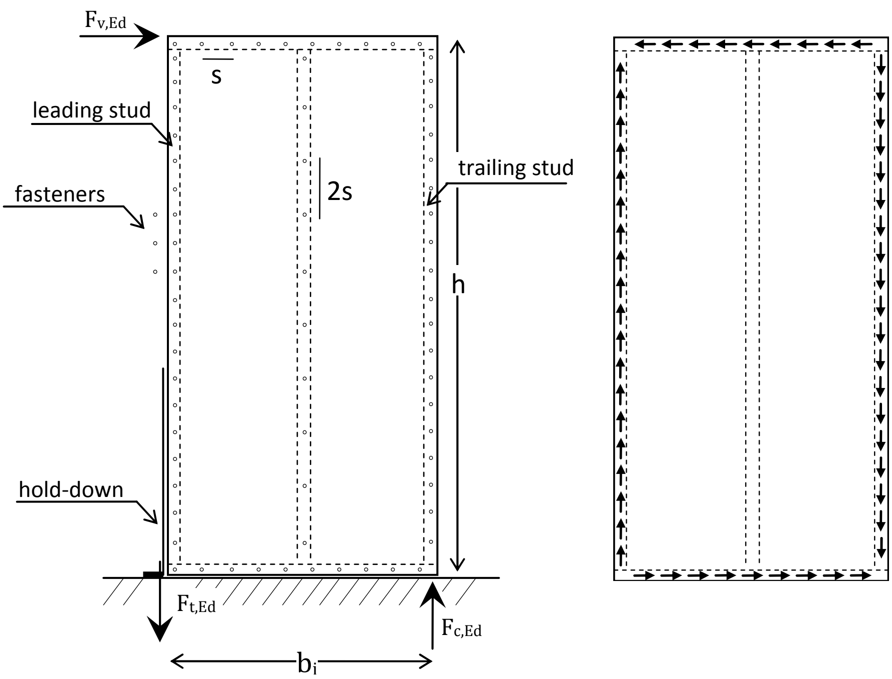


figure 51: Timber-frame shear-wall panel with fastener spacing (s). Forces on fasteners exerted by the sheathing, according to the plastic lower bound method, in Eurocode 5 given as method A.

In the following paragraphs the calculation of each contribution to the horizontal racking deformation will be explained. The calculations are made with use of an Excel spreadsheet that is appended to this report in Appendix VII on page 149.

The calculation in the spreadsheet consist of 7 steps:

Step 1): Load-displacement data resulting from tests, as found in literature.

⇩

Step 2): Determination of test-based shear-wall stiffness R_{test} using NEN-EN594.

⇩

Step 3): Overview and calculation of the timber-frame shear-wall parameters.

⇩

Step 4): Calculation of all contributions to the shear-wall deformation u_{Σ} on basis of the 40% Fmax load-level.

⇩

Step 5): Calculation of the shear-wall stiffness R_{Σ}.

⇩

Step 6): Comparison between R_{test} and R_{Σ}, and v_{04} and $u_{\Sigma;04}$.

⇩

Step 7): Calculation of the shear-wall strength $F_{v,Rd}$ and comparison with 40% F_{max}.

figure 52: Lay-out of the calculation / spreadsheet in Appendix VII.

In step 1, the load-displacement data from shear-wall tests are shown, which were found in literature. Reference is made to the original sources. The applied sheathing material and thickness are mentioned as well as the fastener configuration. This makes simple identification of the tests and distinction between the tests possible. The anchorage conditions are stated as well as other test specific remarks.

Test	Source
(1)	(Yasumura & Kawai, 1997)
(2)	(Andreasson, 2000)
(3)	(Salenikovich, 2000)
(4)	(Ni et al., 2010)
(5)	(NAHB Research Center Inc., 2005)
(6)	(Yasumura & Karacabeyli, 2007)
(8)	(Yasumura, 1991)
(10)	(LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002)
(12)	(VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 04-10-2001)
(13)	(VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)
(14)	(Källsner, 1984)
(16)	(Dolan & Toothman, 2003)

The literature can be found in the sources displayed on page 105.

table 11: Test-series and reference to literature.

In step 2, the shear-wall racking stiffness R_{test} is determined with use of NEN-EN 594 based on the 20% and 40% F_{max} load-level with the matching displacements, v_{02} and v_{04}.

In step 3, all parameters of the shear-wall are shown, or calculated, necessary for the analytical calculation of the shear-wall stiffness. In the forthcoming paragraphs of this chapter, the calculation of these parameters will be discussed.

In step 4, of the spreadsheet the different contributions to the racking deformation of the shear-wall are calculated. These are the contributions mentioned on the page before. The values are based on the equations that will be shown in the next paragraphs.

In step 5, the analytical determined deformation at 40% F_{max}, $u_{\Sigma;04}$, is used to calculate the stiffness of the shear-wall R_{Σ}:

$$R_{\Sigma} = \frac{40\%\ F_{max}}{u_{\Sigma;04}} \qquad 4.4$$

In step 6, the analytically determined shear-wall stiffness R_{Σ} is compared with the test-based shear-wall stiffness R_{test}. Also comparison is made between the calculated deformation $u_{\Sigma;04}$, and the deformation resulting from the tests, u_{04}.

In step 7, the design strength of the shear-wall for the ultimate limit state (ULS), $F_{v,Rd}$, is calculated and compared with 40% F_{max}, F_{04}.

The equations that are hidden in the spreadsheet, and the calculations which are the basis of the spreadsheet, will be derived and explained in the next paragraphs. The results will be analysed in paragraph 4.4, and a final conclusion will be drawn in paragraph 4.5. In paragraph 4.8, the suggested analytical method for the calculation of the timber-frame shear-wall racking stiffness will be shown. In paragraph 4.7, recommendations will be given for improvement and further work.

4.2.1 Fastener slip

From the literature review already could be concluded that the fastener characteristics influence the shear-wall properties enormously. A relatively big part of the deformation is expected to be caused by slip of the fasteners. This will result in rotation of the sheathing and a contribution to the horizontal deflection of the panel, as is shown in the figure below.

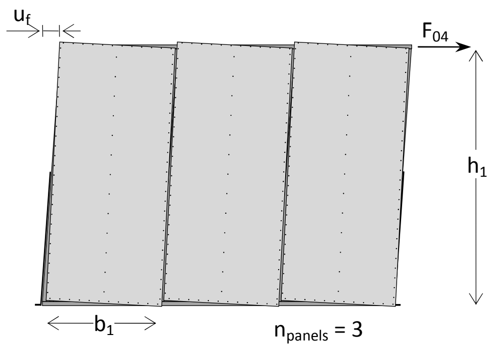


figure 53: Rotation of the sheathing, due to fastener slip, results in horizontal deformation of the shear-wall element.

When the system is considered assuming a constant shear flow along the perimeter of the wall, the slip of the fasteners can be computed with use of the stiffness parameter K_{ser}. Furthermore, the force per fastener is a function of the load on the shear wall (F_{04}), the number and width of the panels ($n_{panels} \cdot b_1$), and the fastener spacing (s). The slip of the fasteners is (in most cases) magnified by the ratio between height and width of the shear wall panels (h_1/b_1). If sheathing (with the same properties) is applied on both sides (faces) of the shear-wall, the deformation u_f may be divided by n_{faces}.

The equation for the contribution of fastener slip in the racking displacement of the timber-frame element can be derived to be:

$$u_f = \frac{2 \cdot F_{04} \cdot s \cdot (1 + \frac{h_1}{b_1})}{n_{panels} \cdot b_1 \cdot K_{ser} \cdot n_{faces}} \qquad 4.5$$

For the six selected tests this equation is used to calculate the contribution of fastener slip in the racking deformation, see Appendix VII (page 149). With use of the figures in this paragraph, the equation for u_f can be derived. u_f is result of the slip of the fasteners along the perimeter of the sheathing.

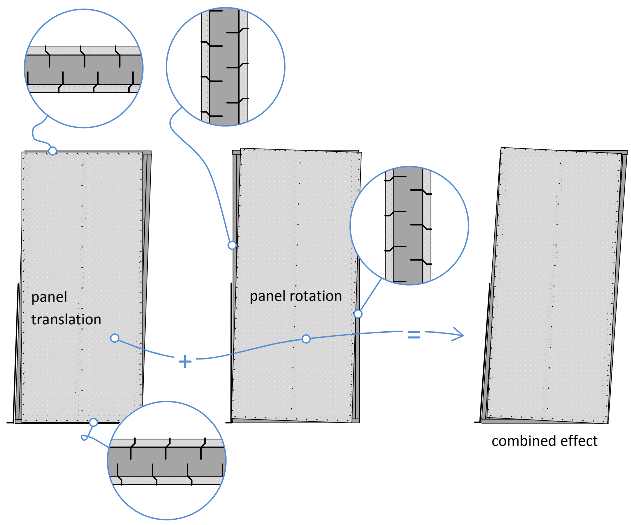


figure 54: Slip of fasteners in horizontal rails and vertical studs will cause sliding and rotation of the panel.

In figure 54 the contribution to the racking deformation of the shear-wall element by slip of the fasteners is divided into two parts. In fact these are not individual mechanisms, but they will develop simultaneously. The choice for considering two contributions was made for reason of providing clear insight. In the figure can be seen that the board will display a horizontal translation because of slip in the top and bottom rail. Horizontal movement of the top-rail can take place because of racking of the frame. Vertical slip of the tensile-loaded stud however, is restrained by the hold-down. Therefore, it is only possible to have slip in the vertical row of fasteners by rotation of the sheathing. Only in this way, the forces along the perimeter of the board will be introduced in the vertical timber framing elements. Consequently, the fastener-slip in the vertical studs will result in a horizontal component because of rotation. Both mechanisms, horizontal translation and the horizontal effect of rotation will result in a combined contribution in the horizontal racking deformation of the panel. This contribution can be calculated with use of equation 4.5 that is derived as following.

First the effect of fastener-slip in the top and bottom rail will be derived. The racking force (F04) on the panel will be divided equally over the number of fasteners in the rail. The number of fasteners is b_1/s. The slip-modulus of the fastener is the relation between the force per fastener and the slip:

the force per fastener: $= F_{04} \cdot \frac{s}{b_1}$

$$u_{f,(B)} = \frac{F_{04} \cdot s}{K_{ser} \cdot b_1}$$

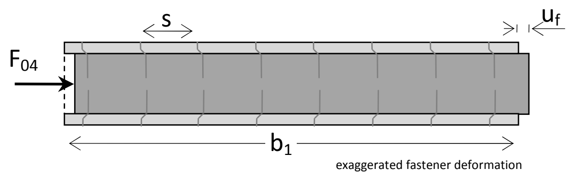


figure 55: Horizontal deformation of the top rail as a result of fastener-slip in the top-rail.

Because this slip will take place in the top and bottom rail, it has to be doubled.

$$u_{f,(H)} = \frac{2 \cdot F_{04} \cdot s}{K_{ser} \cdot b_1}$$

Because of an equal fastener spacing s and equal shear force along the perimeter of the panel, the slip of the fasteners in the vertical studs, $u_{f,(V)}$, is equal to the slip in the top and bottom rail $u_{f,(H)}$. The effect of $u_{f,(V)}$ on the horizontal deformation is magnified by the height to width ratio of the panel. Therefore:

$$u_f = u_{f,(H)} + \frac{h_1}{b_1} \cdot u_{f,(V)}$$

If this relation is expanded, the following equation can be derived:

$$u_f = \frac{2 \cdot F_{04} \cdot s}{K_{ser} \cdot b_1} + \frac{h_1}{b_1} \cdot \frac{2 \cdot F_{04} \cdot s}{K_{ser} \cdot b_1} \qquad 4.6$$

If the shear-wall element is made of n panels, equation 4.6 can be written as:

$$u_f = \frac{2 \cdot F_{04} \cdot s \cdot (1 + \frac{h_1}{b_1})}{n_{panels} \cdot b_1 \cdot K_{ser}}$$

The panels can be sheeted on one or two faces, using the same, or different sheathing material and fastener configuration. If both sides are equal, this will be called a symmetric shear-wall. If only one sides is sheeted, or when different configuration is applied on both sides, this will be called asymmetric. The deformation of a shear-wall sheeted on both sides may be divided by 2:

$$u_f = \frac{2 \cdot F_{04} \cdot s \cdot (1 + \frac{h_1}{b_1})}{n_{panels} \cdot b_1 \cdot K_{ser} \cdot n_{sheets}}$$

For the analytical estimation of the racking stiffness, this equation is used to calculate the contribution of fastener slip in the racking deformation, see Appendix VII (page 149). As explained in chapter 0, the design rules given in Eurocode 5 can be used to calculate a fastener slip-modulus K_{ser} for wood-based panels. For the gypsum paper-board and gypsum fibre-board panels, test-based values for the fastener stiffness k_s were used in the analytical calculation of the shear-wall racking stiffness. The values for K_{ser} and k_s, used in the calculation, can be found in Appendix VIII (page 159). For the asymmetric wall in test (16) with different sheathing and fastener configuration on both sides, the calculation is modified taking into account the different properties.

figure 56: Rotation of the sheathing panels can become quite high at ultimate load. (Ni et al., 2010)

4.2.2 Strain in the hold-down connection

Another contribution to the racking deformation of the shear-wall is the uplift at the tensile stud due to strain in the hold-down anchorage. As explained in paragraph 3.1, a hold-down connector provides resistance to the tensile forces in the tensile loaded edge stud. However, none of the hold-down solution will be of infinite stiffness.

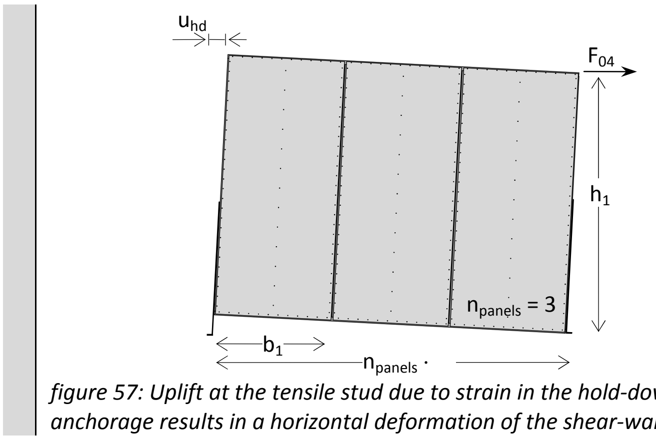


figure 57: Uplift at the tensile stud due to strain in the hold-down anchorage results in a horizontal deformation of the shear-wall element.

In figure 57 is shown how the uplift of the shear-wall panel will result in a contribution to the horizontal deflection of the shear-wall panel. Using the stiffness property K_{hd} of the hold-down, it is possible to quantify this contribution. The following equation can be derived:

$$u_{hd} = \frac{F_{04} \cdot h_1^2}{K_{hd} \cdot (n_{panels} \cdot b_1)^2} \qquad 4.7$$

In case of test (1.2), (8.1) and (8.2), the strain in the hold-down was not taken into account because the load-displacement data for these tests were corrected for sliding and tilting of the shear-wall panel. The uplift at the locations of the hold-down was measured, and the horizontal effect of this uplift was distracted from the horizontal displacement of the top-rail. For the other tests, the strain in the hold-down is taken into account in the calculations. The parameter K_{hd} that needs to be included in the calculations is estimated using the information that was provided in the test-reports. Use is made of the knowledge that was gained in chapter 3. Based on that knowledge, it may be assumed that the estimation of the hold-down stiffness gives a reasonable result. In Appendix IX is shown what type of hold-down was used in the tests, and how the value for the estimated stiffness is calculated (page 149). From test (2.1) the stiffness value of the angle bracket was experimentally determined and given in the test report. For test (14.15) it is assumed that uplift can take place because of compression perpendicular to the grain of the top-rail. In this case, compressive stresses perpendicular to grain in the top-rail are caused at the position of the leading stud. For the remaining tests an estimation of the hold-down stiffness was made.

Equation 4.7 can be derived with use of figure 57. Due to the strain in the hold-down uplift of the panel will occur. For now this uplift will be denoted with Δ_{hd}. The force on the hold-down depends on the racking force on the shear-wall element, and the height to width ratio of the element. The tensile force to be anchored by the hold-down measures:

$$F_{04} \cdot \frac{h_1}{(n_{panels} \cdot b_1)}$$

The uplift Δ_{hd} can be calculated with use of the hold-down stiffness K_{hd}:

$$\Delta_{hd} = \frac{F_{04} \cdot h_1}{K_{hd} \cdot (n_{panels} \cdot b_1)}$$

Because uplift at the tensile stud is of influence on the behaviour of the panel as a whole, the uplift will not be magnified by the height to width ratio of a single panel, but will be lowered by the height to width ratio of the element. Now the contribution of hold-down slip in the racking deformation, u_{hd}, can be derived to be:

$$u_{hd} = \Delta_{hd} \cdot \frac{h_1}{(n_{panels} \cdot b_1)}$$

This will result in:

$$u_{hd} = \frac{F_{04} \cdot h_1^2}{K_{hd} \cdot (n_{panels} \cdot b_1)^2}$$

The equation derived in this paragraph can also be used in the calculation of the effect of compression perpendicular to the grain of the bottom rail, beneath the compressed stud. In this case, K_{hd} will be replaced by K_{pr}. In the next paragraph will be explained how K_{pr} can be determined.

4.2.3 Compression perpendicular to grain

Horizontal deformation of the timber-frame shear-wall element caused by compression perpendicular to grain can be taken into account similarly to the calculation of the deflection caused by hold-down strain. In this case the foundation stiffness of the bottom rail beneath the compressed stud is taken as stiffness parameter.

$$u_c = \frac{F_{04} \cdot h_1^2}{K_{c,90} \cdot (n_{panels} \cdot b_1)^2} \tag{4.8}$$

In some of the tests double or triple edge studs were used. Therefore, the compressive load could be spread over a larger area. The values for the estimated stiffness $K_{c,90}$, were calculated with use of the information that was found in the literature research. Using the secant stiffness value (1.3) a foundation modulus could be determined.

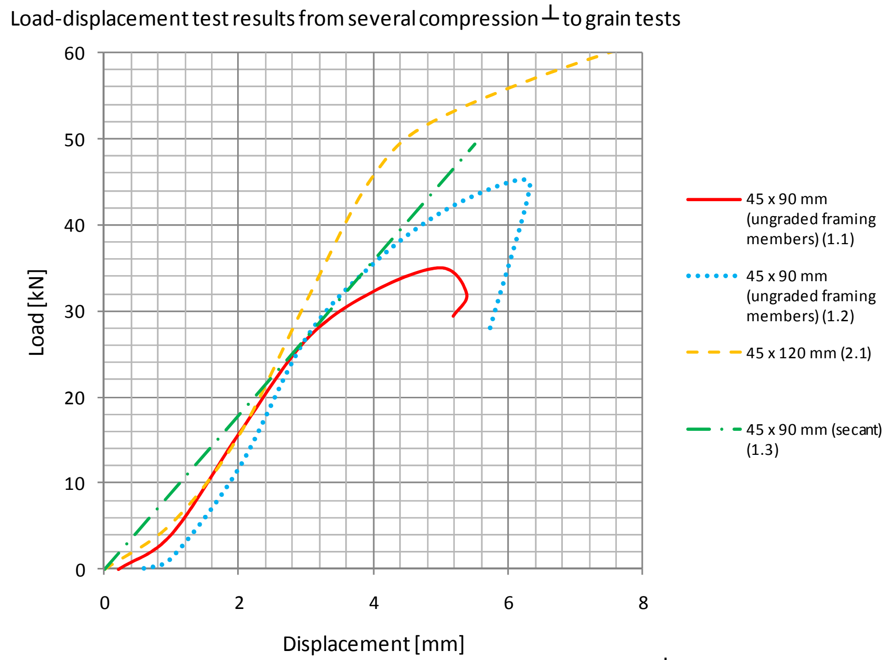


figure 58: Load-displacement test results from several compression ⊥ to grain tests.

The secant stiffness in test (3.1) can be calculated to be:

$$\frac{4{,}9 \cdot 10^1\ kN}{5{,}5 \cdot 10^1\ mm} = 8{,}9 \cdot 10^3\ \mathrm{N/mm} \tag{4.9}$$

In the test a stud was used measuring 45 x 90 mm. Consequently, the effective area according to NEN-EN 1995-1-1:2005 measures:

$$A = (45 + 30) \cdot\ 90\ = 6750\ \mathrm{mm}^2$$

See also the next page.

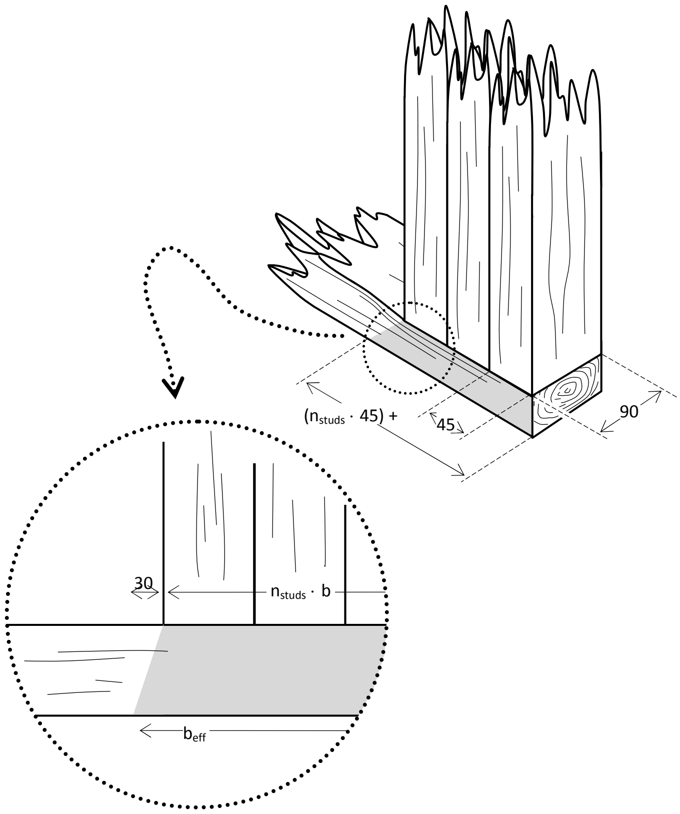


figure 59: Effective width of the compressed zone is larger due to the effect of load spreading, b_{eff} = b +30 mm.

Design rules to quantify the effect of load spreading are given in NEN-EN 1995-1-1. The foundation modulus can be calculated as following:

$$k_{c,90} = \frac{8{,}9 \cdot 10^3}{6750} = 1{,}3\ \mathrm{N/mm^3}$$

For the selected tests, the stiffness parameter $K_{c,90}$ is required in the calculations. This stiffness parameter can be calculated with use of the of the following equation:

$$K_{c,90}\ [\mathrm{N/mm}] = 1{,}3 \cdot ((\mathrm{n_{studs}} \cdot \mathrm{b_2}) + 30) \cdot \mathrm{h_2} \qquad 4.10$$

in which n_{studs} is the number of studs used to transfer the compressive force on the panel edge, and b_2 and h_2 are the cross-sectional width and height of the stud.

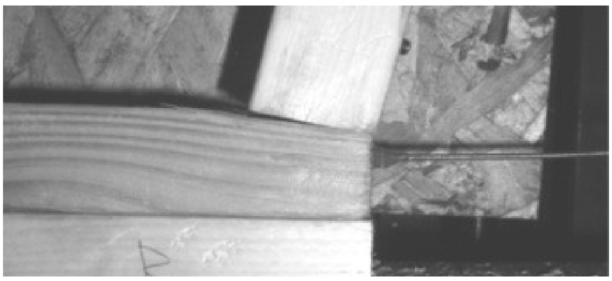


figure 60: Compression perpendicular to the grain at ultimate load in a shear-wall test. (NAHB Research Center, 2003)

4.2.4 Shear deformation of the sheathing

As is already shown earlier, in figure 51 for instance, the timber-frame structure is stabilised by diaphragm action of the shear-walls. Consequently the sheathing on the panel is loaded with a shear force along the perimeter, which causes shear-deformation. The contribution of the shear-deformation into the racking effect will be quantified in this paragraph.

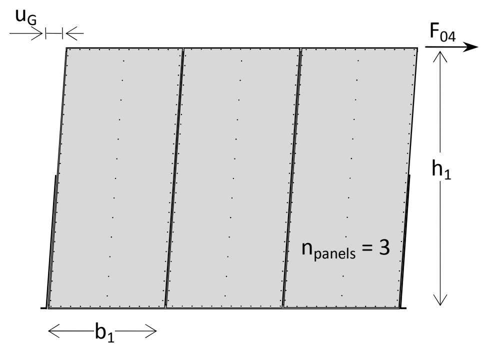


figure 61: Horizontal deformation of the shear-wall element due to shear deformation of the sheathing material.

The horizontal displacement caused by shear-deformation can be calculated with use of the equation below.

$$u_G = \frac{F_{04} \cdot h_1}{n_{panels} \cdot n_{faces} \cdot G_{mean} \cdot b_1 \cdot t} \tag{4.11}$$

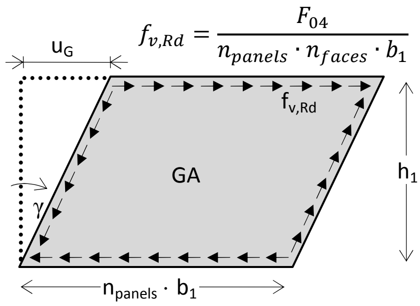


figure 62: Shear contribution to the horizontal racking deformation.

The values for the shear-modulus G used in the calculations are reported below.

Sheathing material:	OSB	Plywood	Particleboard (t = 12 mm)	Gypsum paper board	Gypsum fibre board
Shear-modulus G: [N/mm^2]	1080,00	350,00	960,00	700,00	1600,00

table 12: Shear-modulus G for the sheathing materials used in the tests.

Equation 4.11, will be derived below.

$$u_G = \gamma \cdot h_1 \tag{4.12}$$

$$\gamma = \frac{\tau}{G_{mean}} = \frac{1}{G_{mean}} \cdot \frac{F_{04}}{A}$$

A is the area available for shear, if n_{panels} panels with a width b_1 are used, γ becomes:

$$\gamma = \frac{F_{04}}{G_{mean} \cdot n_{panels} \cdot n_{faces} \cdot b_1 \cdot t} \qquad 4.13$$

in which t is the thickness of the sheathing,

If equation 4.12 and 4.13 are combined this will result in:

$$u_G = \frac{F_{04} \cdot h_1}{G_{mean} \cdot n_{panels} \cdot b_1 \cdot t \cdot n_{faces}}$$

Low shear-modulus G_{mean} for plywood

When table 12 is observed more in detail, a relatively low shear modulus for plywood can be seen. This can be explained partially by the density of the plywood that is a little bit less compared with OSB, particleboard, gypsum paper-board, and gypsum fibre-board. This can be seen from table 1 on page 27. A lower shear modulus can also be seen in case of cross-laminated timber. In the tables of Finnforest Leno® values for G_{mean} are given for several configurations of the cross-laminated products. Depending on the direction of the lamella layers, a shear modulus G_{mean} for the laminated timber plates is supplied, which is on average 450 N/mm^2. The lamellas are timber laths with a strength class C24. In the strength class table in NEN-EN 338, C24 will give a mean shear modulus G_{mean} of 690 N/mm^2. Because of this observation, it can be concluded that the application of wood in a glued and layered product gives a lower shear modulus not only because of different density.

With respect to plywood one of these reasons can be the different structure of the material. No interlocking of particles or strands is present between the glued veneer layers, like in the case of OSB and particleboard. The properties of the applied glue will influence the effect of rotation between layers.

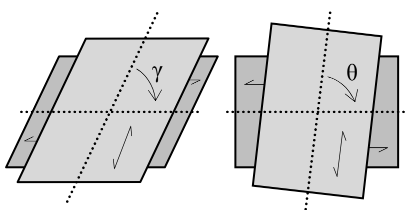


figure 63: Shear and rotation of the veneer layers in plywood.

4.2.5 Deformation due to axial strain in the edge studs

In this paragraph will be explained how the strain in the vertical edge studs will contribute to the horizontal racking deformation of the shear-wall element.

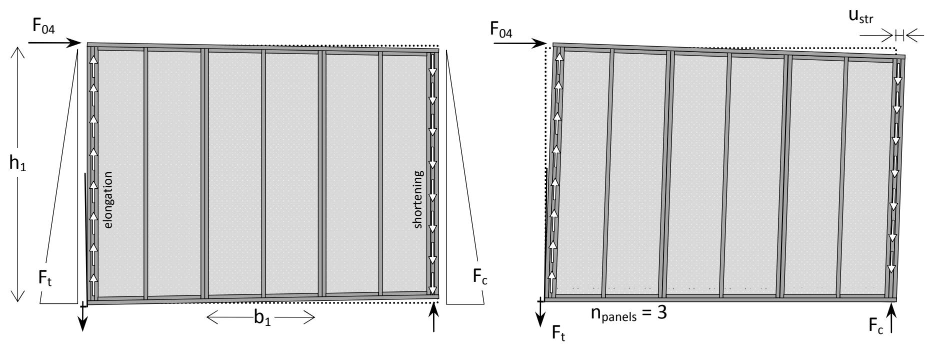


figure 64: Horizontal deformation of the shear-wall element due to elongation and shortening of the vertical edge studs.

Because of the horizontal racking force on the element, a tensile force F_t will develop in the left hand edge stud. This stud is also denoted as the leading stud, because the racking force is exerted on this side of the element. The tensile force F_t is gradually transferred by means of the fasteners which transfer the forces from the sheet to the stud, from top (F_t = 0) to bottom (F_t = F_t). The trailing stud, (in figure 64 on the right-hand side), will be loaded in the same manner by a compressive load F_c.

The horizontal displacement caused by the strain in the studs can be calculated with use of the equation below.

$$u_{str} = \frac{F_{04} \cdot \mathrm{h}_1^3}{E_2 \cdot b_2 \cdot h_2 \cdot n_{studs} \cdot \left(n_{panels} \cdot b_1\right)^2} \qquad 4.14$$

This equation will be derived below:

The force Ft and Fc in the leading and trailing stud are equal and can be calculated observing moment equilibrium:

$$\mathrm{F_t} = \frac{F_{04} \cdot \mathrm{h}_1}{n_{panels} \cdot b_1} \qquad 4.15$$

In which $n_{panels} \cdot b_1$ is the width of the element. The elongation of the stud can be calculated as following:

$$\sigma = \mathrm{E} \cdot \varepsilon$$

$$\frac{\mathrm{F_t}}{\mathrm{b_2} \cdot \mathrm{h_2}} = \mathrm{E_2} \cdot \frac{\Delta\mathrm{l}}{\mathrm{h_1}}$$

This can also be written as:

$$\Delta\mathrm{l} = \frac{\mathrm{F_t} \cdot \mathrm{h_1}}{\mathrm{E_2} \cdot \mathrm{b_2} \cdot \mathrm{h_2}} \qquad 4.16$$

Combining equation 4.15 and 4.16 gives:

$$\Delta\mathrm{l} = \frac{F_{04} \cdot \mathrm{h}_1^2}{\mathrm{E_2} \cdot \mathrm{b_2} \cdot \mathrm{h_2} \cdot \left(n_{panels} \cdot b_1\right)}$$

The elongation and shortening of the studs will be translated in a horizontal deformation *u* because of rotation of as shown in figure 64:

$$\mathrm{u_{str}} = 2 \cdot \Delta\mathrm{l} \cdot \left(\frac{\mathrm{h_1}}{n_{panels} \cdot \mathrm{b_1}}\right)$$

Because the force F_t and F_c are linearly introduced, the elongation of the stud may be divided by 2. If all equations above are summed up, it can be shown that the contribution in the racking deformation due to strain in the leading and trailing stud can be calculated with use of the equation below:

$$u_{str} = \frac{F_{04} \cdot \mathrm{h}_1^3}{E_2 \cdot b_2 \cdot h_2 \cdot \left(n_{panels} \cdot b_1\right)^2} \qquad 4.17$$

4.2.6 Slip between bottom rail and substructure

In the analysed tests, the slip of the bottom rail was either prevented by blocking on the test-rig, or the load-displacement data were corrected for the recorded horizontal slip of the bottom rail. In the latter cases, the horizontal displacement of the bottom rail was measured during testing and distracted from the horizontal racking deformation measured on the top rail. See also figure 50 on page 52. Therefore, this paragraph is not required for the analytical calculation of the racking stiffness for the particular design of the tested walls.

However, to end up this chapter with a useful analytical method for the calculation of the racking stiffness in practical applications, it can be necessary to include the slip of the shear connectors between shear-wall and substructure in the calculation. Therefore, the following equation is proposed.

$$u_v = \frac{F_{04}}{n_{panels} \cdot n_{conn} \cdot K_v} \tag{4.18}$$

In which n_{panels} is the number of panels, n_{conn} is the number of shear connectors per panel, and K_v is the stiffness of the shear connector in the direction of loading.

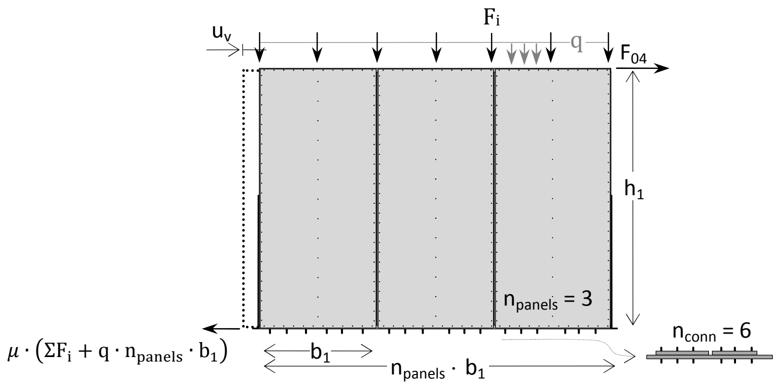


figure 65: Horizontal deflection due to slip between the bottom rail and substructure.

If, for example, bolts are used as a shear connector in a timber-to-timber connection between bottom rail and substructure, reference can be made to Eurocode 5 in which design equations for the slip-modulus K_{ser} are given for connections with bolts.

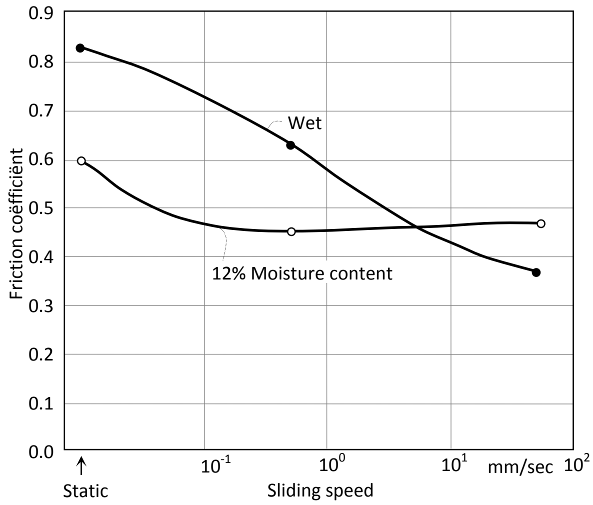
Friction between bottom rail and substructure is present, below a wall loaded by the dead weight of the wall itself and the storeys above. Slip between the bottom rail and substructure only can take place if the racking force F_{04} will exceed this friction capacity. This have to be checked, taking into account using the relevant friction-constant μ, otherwise the racking stiffness of the timber-frame shear-wall will be estimated too low. To prevent slip, the following condition has to be satisfied:

$$\mu \cdot (\Sigma F_i + q \cdot n_{panels} \cdot b_1) \geq F_{04} \tag{4.19}$$

In which ΣF_i is the summation of all vertical concentrated loads on the shear-wall, q is the distributed vertical load on the shear wall, $n \cdot b_1$ is the width of the element, F_{04} is the racking force on the element, and μ is the friction coefficient between two timber surfaces. From literature can be found that for dry timber at moisture content 12%:

$$\mu_{timber-to-timber} \geq 0{,}40 \tag{4.20}$$

See next page.



Wood Science and Technology Vol. 2 (1968) p. 139/152

The Frictional Behaviour of Wood

By W. M. McKenzie and H. Karpovich, Melbourne

Summary

The work described was primarily concerned with determining the more important variables affecting friction between wood and steel, but friction between wood and non-ferrous materials including wood itself was also investigated briefly. Deviations from the classical laws of friction were of interest, the most significant being variation of friction coefficient with sliding speed. With an increase in sliding speed up to 4 m/sec the curves for highly polished steel showed undulations, but with unpolished surfaces there was a monotonic reduction, somewhat greater in wet wood than dry. Other important factors, interacting with sliding speed, were steel roughness and wood moisture content. Effects of load, nominal contact area and fibre direction were minor. The results appear to be adequately explained in terms of adhesion and lubrication.

A world wide selection of species was tested, and it appears that the nature and amount of extractives in most woods is such that they have similar friction coefficients except on very smooth steel, and only a few “greasy” species, have significantly lower coefficients. For most air-dry wood in contact with unpolished steel, the coefficient decreases from a static value about 0.65 to a value about 0.4 at 4 m/sec. For wet wood, the corresponding values are about 0.7 and 0.15.

Coefficients of friction between wood and wood were similar to those between wood and rough steel, and those between wood and other materials varied in a manner which may be related to strength of adhesion.

figure 66: Wood sliding on wood – the effect of sliding speed and moisture content on friction coefficient for “normal” wood: radiata pine, mountain ash. (McKenzie & Karpovich, 1968)

4.2.7 Tension and compression diagonals in the sheathing material

In the paragraphs before is explained how to calculate the deflection due to the shear-deformation in the sheathing. This shear force is caused by a distributed load along the perimeter of the shear wall. The force-distribution in the sheet can also be observed in a different way. In the literature review is explained that the force-distribution in the sheathing can be seen as compression and tension diagonals bracing the frame. In this paragraph, the sheathing will be analysed using this alternative approach. Therefore an assumption is made with respect to the effective width of the compression and tension diagonal.

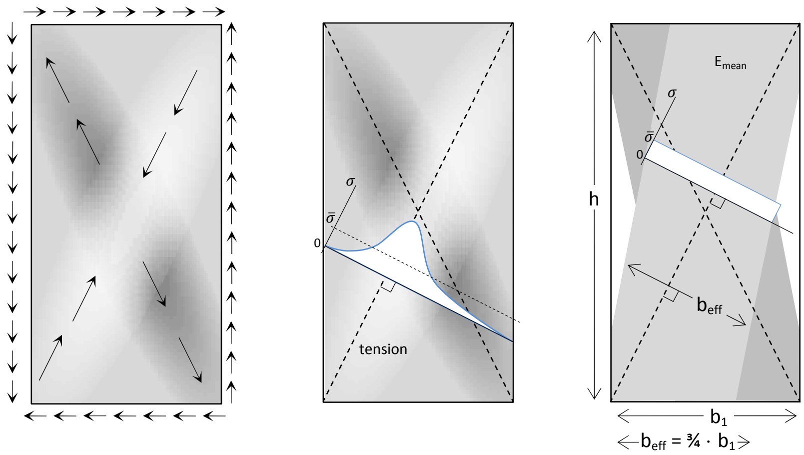


figure 67: Effective width of the tension and compression diagonals in the sheathing

The assumption b_{eff} = ¾ · b_1 is not made by coincidence. This choice is made similar to the method of Hrennikoff, which formed the basis for modern finite element calculations on plates and shells. The lattice method is used to calculate a plate or shell as a set of 1D elements (bars / beams). The area of the bracing elements is ¾ · b_1 · t. The question can arise if this method can be useful for the analytical determination of the racking stiffness.

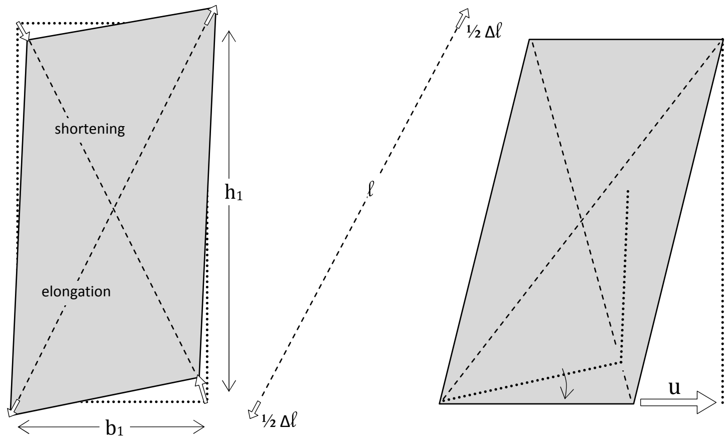


figure 68: Elongation and shortening of diagonals result in horizontal displacement

The horizontal deformation due to the elongation and shortening of the tension and compressive diagonal is calculated in the spreadsheet with use of the equation that is derived below.

From paragraph 5.2 the following equation can be derived:

$$k_R = \frac{E \cdot A}{l} = \frac{\left(\sqrt{\mathrm{b}_1^2 + \mathrm{h}_1^2}\right)^2}{\mathrm{b}^2} \cdot \frac{\mathrm{F_{v,Ed}}}{\mathrm{u}}$$

Because the tensile diagonal is accompanied by a compression diagonal, $F_{v,Ed}$ may be divided by 2. A, the effective area of the tension and compression diagonal, is assumed to be:

$$A = \mathrm{b_{eff}} \cdot t = \mathrm{b_1} \cdot \frac{3}{4} \cdot t$$

$$\mathrm{l} = \sqrt{\mathrm{b_1^2} + \mathrm{h_1^2}}$$

E is the young's modulus of the sheeting material E_1. The horizontal deformation u, is the deformation due to tension and compression in the diagonals, $u_{t\&c}$. $F_{v,Ed}$ is the horizontal racking force F. This will result in:

$$\mathrm{u}_{t\&c} = \frac{\mathrm{F} \cdot \sqrt{\mathrm{b_1^2} + \mathrm{h_1^2}}}{\frac{3}{2} \cdot \mathrm{E_1} \cdot \mathrm{b_1} \cdot t \cdot n_{sheets}} \left(1 + \frac{\mathrm{h_1^2}}{\mathrm{b_1^2}}\right) \tag{4.21}$$

The deformation of the sheathing due to elongation and shortening of the tension and compression diagonal can be compared with the deformation due to shear as explained in paragraph 4.2.4. See Appendix VII step 4* - 7*. From this comparison can be concluded that the magnitude of deformation calculated for shear, is smaller than the magnitude of deformation calculated because of tension and compression in the sheathing. The contribution of shear-deformation in the racking displacement of the shear-wall elements in the tests, on average is calculated to be 0,97 mm. For deformation due to tensile and compressive deformation of the sheathing, the average contribution in the racking displacement of the shear-wall element in the tests is calculated to be 1,10 mm. These values are respectively, on average, 12% and 17% of the total racking displacement. It can be concluded that the calculation of shear in the sheathing is of the same magnitude as would be calculated with sheathing deformation due to tensile and compressive stresses in the diagonals.

4.3 Deformation limits in serviceability limit state

The deformation limits which have to be regarded in serviceability limit state, as well as the requirement for deformation capacity in ultimate limit state will be addressed in this paragraph.

With use of the timber-frame shear-wall stiffness, the deformation of the shear-wall in serviceability limit state can be calculated. In multi-storey timber-frame buildings this calculation can be made for the individual floor level, or for the full building height. In Eurocode 0 (NEN-EN 1990) limits to the horizontal deformation of buildings are presented:

$$\mathrm{u_i} \leq \mathrm{h_i}/300$$

$$\mathrm{u} \leq \mathrm{h}/500$$

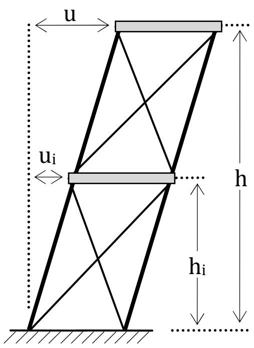


figure 69: Deformations at the individual level and top of the building.

For a certain timber-frame shear-wall element with storey-height of 3000 mm the deformation limit will be 3000/500 = 6,00 mm. If this requirement is not satisfied, measures have to be taken to increase the stiffness of the timber-frame element. Next to these limits, there can be reason to have more severe requirements. These can be conditions stated by the client, or material or system limitations. Damage to ceilings, windows, doors, finishing, or the appearance of the building can occur when deformations exceed the limits.

Another aspect that have to be observed is the required deformation capacity. Although the actual deformation have to be limited, the timber-frame element needs to have a possibility to display a higher deformation than allowed before failure. If the unfortunate case of collapse is caused by overloading of the shear-wall for instance, people inside the building have to be warned by the structure, that the situation is getting dangerous. It is difficult to quantify this requirement in terms of a minimum deformation capacity.

Between the load-level of ultimate allowed deflection, and ultimate load, a trajectory have to be present in which the structure will develop warning symptoms. These can be distortion of the timber-frame panel, and damage to the finishing layers for instance. Observing the materials in the timber-frame shear-wall element, it can be concluded that it is beneficial to design the fasteners as the governing part of the element.

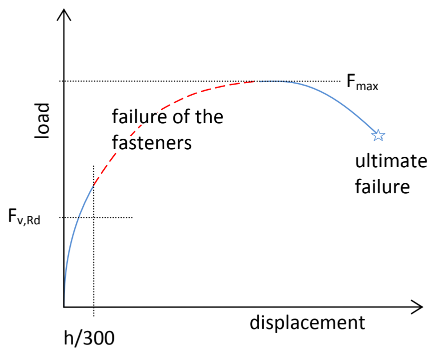


figure 70: The fasteners have to be governing in the shear-wall design.

The fasteners have to fail before the boards rupture in a brittle manner. This requirement from strength perspective is already incorporated in the Eurocode 5 design rules, by means of a lower limit to the fastener spacing s, and a design rule for the centre-to-centre distance of the vertical framing elements that prevent the sheathing from buckling.

4.4 Comparison between test-based and calculated racking stiffness

In the paragraph before the contributions to the racking deformation of the shear-wall were explained. With use of the derived equations, the racking deformation of a certain timber-frame shear-wall element can be calculated analytically. From the literature review the results of 31 shear-wall tests were found. The geometry of these tests was used for the analytical calculation of the racking deformation and racking stiffness. In these calculations a load-level of 40% of the ultimate load reached in the test was used. In this way the test-results with respect to racking stiffness and deformation, and the results of analytical calculation can be compared. Both the test-based determination of the racking stiffness, and the analytical calculation of deformation and racking stiffness was carried out with use of an Excel spreadsheet. This approach limited the amount of work needed for the analyses of 31 tests.

Reference is made to Appendix V for the load-displacement data of the shear-wall tests, to Appendix VI for the determination of the shear-wall racking stiffness on basis of the test-results according to NEN-EN 594. Furthermore reference is made to Appendix VII for the analytical calculation of the shear-wall racking stiffness taking into account the contributions to the racking deformation as shown in the paragraph before. In the analytical calculation of the racking stiffness use was made of a fastener slip-modulus, and a hold-down stiffness value. The values that were used are reported in Appendix VIII and Appendix IX. Finally, a comparison is made between the test-based racking stiffness and analytically calculated racking stiffness in Appendix X, with use of a bar chart.

4.4.1 Observed agreement between test-based and calculated racking stiffness

Load-displacement data and geometry of 31 tests was found in literature. These tests are analysed with use of the analytical calculation method. The load-displacement data was used to determine a test-based racking-stiffness R_{test} (based on F_{02} (20% F_{max}) and F_{04} (40% F_{max}) according to NEN-EN 594). With use of the fastener slip-modulus K_{ser}, the load-level F_{04} (because the hold-down stiffness depends on the load-level), and the analytical calculation method, the racking stiffness R_{Σ} was calculated. The components of the analytical calculation method are shown in paragraph 4.2 and summarized in paragraph 4.8 on page 78. The calculations that were basis for the comparison between calculation and test-results are shown in the appendices Appendix V - Appendix X. The average results of the comparison between the analytical calculation method, and the tests are shown qualitative in figure 71.

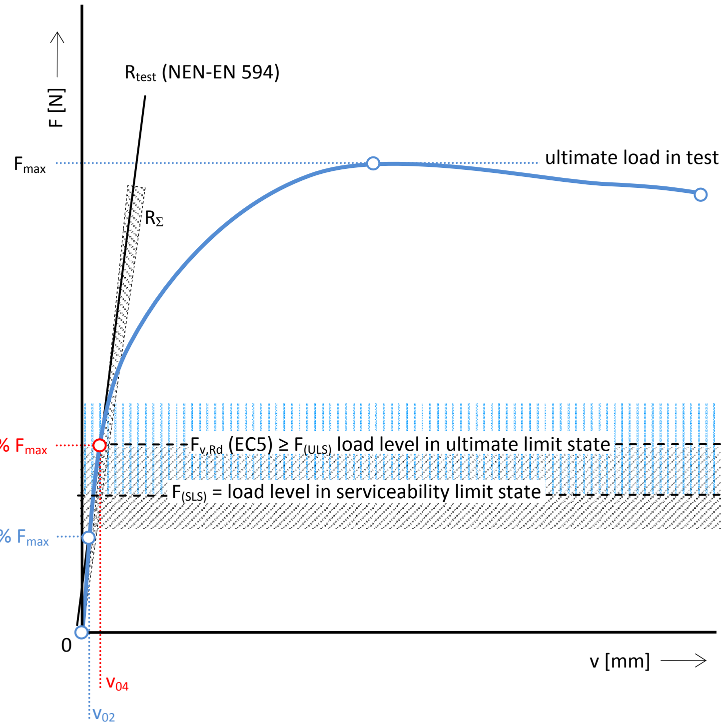


figure 71: Fictitious load-displacement graph with parameters.

Ratio between calculated and test-based results

The analytically calculated racking stiffness and test-based racking stiffness show reasonable agreement for a number of tests. The stiffness in the tests (R_{test}) on average is 11% higher, compared with the analytically calculated racking stiffness (R_{Σ}).

Racking displacement

The ratio between the resulting displacement in the test, and the analytically calculated displacement, v_{04} / u_{Σ} is 0,92. If v_{04} is compared with a limit for the deflection of 1/500 · h = 3000 / 500 = 6,0 mm, it can be seen that v_{04} is higher compared with the limit value. $v_{04} \approx 1{,}13 \cdot 6{,}0$ mm

ratio	mean	mean deviation
R_{test} / R_{Σ}	1,06	0,35
v_{04} / u_{Σ}	0,92	0,22
F_{04} / $F_{(ULS)}$	1,04	0,26
F_{04} / $F_{(SLS)}$	1,40	0,35
v_{04} / $u_{(h/500)}$	1,13	0,45

table 13: Observed ratio's between test-based and calculation results

4.4.2 Racking stiffness of the timber-frame shear-wall in relation to limit state design

Although K_{ser} is used for the fastener-slip modulus, the calculated stiffness R_{Σ} is a bit lower than the test-based stiffness R_{test}. It is proposed to use the analytically determined racking stiffness R_{Σ} as a lower limit to the racking stiffness in serviceability limit state, using K_{ser} for the fastener-slip modulus. If K_u will be used as a slip-modulus, R_{Σ} will be even lower compared with the test-based racking stiffness.

That use of K_{ser} in the analytical calculation method results in a lower limit value for the racking stiffness in SLS, is also supported by the following observation. The load-level in SLS can be calculated by dividing the load-level in ULS by the load-factor γ_Q, which is at least 1,35. $F_{(SLS)} = F_{(ULS)} / 1{,}35$. The load-level $F_{(ULS)}$ will not be higher than the shear-wall design strength $F_{v,Rd}$ that can be calculated with use of the Eurocode 5. The SLS load-level $F_{(SLS)}$ of the observed shear-walls on average is equal to $0{,}71 \cdot F_{04}$. The analytically calculated racking stiffness R_{Σ}, determined on the load-level F_{04}, was shown to be almost equal to R_{test} ($R_{test} / R_{\Sigma} \approx 1{,}06$). If a lower load-level was used for the test-based determined racking stiffness R_{test}, R_{test} would probably have been even higher. It can be concluded that the use of K_{ser} will lead to a calculated racking stiffness R_{Σ}, which will show reasonable agreement with tests on SLS load-level. On ULS load-level ($F_{(ULS)} \approx F_{04}$) it can be possible that K_{ser} gives an overestimation of the shear-wall racking stiffness (R_{test}). In such a case, the use of K_u can provide a lower value for the calculated racking stiffness R_{Σ}. This can also be seen from the bar-charts in Appendix X (page 167).

component	**mean of 31 tests** [mm]	mean deviation [mm]	**share in total deformation**
u_f (fasteners along the perimeter)	3,3	1,6	48%
u_{sh} (shear of the sheeting)	1,0	0,6	12%
u_{str} (strain in leading & trailing stud)	0,7	0,6	8%
u_{hd} (hold-down anchorage)	1,1	0,8	13%
u_c (compression perpendicular to grain)	1,3	1,2	15%
Total deformation	7,4	2,9	100%

table 14: Share of deformation components in total deformation according to analytical calculation (values are separate averages of 31 tests for each observed component)

The calculations that were basis for the comparison between calculation and test-results are shown in the appendices Appendix V - Appendix X. The mentioned ratio's and values are average values obtained from the test- and calculation-results of 31 tests. Deviation from these average values is shown in the tables above. In general, good agreement was shown between tests and calculation results. However some tests show significant disagreement. An attempt will be done to clarify these differences between test and calculation results in the next paragraph, it is not possible to give a definite explanation for the differences in each case.

4.4.1 Reasons for deviation

In general reasonable agreement was shown from comparison between the racking stiffness determined from load-displacement data, and the analytically calculated racking stiffness. Reference is made to the bar-charts in Appendix X. Although a sensible order of magnitude is obtained for the racking stiffness, a significant deviation from the test-based determined stiffness values exist for some tests, if the results of the analytical calculation method are observed.

Limited hold-down stiffness

The deviation between test-based determined racking stiffness, and analytical calculated racking stiffness in case of test (2.1) can possibly be explained observing the applied hold-down solution. Use was made of a certain type of steel angle. Using the test-information from the report, a stiffness value of this angle section could be determined. The load-level that could be reached with use of this hold-down was not mentioned in this report. It may be expected that this hold-down failed in an early phase of the load-displacement trajectory because the steel angle was designed very thin.

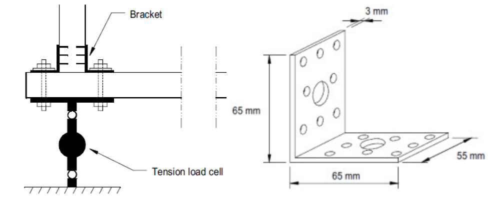


figure 72: Applied hold-down anchor in test (2.1) (Andreasson, 2000).

In the literature review was explained how failure of the hold-down will cause a change in the force distribution. The fasteners between sheathing and bottom rail will be loaded perpendicular to the edge of the panel that will worsen the performance of the shear-wall with respect to racking strength and racking stiffness. See figure 73.

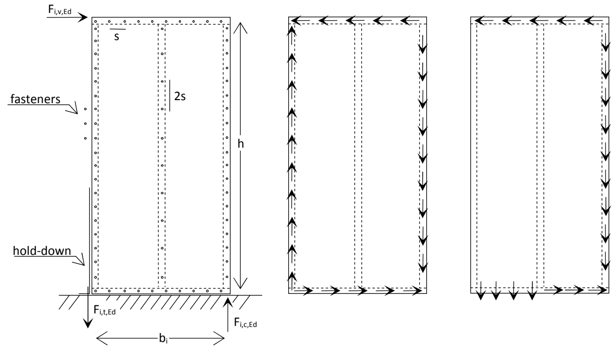


figure 73: Timber-frame shear-wall panel with fastener spacing (s). Forces on fasteners exerted by the sheeting: (L) in case of sufficient hold-down capacity, (R) in case of insufficient hold-down capacity.

Because the literature source did not provide information with respect to the way of failure, and the ultimate load-level of the hold-down anchor, it was not possible to take these effects into account in the analytical calculation method. This led to an overestimation of the racking stiffness in the calculations.

Uncertain fastener slip-modulus

Another issue that can account for a part of the deviations between test-based racking stiffness, and the result of analytical calculation can be the uncertainty in the fastener stiffness. For tests (13.4), (12.4), (13.2), and (12.2) the racking stiffness in the calculations was underestimated. In these tests Rigidur-RH gypsum-fibre board was used as a sheathing material. The difference between test and calculation can possibly be explained by the use of annular ringed shank nails as fastener. The fastener-slip modulus of this type of sheathing-to-timber connection was determined from fastener tests. Because the effect of the annular ringed nail shank on the stiffness seemed unsure, it was decided to choose a somewhat lower value for the fastener-stiffness in the calculation. This led to underestimation of the shear-wall racking stiffness in the analytical calculation.

Uncertainty about the fastener-slip modulus could have been reason for deviation in the calculated racking stiffness for more tests, than only these tests mentioned above. In chapter 0 it was shown that the fastener stiffness in SLS can vary around K_{ser}. Because the fastener stiffness is of big influence on the shear-wall stiffness, it is obvious that deviations between calculated and test-based racking stiffness can be caused by the uncertainty in the fastener-slip modulus.

Friction and bearing between the panels

Another effect on the racking stiffness can have been the influence of friction between the sheathing panels. In the derivation of the equations used in the analytical calculation method, it is assumed that the panels can rotate without restraint. However, friction and bearing can exist between the boards, if not enough spacing was applied in the actual test set-up, and the panels display different rotation relative to each other.

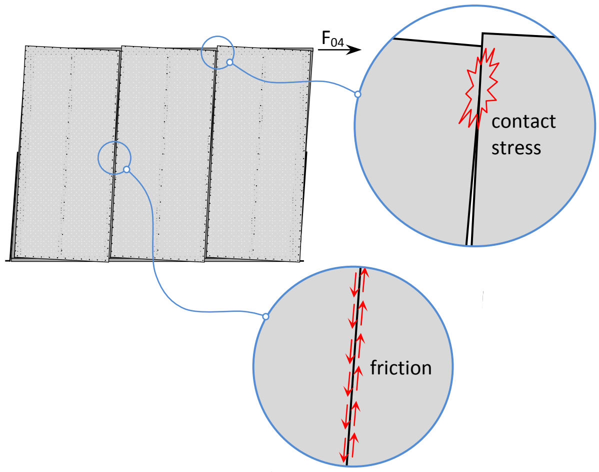


figure 74: Bearing and friction between sheathing panels in experimental testing.

Slant nailing and intermediate studs

Deviation can also be caused by the additional resistance coming from the connections between the timber framing elements. The timber rails and studs are connected together using end-nails or toe-nails through the rails in the end grain of the vertical studs. These fasteners are often mentioned in the test-reports, their effect on the shear-wall racking stiffness however is difficult to quantify, and is not taken into account in the analytical calculation method.

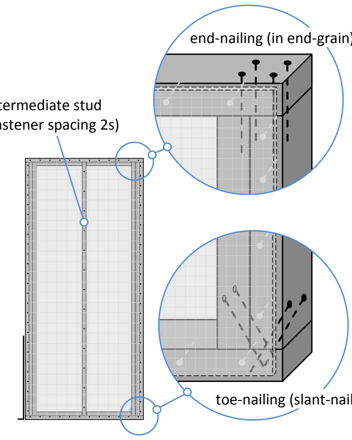


figure 75: Intermediate studs and slant-nails can give additional resistance, however their contribution is difficult to quantify.

4.5 Translation between deformation and stiffness

In the paragraphs before several contributions to the horizontal racking deformation of the shear-wall were shown. Equations were derived to calculate the horizontal deformation coming from fastener slip, strain in the hold-down, compression perpendicular to grain, shear deformation of the sheathing, axial strain in the vertical studs, and slip between bottom rail and substructure. However, in this chapter a method for calculating the racking stiffness is aimed at. Therefore it is needed to change the equations for deformation contribution into equations that give the components of the racking stiffness. The procedure to do this will be shown below for the case of fastener-slip.

The contribution of fastener-slip in the racking deformation can be changed into an equation for the fastener component in the shear-wall racking stiffness as following:

$$u_f = \frac{2 \cdot F_{04} \cdot s \cdot (1 + \frac{h_1}{b_1})}{n_{panels} \cdot b_1 \cdot K_{ser} \cdot n_{sheets}}$$

Stiffness is the relation between force and displacement. In this case the force is F_{04}:

$$\mathrm{R_f} = \frac{\mathrm{F_{04}}}{\mathrm{u_f}}$$

This will result in:

$$R_f = \frac{n_{panels} \cdot b_1 \cdot n_{sheets}}{2 \cdot (1 + \frac{h_1}{b_1})} \cdot \frac{K_{ser}}{s}$$

All the other contributions in the racking deformation were translated to racking stiffness components as shown above. Because timber-frame shear-wall element resembles a serial system, the components in the timber-frame shear-wall racking stiffness may be added together as is shown in the equation below.

$$R_{Rd} = \frac{1}{\frac{1}{R_{f,Rd}} + \frac{1}{R_{hd,Rd}} + \frac{1}{R_{c,Rd}} + \frac{1}{R_{G,Rd}} + \frac{1}{R_{str,Rd}} + \frac{1}{\mathrm{R_{v,Rd}}}}$$

If a timber-frame shear-wall with different sheathing and fastener lay-out on both faces is used, a different summation have to be used, taking into account the parallel action of both shear-wall faces.

4.6 Conclusion

The research question that was shown in the introduction, can be answered with use of the analysis carried out in this chapter.

> 4. *Is it possible to derive an analytical calculation method to calculate the timber-frame shear-wall racking stiffness?*
>
>   a. *How does the calculation method look like?*
>   - *Which contributions in the deformation have to be taken into account?*
>   - *What is the share of each contribution in the total resulting racking deformation?*
>   - *Which parameters and equations have to be used?*
>
>   b. *Are the calculated racking stiffness values useful in structural design?*
>   - *Do the calculation results agree with test-results?*
>   - *For which purpose is the resulting racking stiffness to be used?*

In answer to the research question, the following conclusions can be drawn:

Conclusion 4.:

> *From the observation and discussion of calculation and test-results in this chapter, it is shown to be possible to derive an analytical calculation method for the determination of the timber-frame shear-wall racking stiffness.*

Conclusion 4.a.:

> *The parameters and equations to be used in the analytical calculation method, to calculate the components which contribute to the racking deflection of the shear-wall, can be found in paragraph 4.8 hereafter.*

The components to be taken into account for the calculation of the shear-wall racking deformation are listed below, and can be expressed with use of a percentage of the total racking deformation. These are average percentages based on the analysis of 31 tests:

u_f	:	*fasteners along the perimeter*	:	*48%*
u_{sh}	:	*shear of the sheeting*	:	*12%*
u_{str}	:	*strain in leading & trailing stud*	:	*8%*
u_{hd}	:	*hold-down anchorage*	:	*13%*
u_c	:	*compression perpendicular to grain*	:	*15%*

Conclusion 4.b.:

The racking stiffness calculated with use of the proposed analytical calculation method are useful in the structural design of a timber-frame multi-storey building because clarification can be given about the application and meaning of the resulting racking stiffness values.

Reasonable agreement can be seen from the comparison between analytically calculated racking stiffness values, and the test-based racking stiffness values determined with use of the load-displacement data found in literature.

If K_{ser} is used as the fastener-slip modulus in the analytical calculation method, the resulting racking stiffness value can be used for serviceability limit state (SLS) purposes. For ultimate limit state (ULS) purposes, K_{ser} or K_u can be used in the analytical calculation method, the most disadvantageous result have to be used.

In addition to the conclusions, few recommendations will be given on the next page.

4.7 Recommendations

From the discussion of the observed agreement and differences between test-based determined racking stiffness values, and analytically calculated racking stiffness values, it was shown that there are some issues left for further research. In relation to these issues a few recommendations will be given in this paragraph.

- Additional research (in literature) can be done to receive load-displacement data of tests on full-scale shear-wall elements sheathed with particleboard. The timber-frame shear-wall elements with these materials are not covered in the analysis and comparison reported about in this report.

- It would be useful if the result of this master's thesis can be expanded for the calculation and modelling of asymmetric timber-frame shear-wall elements racking stiffness. Asymmetric in this case implies the application of different sheathing and fastener configuration on both faces of the shear-wall element. In timber-frame practice, it is a common choice to choose for a gypsum-paper of gypsum-fibre board material on the interior side of the wall, and a wood-based panel on the cavity face of the shear-wall element. These walls however are not analysed in this master's thesis. Tests on asymmetric shear-wall elements are available in literature.

On the next two pages the analytical calculation method is summarised.

4.8 Timber-frame racking stiffness calculation method

Explanation about, and calculation of the contributions to the racking deformation are given in the paragraphs before. It is shown that the proposed equations can be used to calculate the racking stiffness of the timber-frame shear-wall panel. In this paragraph the equations are rewritten to result in a straightforward method for the determination of the timber-frame shear-wall stiffness.

4.22
$$k_{R_{Rd}} = R_{Rd} \cdot (1 + \frac{h^2}{b^2})$$
stiffness of the diagonal brace in the model of the timber-frame shear-wall (equation is derived in paragraph 5.2)

and:

4.23
$$R_{Rd} = \frac{1}{\frac{1}{R_{f,Rd}} + \frac{1}{R_{hd,Rd}} + \frac{1}{R_{c,Rd}} + \frac{1}{R_{G,Rd}} + \frac{1}{R_{str,Rd}} + \frac{1}{R_{v,Rd}}}$$
horizontal racking stiffness of the timber-frame shear-wall element (equation is shown in paragraph 4.5)

In which:

4.24
$$R_{f,Rd} = \frac{n_{panels} \cdot n_{faces} \cdot b_1}{2(1 + \frac{h_1}{b_1})} \cdot \frac{K_{ser}}{s}$$
influence of fastener slip-modulus on the racking stiffness

4.25
$$R_{G,Rd} = \frac{n_{panels} \cdot n_{faces} \cdot b_1 \cdot t}{h_1} \cdot G_{mean}$$
influence of shear-modulus in the sheathing on the racking stiffness

4.26
$$R_{hd,Rd} = \frac{(n_{panels} \cdot b_1)^2}{h_1^2} \cdot K_{hd}$$
influence of hold-down stiffness on the racking stiffness

4.27
$$R_{c,Rd} = \frac{(n_{panels} \cdot b_1)^2}{h_1^2} \cdot K_{c,90}$$
influence of compression perpendicular to the grain of the bottom rail on the racking stiffness

4.28
$$R_{str,Rd} = \frac{n_{studs} \cdot b_2 \cdot h_2 \cdot (n_{panels} \cdot b_1)^2}{h_1^3} \cdot E_2$$
influence of strain in tensile loaded leading stud, and compressed trailing stud, on the racking stiffness

4.29
$$R_{v,Rd} = n_{panels} \cdot n_{conn} \cdot K_v$$
influence of stiffness connectors between bottom-rail and substructure on racking stiffness, only to be taken into account if friction force F_{fr} is exceeded by the racking force on the timber-frame shear-wall element (see below)

In which:

b	[mm]	=	$n_{panels} \cdot b_1$ = width of the element
b_1	[mm]	=	width of a single panel
b_2	[mm]	=	thickness of the timber framing elements
E_2	[N/mm²]	=	timber framing elements Young's modulus
G_{mean}	[N/mm²]	=	mean shear modulus of the sheathing material
h	[mm]	=	h_1 = height of the element
h_1	[mm]	=	height of a single panel
h_2	[mm]	=	width of the timber framing elements
$K_{c,90}$	[N/mm]	=	stiffness perpendicular to grain of the bottom rail beneath the compressed stud (see below)
K_{hd}	[N/mm]	=	stiffness of the hold-down connector (see below)
K_{ser}	[N/mm]	=	fastener slip-modulus of the sheathing-to-timber fastener (see below)
K_v	[N/mm]	=	slip-modulus of the shear connectors between bottom rail and substructure (if bolts are applied, K_v can be calculated using the design rules in Eurocode 5 for K_{ser} in case of timber-to-timber connections, steel angle sections can also be used, their stiffness need experimental verification)
n_{panels}	[-]	=	number of panels in an element
n_{faces}	[-]	=	1 if only one side (faces) of the shear-wall is sheeted, 2 if both faces are sheeted
n_{conn}	[-]	=	number of shear connectors per panel in the bottom rail
n_{studs}	[-]	=	number of studs applied on the edge of the shear-wall element
s	[mm]	=	fastener spacing along the perimeter of the shear-wall panels
t	[mm]	=	thickness of the sheathing material (board)

4.30 $F_{fr} = \mu \cdot (\Sigma \mathrm{F_i} + \Sigma \mathrm{q_i} \cdot \mathrm{n_{panels}} \cdot \mathrm{b_1})$ friction resistance caused by the vertical loads on the timber-frame element and friction between bottom rail and substructure

In which:

μ	[-]	=	friction coefficient between bottom rail and substructure (in case of timber-to-timber $\mu \geq 0{,}40$)
$\Sigma \mathrm{F_i}$	[N]	=	sum of all permanent vertical concentrated loads on the timber-frame shear-wall panel
$\Sigma \mathrm{q_i}$	[N/mm]	=	sum of all permanent vertical distributed loads on the timber-frame shear-wall panel

4.31 $K_{c,90} = 1{,}3 \cdot ((n_{studs} \cdot b_2) + 30) \cdot h_2$ [N/mm] foundation stiffness for standard ungraded timber framing members

K_{ser} can be calculated with use of the Eurocode 5 design rules for timber-to-timber connections, wood-based panels-to-timber connections, and steel-to-timber connections. For gypsum paper-board and gypsum fibre-board sheathing-to-timber connections, use can be made of an experimentally determined fastener slip-modulus k_s according to NEN-EN 26891, or the table below.

Fastener	**Material** (thickness t [mm])	**Slip-modulus** [N/mm]	**Test**
N Ø2,34x40 (GN40)	GPB (12,0)	316	(1.1)
N Ø2,2x45	GPB (12,5) Gyproc	565	(7.2)
Scr Ø3,9 x 41	GPB (13,0) Danogips Normal	361	(8.1)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (lined edge)	637	(15.3)
N Ø2,45 x 34,5	GPB (12,5) Gyproc Normal (sawn edge)	552	(15.2)
Scr Ø3,0 x 38	GPB (12,5) Gyproc Normal (sawn edge)	668	(15.5)
St Ø1,4 x 1,6 x 60	GFB (12,5) (1-layer homogeneous) Fermacell	395	(14)
St Ø1,53 x 50	GFB (15,0) (3-layer homogeneous) Rigidur RH	457	(12.1)
N Ø2,5 x 65 (Rillenagel)		270	(12.2)
St Ø1,8 x 65		650	(13.1)
N Ø2,5 x 55		1408	(13.2)

table 15: Experimentally determined (mean) values for fastener slip-modulus k_s (K_{ser})

Gypsum paper-board (GPB)
Gypsum fibre-board (GFB)
Nail (N)
Screw (Scr)
Staple (St)

K_{hd} can be determined experimentally. With use of NEN-EN 26891 a stiffness parameter can be derived. As an alternative, the method can be used that was explained in paragraph 3.5. The stiffness of the fasteners, the stiffness of the steel and timber cross-section have to be taken into account. If the fastener holes exceed the fastener diameter, a hole clearance reduction have to be applied which is related to the load level on the hold-down.

4.32 $$K_{hd} = \frac{1}{\frac{1}{\mathrm{n_{fasteners}} \cdot \mathrm{K_{ser}}} + \frac{1}{k_{steel}} + \frac{1}{k_{timber}}}$$ hold-down stiffness without hole-clearance reduction

4.33 $$K_{hd;reduced} = \frac{F_{hd,Ed}}{\left(\frac{F_{hd,Ed}}{K_{hd}}\right) + \left(\frac{d_{hole} - d_{fastener}}{2}\right)}$$ reduced hold-down stiffness

In which:

$d_{fastener}$	[mm]	=	diameter of the applied hold-down fastener (part of the shank beneath the fastener head)
d_{hole}	[mm]	=	diameter of the fastener-hole in the hold-down steel-section
$F_{hd,Ed}$	[N]	=	load on the hold-down anchor
k_{steel}	[N/mm]	=	Stiffness of the steel section (E·A/L)
k_{timber}	[N/mm]	=	Stiffness of the timber section (E·A/L)
$n_{fasteners}$	[-]	=	number of fasteners used to attach the hold-down to the timber

5 Timber-frame shear-wall modelling approach

In the chapters before is explained how the racking stiffness of timber-frame shear-walls can be determined in an analytical way. In this chapter an answer will be given to research question five:

> 5. *Is it possible to suggest a modelling approach for structural analysis of timber-frame shear-wall racking stiffness, using a regular framework program?*
>   a. *How does the model look like?*
>   b. *For which purposes is the modelling approach to be used?*

In answer to the research question, this chapter presents a modelling approach, which can be used for structural analysis of multi-storey timber-frame structures, using a framework program. This type of software package is common used in the engineering office. The proposed way of modelling makes use of the analytical derived shear-wall racking stiffness, and enables the verification of the structural design. Insight in the force distribution in ULS and deformations in SLS can be gained with use of the modelling approach. In the following paragraphs the lay-out and parameters of the model will be presented, as well as a calculation and modelling example. In the last paragraph the research question will be answered by summarizing the chapter in a concise conclusion.

5.1 Lay-out of the model

It is not difficult to design very complicated models with a lot of nodes and elements. These extensive models can be analysed with use of the finite element software, and powerful hardware. However, building and testing the model will cost a lot of time. Therefore it is more convenient to design a model which is not complicated to build, and can be understand by others without the need of being a software expert. In this paragraph will be shown how a timber-frame shear-wall can be modelled according to these demands. See the figure below.

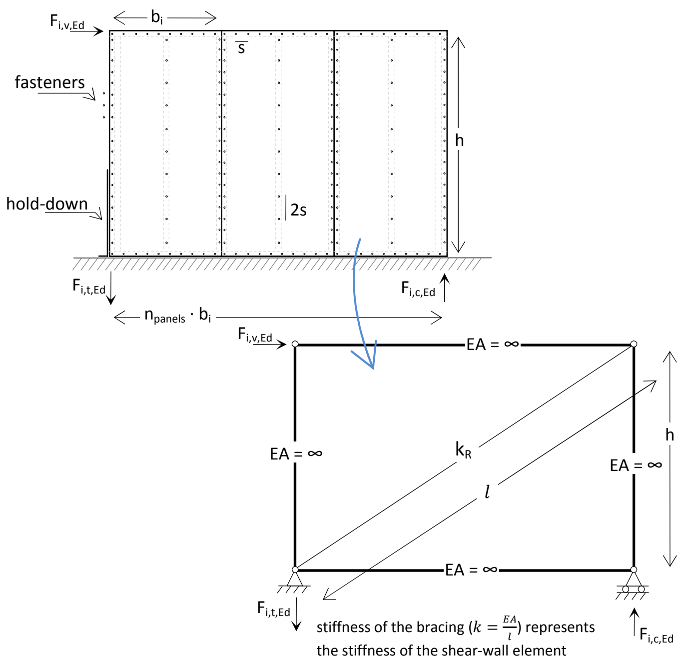


figure 76: Model for the structural analysis of the timber-frame shear-wall in a framework program.

5.2 Parameters in the model

In the chapters before is explained how to translate the shear-wall properties and geometry into one single parameter for the racking stiffness (R). Except for the loading and boundary conditions, only this racking stiffness (R), and geometry of the shear-wall element (h & b) will be the input necessary in the model. The fastener stiffness and spacing, panel shear-modulus and modulus of elasticity, etc. are already incorporated in the racking stiffness value R. As can be seen from figure 76, the racking stiffness R of the shear wall has to be translated into the stiffness k_R of the brace. For this purpose, the following equation can be used.

$$k_R = \left(1 + \frac{h^2}{b^2}\right)\mathrm{R} \qquad 7.1$$

This equation can be derived as following:

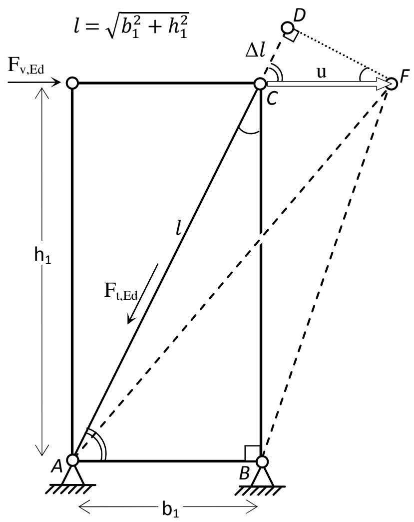


figure 77: Horizontal deflection u due to elongation Δl of the diagonal brace.

Triangle ABC is congruent with CDF as can be seen in figure 77. This gives the following relation:

$$\frac{\mathrm{u}}{\Delta \mathrm{l}} = \frac{\sqrt{\mathrm{h}^2 + \mathrm{b}^2}}{\mathrm{b}}$$

$$\Delta \mathrm{l} = \frac{\mathrm{u} \cdot \mathrm{b}}{\sqrt{\mathrm{h}^2 + \mathrm{b}^2}}$$

It can also be seen that:

$$\mathrm{F}_{\mathrm{t,Ed}} = \frac{\sqrt{\mathrm{h}^2 + \mathrm{b}^2}}{\mathrm{b}} \cdot \mathrm{F}_{\mathrm{v,Ed}}$$

The stiffness k_R of the brace is the ratio between the force $F_{t,Ed}$ and elongation Δl, combining the equations given above, the result will be:

$$\mathrm{k}_{\mathrm{R}} = \frac{\mathrm{F}_{\mathrm{t,Ed}}}{\Delta \mathrm{l}} = \frac{\left(\sqrt{\mathrm{h}^2 + \mathrm{b}^2}\right)^2}{\mathrm{b}^2} \cdot \frac{\mathrm{F}_{\mathrm{v,Ed}}}{\mathrm{u}} \qquad 7.2$$

The horizontal racking stiffness, R, of the shear-wall panel can be determined using the analytical method explained in chapter 4, and can be expressed as:

$$\mathrm{R} = \frac{\mathrm{F}_{\mathrm{v,Ed}}}{\mathrm{u}}$$

See figure 78.

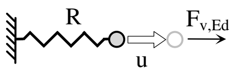


figure 78: Horizontal racking deformation of the shear-wall u, seen as elongation of a spring due to a force F.

Equation 7.2 can be simplified and the following relation can be derived:

$$k_R = R \cdot (1 + \frac{h_1^2}{b_1^2})$$

In which R is the horizontal racking stiffness of the timber-frame shear-wall panel, and k_R is the stiffness of the diagonal brace in the model, and h_1, and b_1 are the dimensions of the timber-frame shear-wall panel. It may be concluded that this relation is also valid for the timber-frame shear-wall element as a whole. In such a case the equation can be generalized to:

$$k_R = R \cdot (1 + \frac{h^2}{b^2}) \tag{7.3}$$

In which $b = \mathrm{n_{panels}} \cdot \mathrm{b_1}$, and $h = \mathrm{h_1}$.

The axial stiffness of the brace, k_R, is the product of Young's modulus and area, divided by the length of the brace, this gives the cross-sectional area of the brace:

$$A = \frac{k_R \cdot l}{E} \tag{7.4}$$

5.3 Using the modelling approach

The analytical calculation method and modelling approach can be used to determine the force distribution within a timber-frame structure. The methods can be very useful in the structural design of multi-storey timber-frame buildings. To determine the effective number of bays for the transfer of the wind-loading shear-force for example.

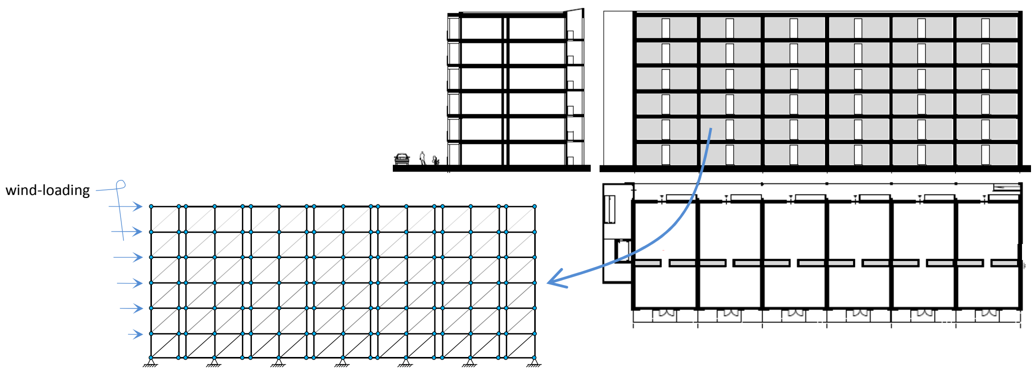


figure 79: Model of a six-storey timber-frame structure.

It is also possible to analyse a timber-frame floor diaphragm with use of the analytical calculation method and the proposed modelling approach. In such a case it can be useful to determine the distribution of wind-loading over the shear-walls. In a 2D analysis, the timber-frame shear-walls can be considered as spring supports. The spring-stiffness in this case is equal to the shear-wall racking stiffness. In the figure below an example of such an approach is given.

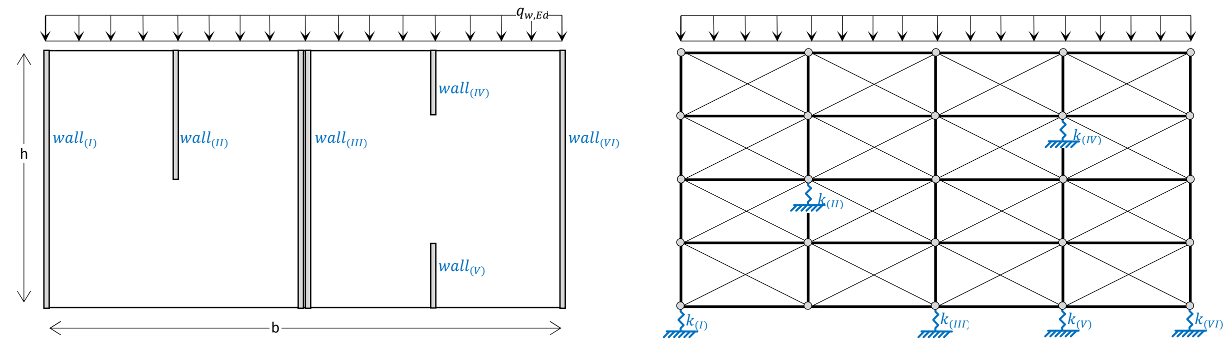


figure 80: Analysing the force-distribution between several shear-walls.

5.4 Modelling and calculation example

In this paragraph will be demonstrated how to use the analytical method to determine the shear-wall stiffness, and how to use the modelling approach in a framework structural analysis program. For this purpose use was made of the shear-wall configuration of test (10.1) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002).

5.4.1 Geometry and shear-wall properties Test (10.1)

Sheathing:	Knauf gypsum paper-board t = 12,5 mm on both faces
Fastener:	nail Ø2,5x45 s = 50 mm
Dimensions:	b x h = 1250 x 2500 mm
	timber framing elements 40 x 100 mm
Hold-down:	BMF anchor 52 x screw Ø5,0x50

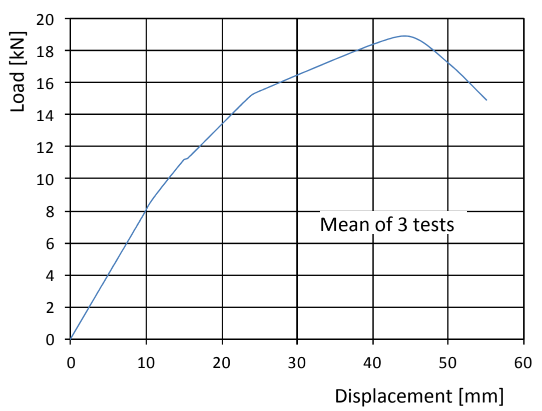


figure 81: Load-displacement graph test (10.1)

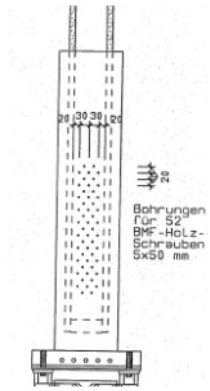


figure 82: Hold-down lay-out test (10.1)

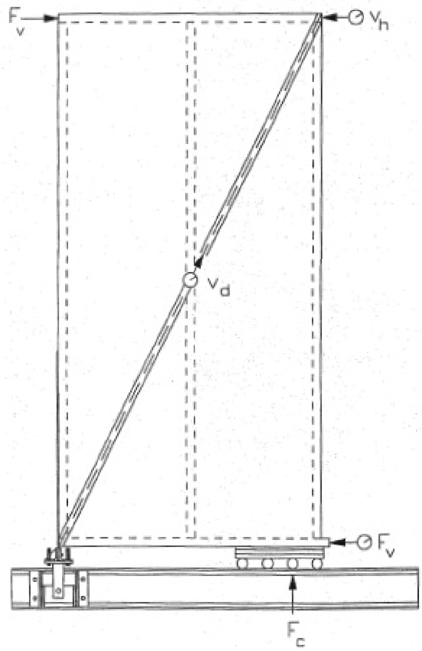


figure 83: Test set-up test (10.1)

For the calculation, a number of parameters is required to be known, these are listed below:

n_{panels}	=	1	[-]	=	number of panels in an element
b	=	1250,00	[mm]	=	$n_{panels} \cdot b_1 = b_1$ = width of the element
b_2	=	40,00	[mm]	=	thickness of the timber framing elements
E_2	=	11000,00	[N/mm^2]	=	timber framing elements Young's modulus
F_{04}	=	7563,28	[N]	=	40% of F_{max} (F_{max} = the maximum load reached in the test)
G_{mean}	=	700,00	[N/mm^2]	=	mean shear modulus of the sheathing material
h	=	2500,00	[mm]	=	h_1 = height of the element
h_2	=	100,00	[mm]	=	width of the timber framing elements
n_{faces}	=	2	[-]	=	both faces of the shear-wall are sheeted
n_{studs}	=	1	[-]	=	number of studs applied on the edge of the shear-wall element
s	=	50,00	[mm]	=	fastener spacing along the perimeter of the shear-wall panels
t	=	12,50	[mm]	=	thickness of the sheathing material (board)
z	=	625,00	[mm]	=	b / 2 = eccentricity of the edge studs to centre of the shear-wall element

5.4.2 Calculation of the racking stiffness

The timber-frame shear-wall racking stiffness can be calculated using the method shown in paragraph 4.8. The fastener slip-modulus K_{ser} can be chosen from table 15: the average value of fastener-tests (7.2), (15.3), and (15.2) will be taken because a nail Ø2,5x45 in 12,5 mm gypsum paper-board is used. This results in:

$$K_{ser} = \frac{636{,}76 + 552{,}36 + 564{,}61}{3} = 584{,}58 \text{ [N/mm]}$$

$K_{c,90}$ is the stiffness of the perpendicular to grain compressed volume beneath the compressed stud. Based on an experimentally determined foundation modulus, $K_{c,90}$ can be calculated to be:

$$K_{c,90} = 1{,}33 \cdot \left((n_{studs} \cdot b_2) + 30\right) \cdot h_2 = 1{,}33 \cdot \left((1 \cdot 40{,}00) + 30\right) \cdot 100{,}00 = 9310{,}00\ [\mathrm{N/mm}]$$

Hold-down stiffness K_{hd}

The hold-down stiffness K_{hd} depends on the stiffness of the steel and timber section, and on the stiffness of the fastener group. Because no information is known about the fastener holes, it is assumed that the hole is 0,5 mm larger than the fastener. A BMF anchor is used with 52 screws Ø5,0x50. The dimensions of the anchor are assumed to be 3 x 100 x 360.

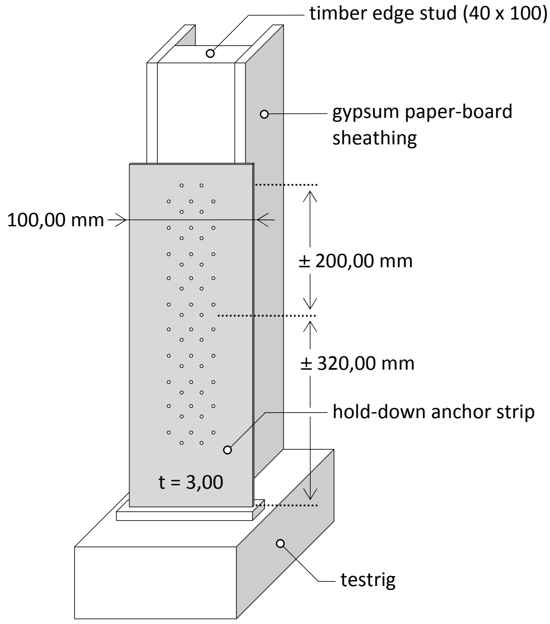


figure 84: Hold-down lay-out test (10.1)

$$k_{steel} = \frac{EA}{l} = \frac{2{,}1 \cdot 10^5 \cdot 300{,}00}{320{,}00} = 1968{,}75 \cdot 10^2\ [\mathrm{N/mm}]$$

$$k_{timber} = \frac{EA}{l} = \frac{11000{,}00 \cdot 4000{,}00}{200{,}00} = 2200{,}00 \cdot 10^2\ [\mathrm{N/mm}]$$

The steel strip is directly attached to the test rig (see also figure 82). Therefore no stiffness of the hold-down foot, the connection between hold-down and substructure, have to be taken into account.

For the calculation of the stiffness of the fasteners the fastener slip-modulus K_{ser} is determined for a steel-to-timber connection. The fastener slip-modulus may be multiplied with 2, ρ_m = 420 kg/m^3 for timber strength class C24, and $d_{ef} = 0{,}66 \cdot d = 0{,}66 \cdot 5{,}0 = 3{,}30$ mm:

$$2 \cdot K_{ser} = \frac{\rho_m^{1{,}5} \cdot d}{23} = 2 \cdot \frac{420{,}00^{1{,}5} \cdot 3{,}30}{23} = 2 \cdot 1234{,}98 = 2469{,}96\ [\mathrm{N/mm}]$$

$$k_{fasteners} = n \cdot 2 \cdot K_{ser} = 52 \cdot 2469{,}96 = 1284{,}38 \cdot 10^2\ [\mathrm{N/mm}]$$

If the stiffness of the steel and timber cross-section are added to the stiffness of the fasteners, the hold-down stiffness can be calculated to be:

$$K_{hd} = \frac{1}{\frac{1}{\mathrm{n_{fasteners}} \cdot \mathrm{K_{ser}}} + \frac{1}{k_{steel}} + \frac{1}{k_{timber}}}$$

$$K_{hd} = \frac{1}{\frac{1}{1284{,}38 \cdot 10^2} + \frac{1}{1968{,}75 \cdot 10^2} + \frac{1}{2200{,}00 \cdot 10^2}}$$

$$K_{hd} = 574{,}36 \cdot 10^2 \text{ [N/mm]}$$

The hold-down stiffness is reduced because of hole-clearance:

$$K_{hd;reduced} = \frac{F_{hd,Ed}}{\left(\frac{F_{hd,Ed}}{K_{hd}}\right) + \left(\frac{d_{hole} - d_{fastener}}{2}\right)}$$

To enable comparison between this calculation, and the test-results, 40% F_{max} will be used as the load-level on the shear-wall in this calculation. This will result in a force on the hold-down of:

$$\mathrm{F_{hd,Ed}} = 7563{,}28 \cdot \frac{h}{b} = 7563{,}28 \cdot \frac{2500{,}00}{1250{,}00} = 15126{,}56 \text{ [N]}$$

$$K_{hd;reduced} = \frac{15126{,}56}{\left(\frac{15126{,}56}{574{,}36 \cdot 10^2}\right) + \left(\frac{5{,}50 - 5{,}0}{2}\right)}$$

$$K_{hd;reduced} = 294{,}66 \cdot 10^2 \text{ [N/mm]}$$

The influence of the fastener slip-modulus on the racking stiffness is calculated to be:

$$R_{f,Rd} = \frac{n_{panels} \cdot n_{faces} \cdot b_1}{2(1 + \frac{h_1}{b_1})} \cdot \frac{K_{ser}}{s}$$

$$R_{f,Rd} = \frac{1 \cdot 2 \cdot 1250{,}00}{2(1 + \frac{2500{,}00}{1250{,}00})} \cdot \frac{584{,}58}{50{,}00}$$

$$R_{f,Rd} = 4871{,}50\ N/mm$$

The influence of the sheathing shear-modulus G on the racking stiffness is calculated to be:

$$R_{G,Rd} = \frac{n_{panels} \cdot b_1 \cdot t}{h_1} \cdot G_{mean}$$

$$R_{G,Rd} = \frac{1 \cdot 1250{,}00 \cdot 12{,}50}{2500{,}00} \cdot 700{,}00$$

$$R_{G,Rd} = 4375{,}00\ N/mm$$

The influence of the hold-down stiffness K_{hd} on the racking stiffness is calculated to be:

$$R_{hd,Rd} = \frac{(n_{panels} \cdot b_1)^2}{h_1^2} \cdot K_{hd}$$

$$R_{hd,Rd} = \frac{(1 \cdot 1250{,}00)^2}{2500{,}00^2} \cdot 294{,}66 \cdot 10^2$$

$$R_{hd,Rd} = 7366{,}50\ N/mm$$

The influence of the compression perpendicular to grain in the bottom rail on the racking stiffness is calculated to be:

$$R_{c,Rd} = \frac{(n_{panels} \cdot b_1)^2}{h_1^2} \cdot K_{c,90}$$

$$R_{c,Rd} = \frac{(1 \cdot 1250{,}00)^2}{2500{,}00^2} \cdot 9310{,}00$$

$$R_{c,Rd} = 2327{,}5\ N/mm$$

The influence of the flexural stiffness on the racking stiffness is calculated to be:

$$R_{str,Rd} = \frac{n_{studs} \cdot b_2 \cdot h_2 \cdot \left(n_{panels} \cdot b_1\right)^2}{\mathrm{h}_1^3} \cdot E_2$$

$$R_{str,Rd} = \frac{1 \cdot 40{,}00 \cdot 100{,}00 \cdot (1 \cdot 1250{,}00)^2}{2500{,}00^3} \cdot 11000{,}00$$

$$R_{be,Rd} = 4400{,}00\ N/mm$$

Slip between bottom rail and substructure is prevented by the test-rig. Consequently, this effect is not taken into account in the calculation of the timber-frame shear-wall stiffness.

With use of the calculated separate contributions, the stiffness of the shear-wall element can be calculated:

$$R_{Rd} = \frac{1}{\frac{1}{4871{,}50} + \frac{1}{7366{,}50} + \frac{1}{2327{,}50} + \frac{1}{4375{,}00} + \frac{1}{4400{,}00}}$$

$$R_{Rd} = 815{,}32\ N/mm$$

This value, R_{Rd}, for the racking stiffness will be used in a modelling approach to calculate the deflections at the load-level of $F_{v,Ed} = F_{04} = 7563{,}28$ N.

5.4.3 Structural model of the shear-wall

The modelling approach, and analytical method were used, to calculate the deflection of the shear-wall in a standard program for structural analysis. The type of software that is used is available within every engineering company. In this case Technosoft®Raamwerken is used.

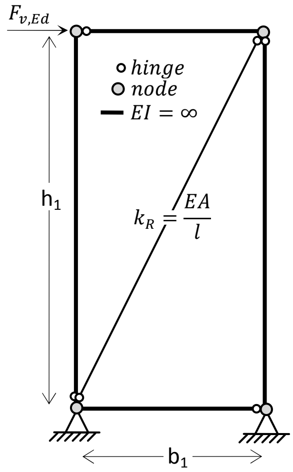


figure 85: Model made in Technosoft Raamwerken

The timber framing elements are considered to be of infinite stiffness, and are connected to the nodes by means of hinged connections. Only the diagonal brace is of finite stiffness. In the proposed modelling approach the stiffness of this diagonal can be calculated to be:

$$k_{R_{Rd}} = R_{Rd} \cdot (1 + \frac{h^2}{b^2})$$

$$k_{R_{Rd}} = 815{,}32 \cdot (1 + \frac{2500{,}00^2}{1250{,}00^2})$$

$$k_{R_{Rd}} = 4076{,}59 \text{ N/mm}$$

In the model use was made of the standard materials library, from which timber with strength class C18 was selected for the diagonal. With a length $l = \sqrt{h_1^2 + b_1^2} = 2795{,}08 \text{ mm}$, the cross-section of the diagonal will measure:

$$A = \frac{k_{R_{Rd}} \cdot l}{E\ (C18)} = \frac{4076{,}59 \cdot 2795{,}08}{9000{,}00} = 1266{,}04 \text{ mm}^2$$

5.4.4 Calculation with structural framework program

The calculated deflection measured 9,1 mm.

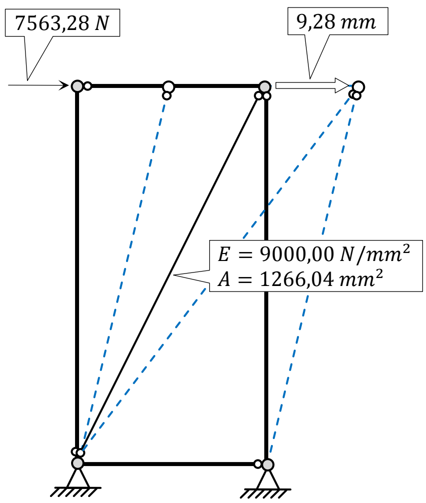


figure 86: Results of the calculation

If the results of modelling and analytical calculation are compared with the test-results, it can be concluded that the proposed method results in good agreement with the tests. In the table below this comparison is shown.

	Racking stiffness [N/mm]	Racking deformation at F_{04} [mm]
Analytical calculation & modelling	815,32	9,28
Test-result	804,56	9,40

table 16: Test and calculation results for test (10.1).

5.5 Conclusion

In this paragraph research question five will be answered with use of brief conclusions. Reference is made to the paragraphs before which are the basis for these conclusions.

Research question:

> 5. *Is it possible to suggest a modelling approach for structural analysis of timber-frame shear-wall racking stiffness, using a regular framework program?*
>   a. *How does the model look like?*
>   b. *For which purposes is the modelling approach to be used?*

Conclusion 5.:

> *It is possible to suggest a modelling approach that is suitable for structural analysis with use of regular framework programs. This can be concluded from the calculation and modelling example in which agreement was proven between test-results, calculation-results, and modelling-results.*

Conclusion 5.a.:

> *The model will be a truss type model in which the elements are connected by means of hinges. The general geometrical dimensions of the actual shear-wall are equal to the overall height and width of the representative truss in the model. The vertical and horizontal bar elements are considered to be of infinite stiffness, while the diagonal brace will be used to represent the shear-wall properties. This diagonal brace will be of a specific axial stiffness to inherit the specific timber-frame shear-wall parameters like: fastener-slip modulus, fastener spacing, sheathing panel thickness, shear-modulus, hold-down stiffness, etc. Reference is made to the figures and equations in the paragraphs 5.1 and 5.2.*

Conclusion 5.b.:

> *The modelling approach can be used for purpose of modelling the multi-storey timber-frame shear-wall elements racking stiffness. This can be the stiffness in serviceability limit state (SLS), as well as the shear-wall racking stiffness in ultimate limit state (ULS). Application of the modelling approach for the structural design of multi-storey buildings can be very useful for example. It is also possible to analyse a timber-frame floor diaphragm with use of the analytical calculation method and the proposed modelling approach. In such a case the model can provide insight in the distribution of wind-loading over the shear-walls.*

5.6 Recommendations

In this chapter a truss-type modelling approach was presented and analysed. Some issues remained unverified in this master's thesis.

- Including vertical loading in the model can disturb the correct results of the model. The calculation method and modelling approach are aimed at the calculation of the horizontal racking stiffness. The way in which the hold-down stiffness is incorporated in the diagonal brace can give incorrect results when trying to study the 2^{nd} order effect because of vertical loading for example. Therefore, it is recommended to study the possibilities of this modelling approach for including vertical loading in the model. Which difficulties arise when adding vertical loading to the model, and how can these be solved?

6 Racking stiffness of perforated timber-frame elements

In the chapters before an analytical method is presented this can be used to calculate the timber-frame shear-wall stiffness. By comparison with results of tests on full-scale shear-walls, it is shown that the method can be used to determine the racking stiffness for a standard timber-frame shear-wall element. In this chapter the perforated timber-frame shear-wall element is analysed. In the literature review test-results were found of shear-wall elements containing doors and windows. Two of these tests are used to provide the geometry and properties of a perforated shear-wall for analytical calculation and modelling. Three different approaches will be used to estimate a stiffness value for the perforated shear-walls. These analysis will lead to an answer for the research question:

> 6. *Is it possible to use the proposed analytical calculation method and modelling approach to analyse perforated shear-walls?*

6.1 Perforated shear-wall tests

A number of tests can be found when looking for the experimental research on perforated shear-walls. Among others, Bo Källsner did destructive wall tests to judge the validity of his calculation model. One of these tests will be used to evaluate the usefulness of the analytical method and modelling approach presented in the chapters before.

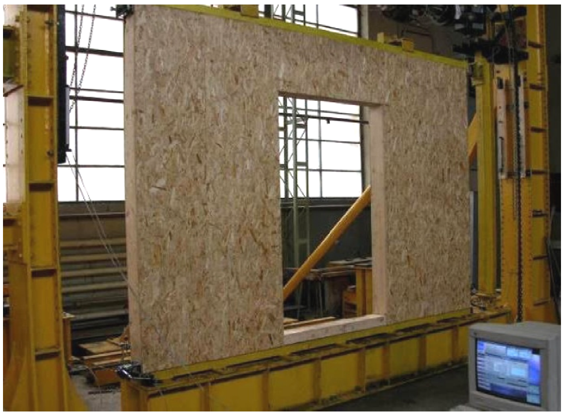


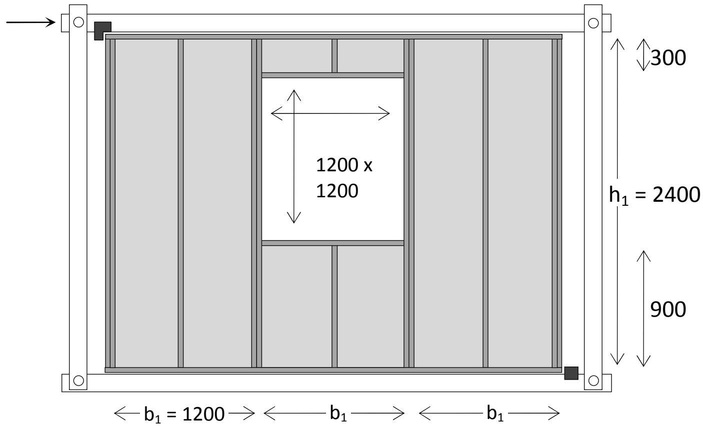
figure 87: Illustrative test-setup to determine the load-displacement characteristics of a perforated timber-frame shear-wall.
(Fülöp et al., 2006)

Källsner conducted a series of tests on shear-walls with different geometry and materials to validate a model for the static analysis of timber-frame shear-walls. The tested timber-frame shear-walls, considered in this chapter, have the following properties:

Test:	**(14.1)**	**(14.3)**	**(14.8)**	**(14.15)**	**(14.16)**
Sheathing material:	Gypsum paper board t = 12,5 mm (Gyproc)	Gypsum paper board t = 12,5 mm (Gyproc)	Gypsum paper board t = 12,5 mm (Gyproc)	Particleboard t = 12,3 mm	Particleboard t = 12,3 mm
Fastener:	Screw Ø3,0 x 37,6 s = 200 mm	Screw Ø3,0 x 37,6 s = 200 mm	Screw Ø3,0 x 37,6 s = 200 mm	Nail Ø2,13 x 50,1 s = 100 mm	Nail Ø2,13 x 50,1 s = 100 mm
Diaphragm dimensions:	1200 x 2400 mm	3600 x 2400 mm	3600 x 2400 mm	3600 x 2400 mm	3600 x 2400 mm
Perforation dimensions:	none	none	window: 1200 x 1200 mm	none	window: 1200 x 1200 mm

table 17: Properties of perforated and non-perforated timber-frame shear walls in tests, according to NEN-EN 594.

The lay-out of the shear-walls, and the test-results are shown on the next page. In the tests performed by Källsner no typical hold-down anchorage is applied. The function of the hold-down connector, preventing the timber-frame shear-wall element from uplifting is fulfilled by the test-rig. In figure 88 is shown how the element is prevented from uplift with use of a hold-down provision in the top-left area of the timber-frame element. In paragraph 4.2.2 is explained that for the tests done by Källsner, the hold-down stiffness is equal to the stiffness perpendicular to grain below the compressed stud (trailing stud). This stiffness value is estimated at: $9{,}3 \cdot 10^3$ N/mm.



The timber-frame element consists of 3 panels sheeted on one face of the element.

In the centre panel a window perforation is made measuring 1200 x 1200 mm

Provisions to prevent uplift of the element on top of the leading stud, and slip of the bottom rail are shown in the figure.

figure 88: Geometry and properties of the perforated shear-walls tested by (Källsner, 1984).

The perforated shear-wall elements were tested on a test-rig similar to what is shown in figure 87. Load and displacement were recorded during the test and are displayed below.

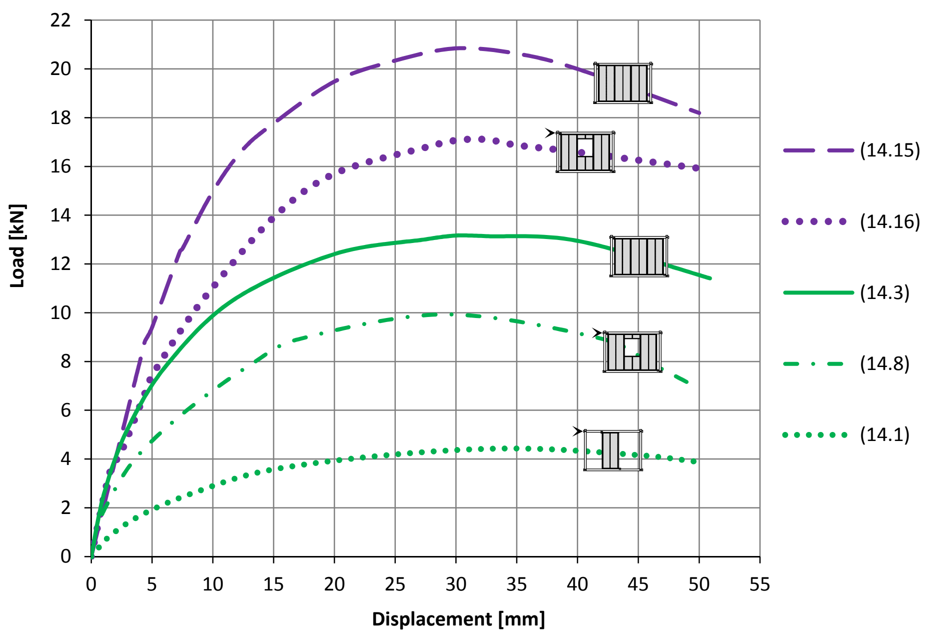


figure 89: Test-results: load-displacement graphs for four tested configurations (Källsner, 1984).

With use of the standard NEN-EN 594 the stiffness of the shear-walls can be calculated. These stiffness values are shown in table 18:

Test:	(14.1)	(14.3)	(14.8)	(14.15)	(14.16)
F_{max} [N] (Ultimate load in the test)	$4{,}4 \cdot 10^3$	$1{,}3 \cdot 10^4$	$9{,}9 \cdot 10^3$	$2{,}1 \cdot 10^4$	$1{,}7 \cdot 10^4$
F_{04} [N] (40 % of F_{max})	$1{,}8 \cdot 10^3$	$5{,}3 \cdot 10^3$	$4{,}0 \cdot 10^3$	$8{,}3 \cdot 10^3$	$6{,}8 \cdot 10^3$
F_{02} [N] (20% of F_{max})	$8{,}9 \cdot 10^2$	$2{,}6 \cdot 10^3$	$2{,}0 \cdot 10^3$	$4{,}2 \cdot 10^3$	$3{,}4 \cdot 10^3$
v_{04} [mm] (Deformation at 40% F_{max} load-level)	4,4	3,0	3,6	4,2	4,5
v_{02} [mm] (Deformation at 20% F_{max} load-level)	1,6	1,1	1,1	2,1	1,5
R_{test} [N/mm] (Based on F_{04} , F_{02})	$3{,}2 \cdot 10^2$	$1{,}4 \cdot 10^3$	$8{,}2 \cdot 10^2$	$2{,}0 \cdot 10^3$	$1{,}1 \cdot 10^3$

table 18: Stiffness of the perforated and non-perforated timber-frame shear walls in tests, according to NEN-EN 594.

6.2 Modelling methods for analysis of perforated shear-wall elements

Aim of this chapter is to evaluate the applicability of the analytical calculation method and modelling approach, to the perforated timber-frame shear-wall. Is it possible to determine the racking stiffness of a perforated shear-wall with use of the calculation method proposed in chapter 4, and the modelling approach as shown in chapter 5?

Three methods will be used to estimate the racking stiffness of the perforated shear-wall element in test (14.8) and test (14.16).

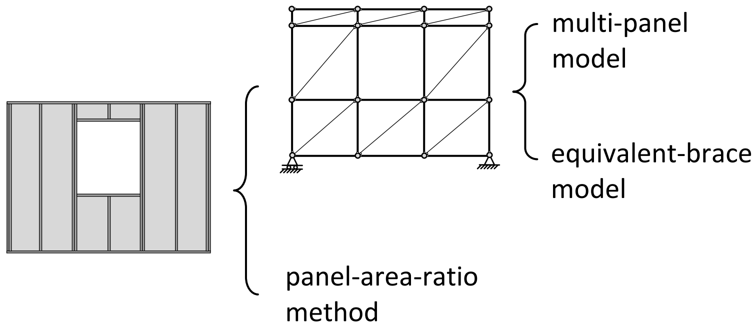


figure 90: Three methods to determine the stiffness of the perforated shear-wall element.

First, the applicability of the panel-area-ratio method will be shown. Secondly, the analytical method will be used to estimate a stiffness value for the perforated shear-wall using a truss-type model. The truss-type models can be made in two ways. In the first approach, the shear-wall element is divided in a number of sections for which a diagonal stiffness will be determined separately using the analytical method, this will be denoted as the multi-panel approach. The second truss-type modelling approach will assume a standard non-perforated shear-wall element at first. With use of the analytical calculation method a value for the racking stiffness will be determined. This stiffness value for the shear-wall will be translated to a so-called equivalent-brace model of the shear-wall. After that, the geometry of the window is added by removing one of the braces. This method will be denoted as the equivalent-brace approach. Both the multi-panel and equivalent-brace approach will be explained further in paragraph 6.2.2.

6.2.1 Panel-area-ratio method developed by Sugiyama and Yasumura

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 of a perforated shear-wall can be determined as following:

$$\mathrm{R}' = \frac{\mathrm{r}}{3 - 2\mathrm{r}} \cdot \mathrm{R} \qquad 6.1$$

in which: R' is the reduced racking stiffness of a perforated shear-wall
R is the racking stiffness of the non-perforated shear-wall

and:

$$\mathrm{r} = \frac{1}{1 + \frac{\alpha}{\beta}} = \frac{\mathrm{h} \cdot \sum \mathrm{b_i}}{\mathrm{h} \cdot \sum \mathrm{b_i} + \sum \mathrm{A_i}} \qquad 6.2$$

in which: r is panel area ratio
h is height of the wall element

b	is length of the wall element
$\sum b_i$	is length of full height wall segments
$\sum A_i$	is sum of openings

$$\alpha = \frac{\sum A_i}{h \cdot b}$$ is ratio of openings in wall element

$$\beta = \frac{\sum b_i}{b}$$ is ratio of full wall segments

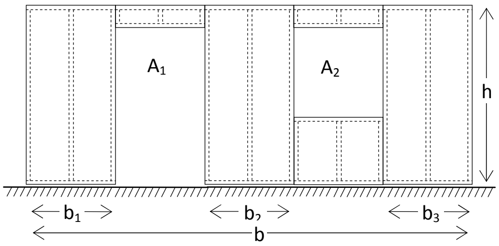


figure 91: Definition of parameters for calculating ‘panel area ratio’.

The method can be visualized with use of the graph below. In this graph the ratio between the stiffness of the standard, and perforated shear-wall is given as a function of the panel area ratio r:

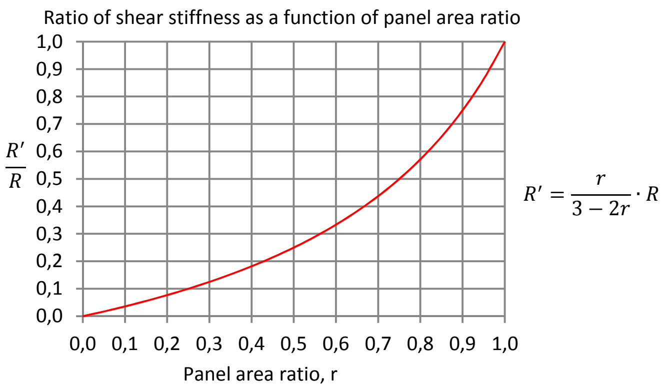


figure 92: Graph displaying the relation between shear stiffness ratio and panel area ratio.

The panel-area-ratio method can be used to estimate the stiffness of the perforated shear-walls based on the result of tests on the non-perforated shear-walls:

Test:	**(14.3)**	**(14.15)**
R_{test} [N/mm] (Based on F_{04} , F_{02})	$14 \cdot 10^2$	$20 \cdot 10^2$
Test:	**(14.8)**	**(14.16)**
α	0,17	0,17
β	0,67	0,67
r	0,80	0,80
$R'_{panel\text{-}area\text{-}ratio}$ [N/mm]	775	1142
R_{test} / $R'_{panel\text{-}area\text{-}ratio}$ [-]	1,06	0,99

table 19: Stiffness of the perforated timber-frame shear walls according to the panel-area-ratio method.

This calculation can also be made again using the racking stiffness of the non-perforated shear-wall, as was calculated with the analytical calculation method:

Test:	(14.3)	(14.15)
$R_{analytical}$ [N/mm]	1441	2231

Test:	(14.8)	(14.16)
α	0,17	0,17
β	0,67	0,67
r	0,80	0,80
$R'_{panel\text{-}area\text{-}ratio}$ [N/mm]	823	1275

R_{test} / $R'_{panel\text{-}area\text{-}ratio}$ [-]	1,00	0,89

table 20: Stiffness of the perforated timber-frame shear walls according to the panel-area-ratio method.

Two examples show that the panel-area-ratio method result in reasonable values for the racking stiffness when using the non-perforated stiffness value from testing, as well as from analytical calculation.

In the following paragraph, for both tests (14.8) and (14.16), the racking stiffness of the shear-wall will be determined using the multi-panel method, and equivalent-brace method.

6.2.2 Modelling perforated timber-frame shear-walls

The shear-wall geometry which was shown in figure 88 (page 92), will be translated in a model as is shown in the figure below.

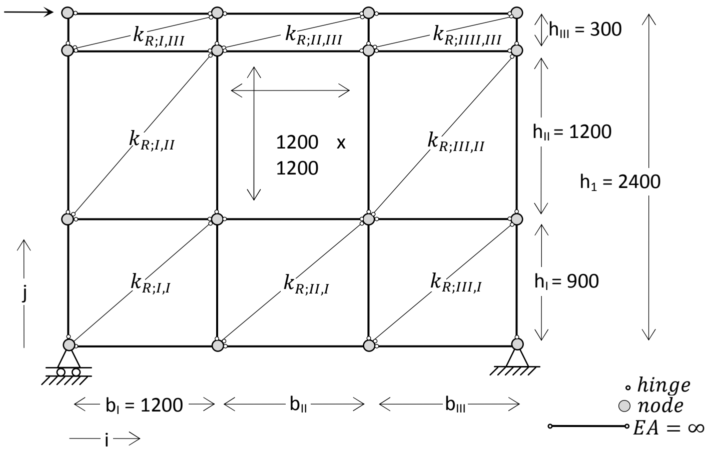


figure 93: Definition of elements and parameters in the model.

In the figure can be seen that a stiffness parameter k_R is used as stiffness for the braces. These braces incorporate the specific shear-wall properties in the model. Geometrical dimensions of shear-wall element and opening do not change. The stiffness of the braces can be determined with use of the analytical calculation method. Two different perspectives can be chosen, the multi-panel approach, and the equivalent-brace approach. These two methods will be explained on the next page.

6.2.3 Multi-panel approach

In the multi-panel approach the element will be treated as if it is an assembly of separate panels. The geometry of the panel division depends on the geometry of the perforation. For each separate panel the representative diagonal stiffness $k_{R;i,j}$ is calculated using the analytical method as shown in paragraph 4.8. The multi-panel modelling method is demonstrated with use of the figure below.

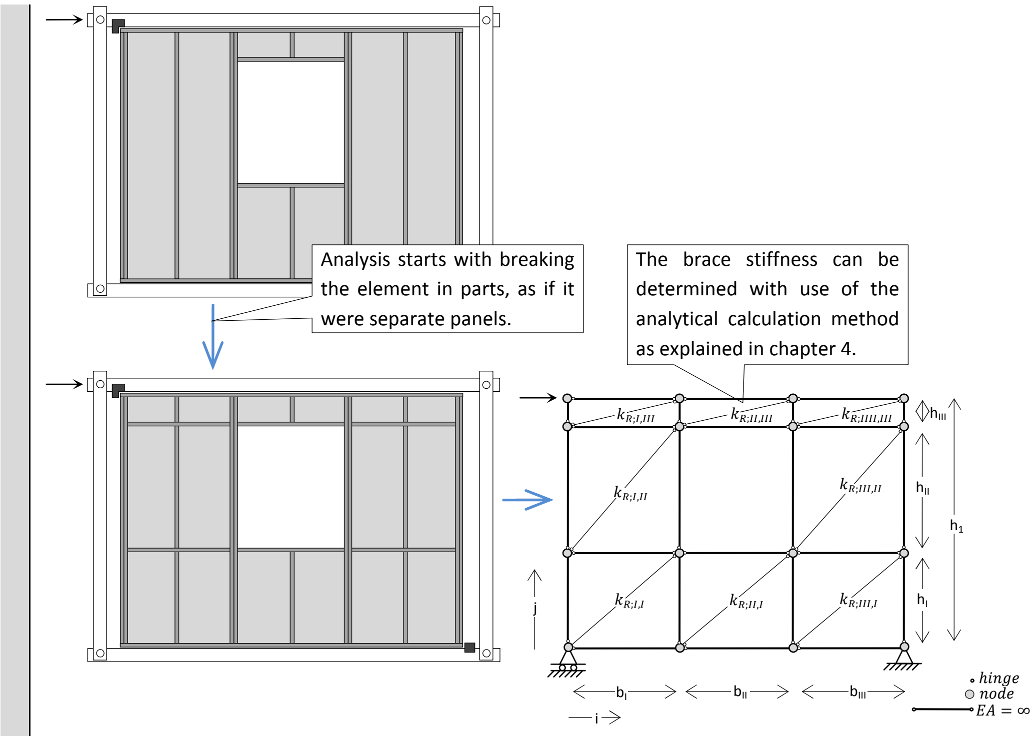


figure 94: Multi-panel model of perforated timber-frame shear-wall element.

Fastener component in the multi-panel model

In the multi-panel approach, the fastener contribution have to be calculated in a different way as shown in the analytical calculation method. In the figure below a standard timber-frame shear-wall panel is divided into three parts. The fasteners are indicated with use of the blue dotted line. As can be seen from the figure, for panel part I,II there are only two vertical fastener rows. In the analytical calculation method, fasteners are assumed to be applied along the perimeter of the panel. Consequently the equation to calculate the fastener component of the shear-wall stiffness have to be adapted.

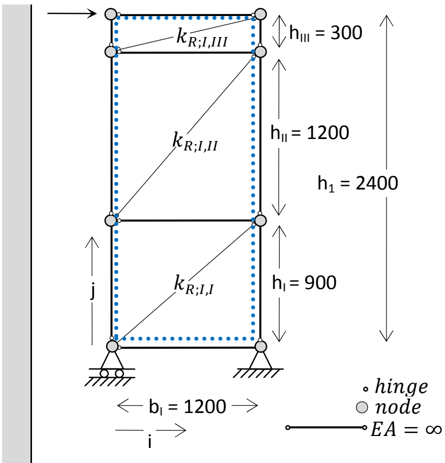


figure 95: Model test (14.1)

With respect to the fasteners, it can be seen that $k_{R;I,I}$ and $k_{R;I,III}$ will contain the effect of two vertical fastener rows, and one horizontal fastener row. $k_{R;I,II}$ will contain the effect of only two vertical fastener rows. From paragraph 4.2.1 can be seen how the equation for the fastener-slip is derived:

$$R_{f,Rd} = \frac{b_1}{2(1+\frac{h_1}{b_1})} \cdot \frac{K_{ser}}{s} \qquad 6.3$$

The factor h_1 / b_1 is added to the equation to take in account the horizontal effect of fastener-slip in the vertical rows. The factor 2 in the equation is added because in the standard panel the effect is always present twice. Slip will occur in the horizontal fastener-row in the bottom rail, and slip will be seen in the horizontal fastener-row in the top-rail. Slip of the vertical fastener-row in the leading stud, and slip in the vertical fastener-row in the trailing stud. Keeping in mind, the way in which the equation for the fastener contribution was derived, the following relations can be derived for the effect of fasteners in $k_{R;I,I}$, $k_{R;I,II}$, and $k_{R;I,III}$, can be determined as following:

I,I: one horizontal row, two vertical rows:

$$R_{f,Rd} = \frac{b_{I,I}}{(1 + \frac{2 \cdot h_{I,I}}{b_{I,I}})} \cdot \frac{K_{ser}}{s}$$

I,II: two vertical rows:

$$R_{f,Rd;I,II} = \frac{b_{I,II}}{\left(\frac{2 \cdot h_{I,II}}{b_{I,II}}\right)} \cdot \frac{K_{ser}}{s}$$

I,III: one horizontal row, two vertical rows:

$$R_{f,Rd;I,III} = \frac{b_{I,III}}{(1 + \frac{2 \cdot h_{I,III}}{b_{I,III}})} \cdot \frac{K_{ser}}{s}$$

In the multi-panel approach the fastener contribution in the racking-stiffness will be taken into account as presented above.

In the last part of 0, an analysis is made for the case if the fastener-equation was not modified. Although such a choice might seem incorrect after the explanation and derivation above, it turned out that ignoring the modification of the fastener equation does not have to result in bad results necessarily. For reason of simplicity one might limit the amount of work by neglecting the modification of the fastener equations, as it was explained in this paragraph.

Effect of the multi-panel modelling approach on racking stiffness

The multi-panel modelling approach will have a certain deviation due to the division of the timber-frame shear-wall element in smaller panels. The effect of this deviation on the racking-stiffness of the model can be shown with use of three different multi-panel models for test (14.1):

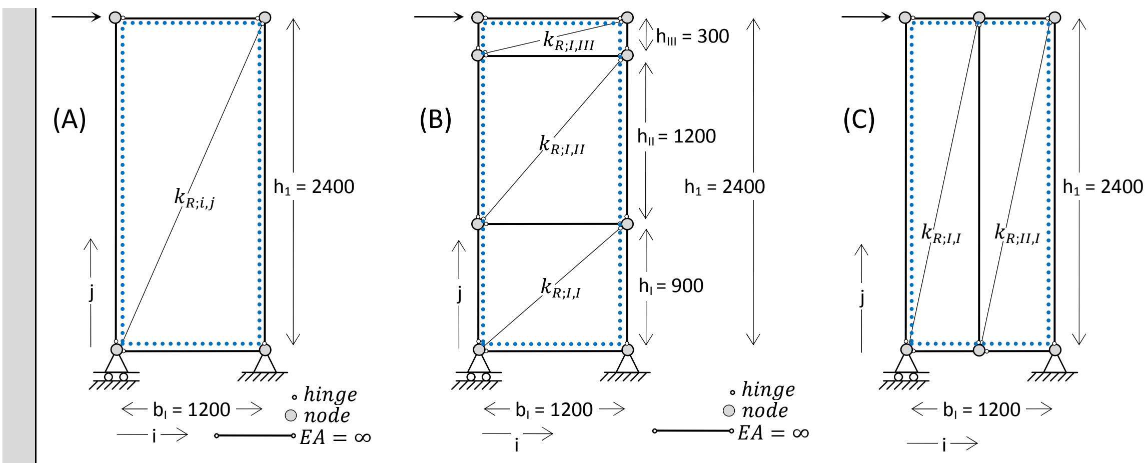


figure 96: Three different multi-panel models for test (14.1).

In the table below the results of the three different multi-panel models are shown. The calculations and TechnosoftRaamwerken® models, which were basis for these results, are appended in Appendix XII.

Alternative model for test (14.1)	(A)	(B)	(C)	
R_{model} [N/mm]	$\frac{1772}{4{,}96} = 357{,}26$ (111%)	$\frac{1772}{4{,}68} = 378{,}63$ (118%)	$\frac{1772}{4{,}19} = 422{,}91$ (132%)	$R_{test} = 32 \cdot 10^1$ (100%) $R_{analytical} = 357{,}87$ (112%)

table 21: Results of multi-panel modelling approaches test (14.1)

As can be seen from table 21, the analytically determined racking stiffness is 12% higher than the racking stiffness determined with use of test-based load-displacement data. Also the multi-panel modelling approach gives somewhat higher racking-stiffness, on one hand this can be explained by the use of the analytical calculation method that result in a 12% higher racking stiffness in this case, but also the effect of the division into smaller panels turned out to given an additional increase in the model's racking stiffness.

Before analysing the perforated shear-walls with use of the multi-panel modelling approach, the equivalent-brace modelling approach will be explained.

6.2.4 Equivalent-brace approach

In the equivalent-brace approach, a value for the racking stiffness of the perforated shear-wall element is calculated with use of the analytical calculation method shown in paragraph 4.8. In this calculation the shear-wall element will be considered to be non-perforated. A model will be chosen similar as in the multi-panel approach, the stiffness of the braces however will be calculated with use of $R_{analytical}$, and the equation below.

$$k_{R;i,j} = \frac{h_1}{h_j} \cdot \frac{b_i}{b_1} \cdot R_{analytical} \cdot \left(1 + \frac{h_j^2}{b_i^2}\right) \tag{6.4}$$

With use of this equation, the racking stiffness of the non-perforated timber-frame element, $R_{analytical}$, is divided over a number of braces in such a way that the model represents exactly the same stiffness $R_{analytical}$. The geometry of the perforation can be included in the model by removing the brace representing the location of the perforation. The equivalent-brace modelling approach is demonstrated with use of the figure below.

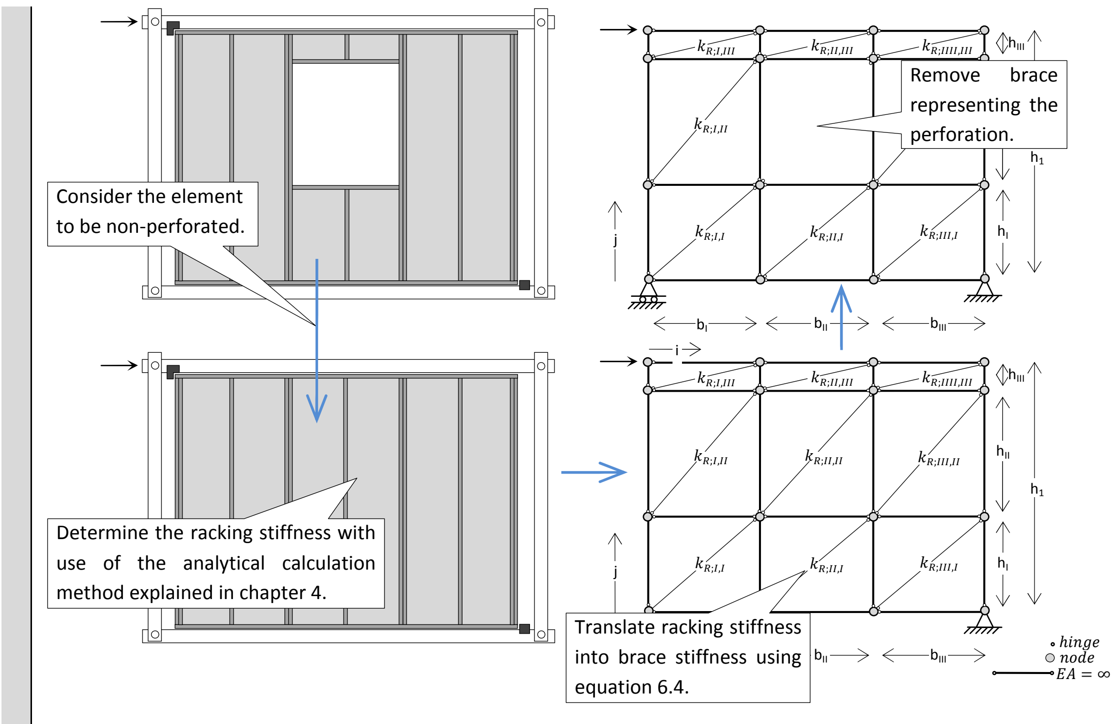


figure 97: Equivalent-brace model of a perforated timber-frame shear-wall element.

Dividing the racking stiffness over equivalent-braces

Without giving notice it was stated that the stiffness of the braces could be determined with use of the following equation:

$$k_{R;i,j} = \frac{h_1}{h_j} \cdot \frac{b_i}{b_1} \cdot R_{analytical} \cdot \left(1 + \frac{h_j^2}{b_i^2}\right) \qquad 6.5$$

In the equation, the analytically calculated racking stiffness of the non-perforated shear-wall is divided over a number of braces such that it results in a truss model with equivalent stiffness. The equation was derived step-by-step. First the standard timber-frame shear-wall from test (14.1) was modelled using the modelling approach presented in paragraph 5.1, using the matching equation derived in the same chapter:

$$k_R = R_{analytical} \cdot \left(1 + \frac{{h_1}^2}{{b_1}^2}\right) \qquad 6.6$$

After this, it was investigated how to change the equation in such a way that the racking stiffness could be divided over more than one brace, as shown in the figure below.

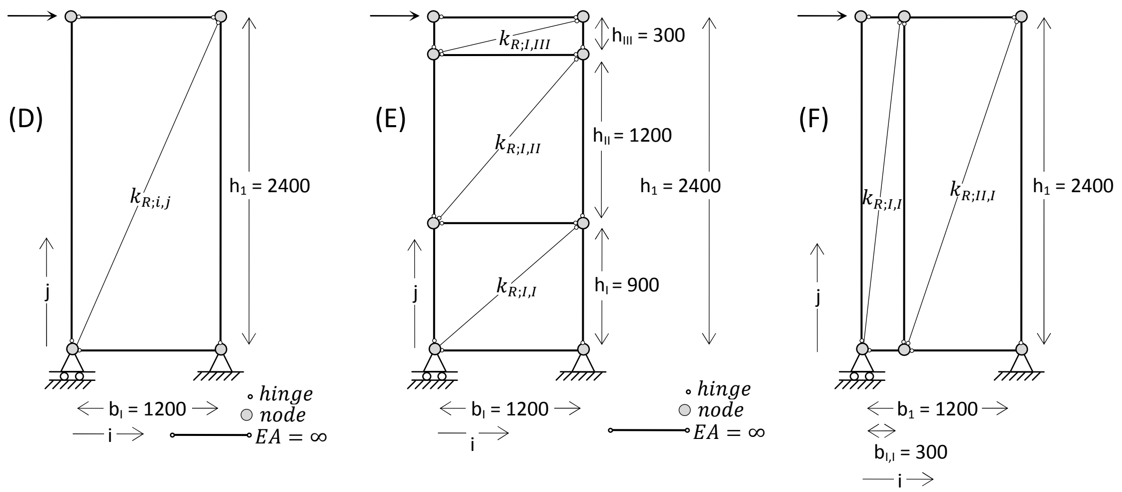


figure 98: Three different equivalent-brace models for test (14.1).

In case of equivalent-brace model (E), the horizontal racking load to be transferred by the braces is the same for all *j* parts. The braces can be considered as a serial spring-system. On the condition that the displacement of the panel remains a straight line, it was found that the racking stiffness of the timber-frame element as a whole could be divided by the ratio of h_1 / h_j. This resulted in the following equation:

Dividing the element in *j* full-width parts (E):

$$k_{R;j} = \frac{h}{h_j} \cdot R_{analytical} \cdot \left(1 + \frac{h_j^2}{b_1^2}\right) \qquad 6.7$$

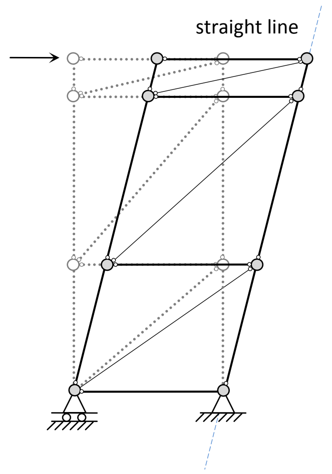


figure 99: Equivalent-brace model (E) test (14.1)

In case of equivalent-brace model (F), the horizontal displacement is the same for all *i* parts. The braces can be considered to be a system of parallel springs. On the condition that the combined stiffness of both parts is equal to the timber-frame element racking stiffness, it was found that the stiffness of a brace depends on the ratio b_i / b_1. This resulted in the following equation:

Dividing the element in *i* full-height parts (F):

$$k_{R;j} = \frac{b_i}{b} \cdot R_{analytical} \cdot \left(1 + \frac{h_1^2}{b_i^2}\right) \tag{6.8}$$

In the table below the results of the three different equivalent-brace models are shown. The calculations and TechnosoftRaamwerken® models, which were basis for these results, are appended in Appendix XII.

Alternative model for test (14.1)	**(D)**	**(E)**	**(F)**	
R_{model} [N/mm]	$\frac{1772}{4,96} = 357,26$ (111%)	$\frac{1772}{4,97} = 356,54$ (111%)	$\frac{1772}{4,97} = 356,54$ (111%)	$R_{test} = 32 \cdot 10^1$ (100%) $R_{analytical}$ = 357,87 (112%)

table 22: Results of equivalent-brace modelling approach test (14.1)

From table 22 can be concluded that dividing the racking stiffness over a number of equivalent braces does not affect the racking-stiffness. The equations are proven to give correct results.

For the modelling of larger timber-frame shear-wall elements it was required to combine equations 6.7 and 6.8. The element can be divided horizontally and vertically to an equivalent system of braces. The stiffness of these braces can be calculated with use of the following equation:

Dividing the element in *i x j* parts:

$$k_{R;i,j} = \frac{h_1}{h_j} \cdot \frac{b_i}{b_1} \cdot R_{analytical} \cdot \left(1 + \frac{h_j^2}{b_i^2}\right) \tag{6.9}$$

Equation 6.9 was made by combining equations 6.7 and 6.8.

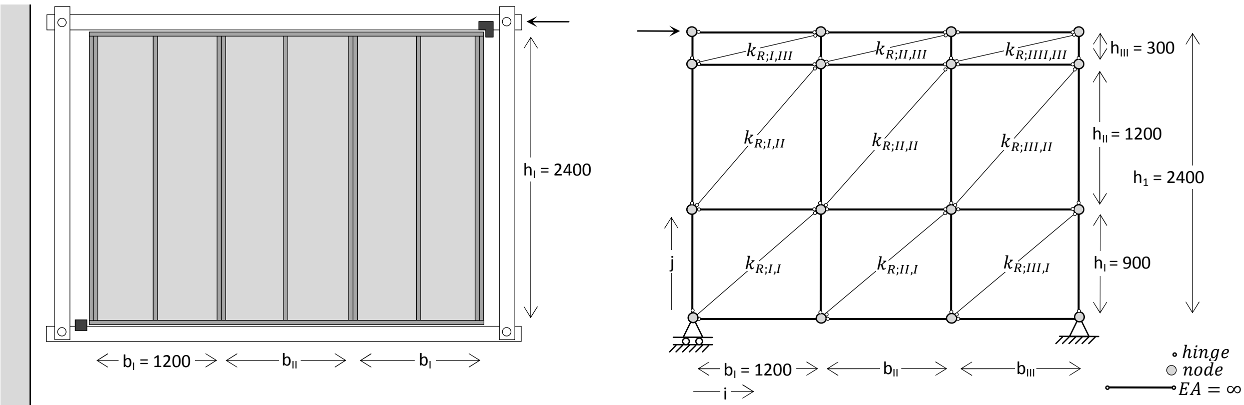


figure 100: Equivalent-brace model of a timber-frame shear-wall element.

Now the use of both modelling approaches is explained, two perforated shear-walls will be analysed. In the literature review load-displacement data of experimental testing on perforated timber-frame shear-walls was found, as well as test-results on their non-perforated counterparts. The load-displacement graphs of these tests were already shown in figure 89 (page 92). The result of analysis with the panel-area-ratio method was reported in paragraph 6.2.1 (page 93).

In the following paragraph the results of the multi-panel and equivalent-brace modelling approaches will be compared with the results of the panel-area-ratio method, and the test-based racking stiffness values.

6.3 Comparison

In this paragraph, three methods for analysis and modelling of perforated timber-frame shear-walls will be compared. These methods were explained in paragraph 6.2, and used to receive several alternative values for the racking stiffness of two perforated timber-frame shear-walls. Test-based load-displacement data of racking tests on these shear-walls were available from the literature review. In this paragraph the results of the panel-are-ratio method, the multi-panel model, and equivalent-brace modelling approach will be compared.

6.3.1 Observed agreement between test-based and modelling-based racking stiffness

The prediction of the racking-stiffness for tests (14.8) and (14.16), which were made with use of the three alternative methods, are based on the analytically calculated racking-stiffness values for the shear-walls in tests (14.3) and (14.16). Reference is made to 0 for the models and calculations that were made with use of the analytical-calculation method, and the models mad in TechnosoftRaamwerken®.

As already explained, the timber-frame element in test (14.8) is similar to the timber-frame element in test (14.3) except for the window perforation, as well as the shear-wall in test (14.16) is similar to the shear-wall in test (14.15) except for the window perforation.

Test:	(14.3)		(14.8)		(14.15)		(14.16)	
R_{test}	$14 \cdot 10^2$	100%	$82 \cdot 10^1$	100%	$20 \cdot 10^2$	100%	$11 \cdot 10^2$	100%
$R_{analytical}$	1440,66	106%			2476,82	125%		
$R_{panel\text{-}area\text{-}ratio}$			823,00	100%			1275,00	113%
$R_{multi\text{-}panel}$	1666,77	123%	934,82	114%	3231,78	163%	1864,03	165%
$R_{equivalent\text{-}brace}$	1435,15	106%	957,35	116%	2459,59	124%	1640,53	145%
				perforated				perforated

table 23: Stiffness [N/mm]of the (perforated) timber-frame shear wall elements according to three methods.

From table 23 can be seen that both the panel-area-ratio method and multi-panel model, as well as the equivalent-brace modelling approach, give comparable and reasonable results in predicting the effect of a perforation on the timber-frame shear-wall racking stiffness. In general nice agreement can be seen between test-based racking stiffness and the results of all three modelling approaches. It have to be noticed however that these conclusions are based on comparison of only 2 tests.

The deviation of 65% and 45% of the multi-panel model and equivalent-brace modelling approach for test (14.16) can be partly explained by the deviation between the analytically calculated racking stiffness, and test-based racking stiffness that can account for 25% of this difference.

6.4 Conclusion

Three different approaches were derived and compared to estimate the racking stiffness of perforated shear-walls. The analysis was made to provide an answer for the research question:

> 6. *Is it possible to use the proposed analytical calculation method and modelling approach to analyse perforated shear-walls?*

In conclusion can be said that:

Conclusion 6.:

> *Both the multi-panel and equivalent-brace modelling approaches derived in the context of this master’s thesis, as well as the panel-are-ratio method developed by Sugiyama, make it possible to use the analytical calculation method and modelling approach (as presented in chapter 4 and chapter 5) for analysis of perforated shear-walls.*

6.5 Recommendations

Based on the methods and results presented in this chapter, a few recommendations can be given:

- With respect to this master’s thesis only two comparisons could be made, between the result of modelling and analysis of perforated shear-walls, and test-based results. I would be useful if additional research is done to obtain more useful test-results from literature.

- It would be valuable if the applicability of both the multi-panel and equivalent-brace modelling approach, to the structural analysis of perforated timber-frame shear-walls is verified, for shear-walls containing multiple perforations such as doors and windows.

- Additional research in literature is needed to judge the applicability of the Sugiyama method for the calculation of the racking stiffness for perforated timber-frame shear-walls.

7 Conclusions and recommendations

In answer to the research questions shown in paragraph 1.5, the conclusions will be presented in this chapter. In addition to the conclusions a few recommendations can be given.

7.1 Conclusions

In the initial phase of the graduation project the following research question was postulated:

> *"Which parts or elements are of importance, and have to be included in the structural analysis and modelling of the timber-frame structure, in such a way that a useful prediction of deformations in serviceability limit state (SLS), and force distribution in ultimate limit state (ULS) can be made?"*

This main research question was divided into the sub questions in the introduction in paragraph 1.5. These sub questions will be answered below.

1. *A lot of information was found in literature with respect to analysis and modelling of timber-frame shear-wall racking stiffness. This information is reported in literature review, as can be seen in the table of contents.*

    a. *Most important parts influencing the timber-frame shear-wall racking stiffness are the fasteners along the perimeter of the shear-wall, and the hold-down anchor. Sufficient hold-down capacity is needed to prevent a disadvantageous internal force distribution in the timber-frame shear-wall element. In such a case the fasteners can perform best in transfer of shear-forces from the frame into the sheathing and back again.*

    b. *From the literature review load-displacement data was found resulting from experimental testing on (perforated) timber-frame shear-wall elements, hold-down products, and a variety of sheathing-to-timber fastener connections. These test-results are used for verification of the proposed analytical calculation method and modelling approach.*

The first research question was already answered in the literature and knowledge review[5]. From the literature study could be concluded that the fastener and hold-down conditions are the most influential parameters for the racking stiffness of the shear-wall element. Therefore, the fasteners and hold-down stiffness will be analysed first. This will be done by answering the following questions:

In chapter 0 the fastener slip-modulus has been analysed. Both the Eurocode 5 design rules, and experimental results were observed.

2. *It is possible to receive clarification about the fastener slip-modulus K_{ser} and K_u for calculating the racking stiffness of the timber-frame elements.*

    a. *Reasonable agreement can be seen between the code-based slip-modulus K_{ser}, and the test-based slip-modulus k_s. The slip-modulus K_{ser} for OSB sheathing-to-timber connections on average was 16 % higher, compared with k_s. The slip-modulus K_{ser} for Plywood sheathing-to-timber connections on average was 8 % higher, compared with k_s.*

        For gypsum-fibre board and gypsum-paper board the test-based slip-modulus k_s can be used. Reference is made to table 4 (page 35).

    b. *In the structural analysis of stiffness of timber-frame shear-wall, the calculated fastener-slip-modulus K_{ser} and the experimentally determined fastener-slip-modulus k_s may be used for serviceability limit state*

[5] *The literature review (11-10-2011) is the first part of the MSc thesis 'Multi-storey timber-frame building'. In this report load-displacement data from several sources in literature were brought together. Findings about the most important parts in the timber-frame shear-wall, and the characteristics of the behaviour, were reported as well.*

purposes such as calculating the deflections of the timber-frame shear-wall. For ultimate limit state purposes, such as determining the force distribution within the structure, both K_u and K_{ser} have to be evaluated, the governing value will be the least advantageous one. $K_u = 2/3 \cdot K_{ser}$ (or $2/3 \cdot k_s$)

In chapter 3 the stiffness of the hold-down anchorage has been studied. A comparison was made between analytical calculation of the hold-down stiffness and test-based results found from literature.

3. *It is possible to predict the hold-down stiffness with use of an analytical calculation method. This calculation method takes into account the stiffness of the steel and timber section, the stiffness of the fasteners, hole-clearance, and the actual load-level. For further guidance reference is made to page 46.*

    a. *Reasonable agreement was proven between the results of experimental testing found in literature, and the result of analytical calculation. The calculated values for the hold-down stiffness, using the fastener-slip-modulus K_{ser}, were 12% higher on average, compared with the stiffness values as determined from the load-displacement data of 16 tests.*

In chapter 4 an analytical calculation method was derived which can be used for the calculation of the timber-frame shear-wall racking stiffness.

4. *From the observation and discussion of calculation and test-results in chapter 4, it is shown to be possible to derive an analytical calculation method for the determination of the timber-frame shear-wall racking stiffness.*

    a. *The parameters and equations to be used in the analytical calculation method, to calculate the components that contribute to the racking deflection of the shear-wall, can be found in paragraph 4.8 (page 78).*

    The components to be taken into account for the calculation of the shear-wall racking deformation are listed below, and can be expressed with use of a percentage of the total racking deformation. These are average percentages based on the analysis of 31 tests:

u_f	:	*slip of fasteners along the panel perimeter*	:	*48%*
u_{sh}	:	*shear of the sheathing board material*	:	*12%*
u_{str}	:	*strain in leading & trailing stud*	:	*8%*
u_{hd}	:	*strain in the hold-down anchorage*	:	*13%*
u_c	:	*compression perpendicular to grain in the bottom rail*	:	*15%*

    b. *It became clear what the meaning is of the resulting racking stiffness values, and the purposes for which these values can be used. For this reason, the proposed analytical calculation method is useful in the structural design of a timber-frame multi-storey building*

    Reasonable agreement can be seen from the comparison between analytically calculated racking stiffness values, and the test-based racking stiffness values determined with use of the load-displacement data found in literature.

    If K_{ser} is used as the fastener-slip modulus in the analytical calculation method, the resulting racking stiffness value can be used for serviceability limit state (SLS) purposes such as calculating the deformation of the timber-frame shear-wall. For ultimate limit state (ULS) purposes, K_{ser} or K_u can be used in the analytical calculation method, the most disadvantageous result have to be used.

In chapter 5 an approach is presented for the modelling of timber-frame shear-wall racking stiffness, in which use is made of the analytical calculation method from chapter 4.

5. *It is possible to propose a modelling approach that is suitable for structural analysis with use of regular framework programs. This can be concluded from the calculation and modelling example in chapter 5, in which agreement was proven between test-results, calculation-results, and modelling-results.*

    a. *The model will be a truss type model in which the elements are connected by means of hinges. The general geometrical dimensions of the actual shear-wall are equal to the overall height and width of the representative truss in the model. The vertical and horizontal bar elements are considered to be of infinite stiffness, while the diagonal brace will be used to represent the shear-wall properties. The axial stiffness of this diagonal brace will inherit the specific shear-wall properties using the racking stiffness as calculated with use of the analytical calculation method derived in chapter 4. Reference is made to the figures and equations in the paragraphs 5.1 and 5.2 (page 81).*

    b. *The modelling approach can be used for purpose of modelling the multi-storey timber-frame shear-wall elements racking stiffness. This can be the stiffness in serviceability limit state (SLS), as well as the shear-wall racking stiffness in ultimate limit state (ULS). Application of the modelling approach for the structural design of multi-storey buildings can be very useful. It is also possible to analyse a timber-frame floor diaphragm with use of the analytical calculation method and the proposed modelling approach. In such a case the model can provide insight in the distribution of wind-loading over the shear-walls.*

In chapter 6 three different modelling approaches were derived and compared to estimate the racking stiffness of perforated timber-frame shear-walls.

6. *Both the multi-panel and equivalent-brace modelling approaches derived in chapter 6, as well as the panel-area-ratio method developed by Sugiyama, make it possible to use the analytical calculation method and modelling approach (as presented in chapter 4 and chapter 5) for analysis of perforated shear-walls. Reference is made to paragraph 6.2 (page 93).*

7.2 Recommendations

In addition to the conclusions some recommendations can be given.

As a result of the analysis of the fastener stiffness in chapter 0, a few recommendations can be given.

I: More tests are needed with respect to the sheathing-to-timber connection with particleboard. Only two tests were found in the research for the literature review, which is a basis to small for drawing a conclusion. More searching for experimental test-results on sheathing-to-timber connections would be useful, in the literature review a limited amount of time could be invested in searching for the results of these tests. It is expected that more experimental results can be found. (This can be relevant for the other sheathing materials too.)

II: The big spread in the results of the tests on gypsum-based sheathing-to-timber connections suggest that the stiffness of these type of connections not only depends on density, and fastener diameter. Other effects, like edge distances and penetration of the paper reinforcement, can play an important role. Perhaps more severe criteria have to be applied to the gypsum-based sheathing-to-timber connections compared with the wood-based sheathing-to-timber connections. The formulation of these criteria and additional (experimental) research could lead to the formulation of an equation to calculate a fastener-slip-modulus K_{ser} *for connections executed with gypsum-paper board, and gypsum-fibre board.*

As a result of chapter 3, the following can be recommended:

III: It can be useful to perform experimental testing on hold-downs with different lay-out. Take for instance hold-down solutions with a larger cross-sectional area of the steel, or hold-down configurations with less but more strong fasteners.

From the comparison between test-based determined racking stiffness values, and analytically calculated racking stiffness values in chapter 4, it was shown that there are some issues left for further research.

IV: Additional research (in literature) can be done to receive load-displacement data of tests on full-scale shear-wall elements sheathed with particleboard. The timber-frame shear-wall elements with these materials are not covered in the analysis and comparison reported about in this report.

V: It would be useful if the result of this master’s thesis can be expanded for the calculation and modelling of asymmetric timber-frame shear-wall elements racking stiffness. Asymmetric in this case implies the application of different sheathing and fastener configuration on both faces of the shear-wall element. In timber-frame practice, it is a common choice to choose for a gypsum-paper of gypsum-fibre board material on the interior side of the wall, and a wood-based panel on the cavity face of the shear-wall element. These walls however are not analysed in this master’s thesis. Tests on asymmetric shear-wall elements are available in literature.

In chapter 5 a truss-type modelling approach was presented and analysed. Some issues remained unverified in this master’s thesis.

VII: Including vertical loading in the model can disturb the correct results of the model. The calculation method and modelling approach are aimed at the calculation of the horizontal racking stiffness. The way in which the hold-down stiffness is incorporated in the diagonal brace can give incorrect results when trying to study the 2^{nd} order effect because of vertical loading for example. Therefore, it is recommended to study the possibilities of this modelling approach for including vertical loading in the model. Which difficulties arise when adding vertical loading to the model, and how can these be solved?

Three modelling approaches were presented for the calculation of the racking stiffness of perforated timber-frame shear-walls. Analysis of this modelling methods in chapter 6 was done using only two tests, found from literature. Therefore the following recommendations are given:

VIII: With respect to this master’s thesis only two comparisons could be made between the result of modelling and analysis of perforated shear-walls, and test-based results. I would be useful if additional research is done to obtain more useful test-results from literature.

XI: It would be valuable if the applicability of both the multi-panel and equivalent-brace modelling approach, to the structural analysis of perforated timber-frame shear-walls is verified, for shear-walls containing multiple perforations such as doors and windows.

X: It is expected that more information with respect to the Sugiyama panel-area-ratio method is present in literature. Additional research in literature is needed to receive this information, and to verify the applicability of the Sugiyama method for the calculation of the racking stiffness for perforated timber-frame shear-walls.

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Appendices

Appendix I Calculated and test-based slip-modulus k_s and K_{ser}

Fastener tests:								
Material:	OSB	OSB	OSB	OSB	OSB	OSB	OSB	OSB
Material thickness t: [mm]	9,5	9,5	9,5	9,5	9,5	11,11	11	11
Fastener properties: N = nail, Scr = Screw, St = Staple	N Ø2,5x50	N Ø2,87x50 (CN50)	N Ø3,0x51	N Ø3,3x63,5	N Ø3,8x76 (with washer)	N Ø2,87x63,5 (common 8d 0,113")	N Ø3,1x80 (spiral)	N Ø3,3x65 (8d SENKO®) (// to stud grain) (a_4 = 51 mm) (f_u = 827 Mpa)
Density ρ of material:	genomen: OSB/3 $\rho_k = \rho_m = 550$	genomen: OSB/3 $\rho_k = \rho_m = 550$	Performance rated W24 OSB. genomen: OSB/3 $\rho_k = \rho_m = 550$	Performance rated W24 OSB. genomen: OSB/3 $\rho_k = \rho_m = 550$	Performance rated W24 OSB. genomen: OSB/3 $\rho_k = \rho_m = 550$	genomen: OSB/3 $\rho_k = \rho_m = 550$	genomen: OSB/3 $\rho_k = \rho_m = 550$	genomen: OSB/3 $\rho_k = \rho_m = 550$
Remark:	Omhullende van cyclische beproevingen. framing lumber 38 x 89 mm Mean of 15 specimens	S-P-F framing lumber 38 x 89 mm Based on 6 specimens	Spruce-Pine-Fir framing lumber 38 x 89 mm Mean of 6 specimen	Spruce-Pine-Fir framing lumber 38 x 89 mm Mean of 6 specimen	Spruce-Pine-Fir framing lumber 38 x 89 mm Mean of 6 specimen	Hemlock-Fir framing lumber Mean of 15 specimen	SPF (Spruce-Pine-Fir) framing lumber 45 x 150 mm Mean of 3 specimen	framing lumber 38 x 89 mm (SPF) mean of 10 specimen

Reference:	(11.1)		(1.2)		(6.1)		(6.2)		(6.3)		(5.3)		(3.5)		(9.1)	
D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	6,3E-01	4,3E+02	1,3E+00	6,9E+02	1,0E+00	5,7E+02	1,0E+00	7,5E+02	1,0E+00	8,3E+02	1,0E+00	7,1E+02	2,0E-01	4,1E+02	1,0E+00	8,8E+02
	1,3E+00	5,5E+02	2,5E+00	9,5E+02	2,0E+00	8,0E+02	2,0E+00	1,0E+03	2,0E+00	1,2E+03	2,0E+00	9,7E+02	4,0E-01	5,3E+02	1,0E+00	9,2E+02
	2,5E+00	7,2E+02	3,8E+00	1,1E+03	3,0E+00	9,0E+02	3,0E+00	1,2E+03	3,0E+00	1,3E+03	3,0E+00	1,1E+03	6,0E-01	6,1E+02	2,0E+00	1,1E+03
	5,0E+00	9,4E+02	5,0E+00	1,2E+03	4,0E+00	9,6E+02	4,0E+00	1,2E+03	4,0E+00	1,4E+03	4,0E+00	1,1E+03	1,2E+00	7,8E+02	3,0E+00	1,2E+03
	7,5E+00	1,0E+03	6,3E+00	1,2E+03	5,0E+00	9,9E+02	5,0E+00	1,3E+03	5,0E+00	1,4E+03	5,0E+00	1,2E+03	1,8E+00	8,8E+02	4,0E+00	1,3E+03
	1,1E+01	1,1E+03	7,5E+00	1,2E+03	6,0E+00	1,0E+03	6,0E+00	1,3E+03	6,0E+00	1,5E+03	6,0E+00	1,2E+03	2,4E+00	9,5E+02	5,0E+00	1,4E+03
	1,4E+01	9,0E+02	8,8E+00	1,1E+03	7,0E+00	1,1E+03	7,0E+00	1,3E+03	7,0E+00	1,5E+03	7,0E+00	1,2E+03	3,0E+00	1,0E+03	6,0E+00	1,4E+03
	1,9E+01	8,0E+02	1,0E+01	1,1E+03	8,0E+00	1,1E+03	8,0E+00	1,4E+03	8,0E+00	1,5E+03	8,0E+00	1,3E+03	3,6E+00	1,0E+03	7,0E+00	1,4E+03
	2,7E+01	5,0E+02	1,1E+01	1,0E+03	9,0E+00	1,1E+03	9,0E+00	1,4E+03	9,0E+00	1,6E+03	9,0E+00	1,3E+03	4,2E+00	1,1E+03	8,0E+00	1,4E+03
	2,9E+01	3,6E+02	1,3E+01	9,4E+02	1,0E+01	1,1E+03	1,0E+01	1,4E+03	1,0E+01	1,6E+03	1,0E+01	1,3E+03	4,8E+00	1,1E+03	9,0E+00	1,4E+03
			1,4E+01	9,0E+02	1,1E+01	1,1E+03	1,1E+01	1,4E+03	1,1E+01	1,6E+03	1,1E+01	1,3E+03	5,4E+00	1,2E+03	1,0E+01	1,4E+03
			1,5E+01	8,7E+02	1,2E+01	1,1E+03	1,2E+01	1,4E+03	1,2E+01	1,6E+03	1,2E+01	1,4E+03	6,0E+00	1,2E+03	1,1E+01	1,5E+03
			1,6E+01	8,6E+02	1,3E+01	1,1E+03	1,3E+01	1,4E+03	1,3E+01	1,6E+03	1,3E+01	1,4E+03	6,6E+00	1,2E+03	1,2E+01	1,5E+03
			1,8E+01	8,4E+02	1,4E+01	1,1E+03	1,4E+01	1,4E+03	1,4E+01	1,5E+03	1,4E+01	1,4E+03	7,2E+00	1,2E+03	1,3E+01	1,5E+03
			1,8E+01	8,3E+02	1,5E+01	1,0E+03	1,5E+01	1,3E+03	1,5E+01	1,5E+03	1,5E+01	1,4E+03	7,8E+00	1,3E+03	1,4E+01	1,5E+03
					1,6E+01	1,0E+03	1,6E+01	1,3E+03	1,6E+01	1,5E+03			8,4E+00	1,2E+03	1,5E+01	1,5E+03
					1,7E+01	1,0E+03	1,7E+01	1,3E+03	1,7E+01	1,5E+03			9,0E+00	1,2E+03	1,6E+01	1,4E+03
					1,8E+01	9,9E+02	1,8E+01	1,3E+03	1,8E+01	1,5E+03			9,5E+00	1,2E+03	1,7E+01	1,4E+03
					1,9E+01	9,7E+02	1,9E+01	1,3E+03	1,9E+01	1,4E+03					1,8E+01	1,4E+03
					2,0E+01	9,5E+02	2,0E+01	1,2E+03	2,0E+01	1,4E+03					1,9E+01	1,4E+03
					2,1E+01	9,3E+02	2,1E+01	1,2E+03	2,1E+01	1,4E+03					2,0E+01	1,3E+03
					2,2E+01	9,1E+02	2,2E+01	1,2E+03	2,2E+01	1,4E+03					2,1E+01	1,3E+03
					2,3E+01	8,9E+02	2,3E+01	1,2E+03	2,3E+01	1,4E+03					2,2E+01	1,2E+03
					2,4E+01	8,7E+02	2,4E+01	1,2E+03	2,4E+01	1,3E+03					2,3E+01	1,2E+03
					2,5E+01	8,5E+02	2,5E+01	1,1E+03	2,5E+01	1,3E+03					2,4E+01	1,1E+03
					2,6E+01	8,3E+02	2,6E+01	1,1E+03	2,6E+01	1,3E+03					2,5E+01	1,0E+03
					2,7E+01	8,1E+02	2,7E+01	1,1E+03	2,7E+01	1,3E+03						
					2,8E+01	7,9E+02	2,8E+01	1,1E+03	2,8E+01	1,3E+03						
					2,9E+01	7,7E+02	2,9E+01	1,1E+03	2,9E+01	1,2E+03						
					3,0E+01	7,5E+02	3,0E+01	1,0E+03	3,0E+01	1,2E+03						
					3,1E+01	7,3E+02	3,1E+01	1,0E+03	3,1E+01	1,2E+03						
					3,2E+01	7,1E+02	3,2E+01	1,0E+03	3,2E+01	1,2E+03						
					3,3E+01	6,9E+02	3,3E+01	9,9E+02	3,3E+01	1,2E+03						
					3,4E+01	6,7E+02	3,4E+01	9,7E+02	3,4E+01	1,1E+03						
					3,5E+01	6,5E+02	3,5E+01	9,5E+02	3,5E+01	1,1E+03						
					3,6E+01	6,3E+02	3,6E+01	9,3E+02	3,6E+01	1,1E+03						
					3,7E+01	0,0E+00	3,7E+01	0,0E+00	3,7E+01	0,0E+00						

Slip modulus k_s calculation according to NEN-EN 26891								
F_{est} [N]	1,1E+03	1,2E+03	1,1E+03	1,4E+03	1,6E+03	1,4E+03	1,3E+03	1,5E+03
F_{04} [N]	4,3E+02	4,8E+02	4,6E+02	5,8E+02	6,5E+02	5,6E+02	5,0E+02	5,9E+02
F_{01} [N]	1,1E+02	1,2E+02	1,1E+02	1,4E+02	1,6E+02	1,4E+02	1,3E+02	1,5E+02
v_{04} [mm]	6,2E-01	8,6E-01	8,0E-01	7,7E-01	7,8E-01	7,9E-01	3,5E-01	6,6E-01
v_{01} [mm]	1,6E-01	2,2E-01	2,0E-01	1,9E-01	1,9E-01	2,0E-01	6,2E-02	1,7E-01
$v_{i,mod}$ [mm]	6,2E-01	8,6E-01	8,0E-01	7,7E-01	7,8E-01	7,9E-01	3,9E-01	6,6E-01
k_s (Based on F_{04} , F_{01}) [N/mm]	6,9E+02	5,5E+02	5,7E+02	7,5E+02	8,3E+02	7,1E+02	1,3E+03	8,8E+02

Slip modulus K_{ser} calculation according to *NEN-EN 1995-1-1*								
$\rho_{m,1}$ [kg/m^3] strength class C18	380,00	380,00	380,00	380,00	380,00	380,00	380,00	380,00
$\rho_{m,1}$ [kg/m^3] strength class C24	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00
$\rho_{m,2}$ [kg/m^3]	550,00	550,00	550,00	550,00	550,00	550,00	550,00	550,00
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C18)	457,17	457,17	457,17	457,17	457,17	457,17	457,17	457,17
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)	480,62	480,62	480,62	480,62	480,62	480,62	480,62	480,62
d [mm]	2,50	2,87	3,00	3,30	3,80	2,87	3,10	3,30
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C18)	678,17	757,35	784,67	846,84	948,02	757,35	805,52	846,84
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)	731,04	816,38	845,83	912,85	1021,91	816,38	868,31	912,85

Slip modulus K_u calculation according to *NEN-EN 1995-1-1*								
$K_u = 2/3 \cdot K_{ser}$	487,36	544,25	563,89	608,57	681,28	544,25	578,88	608,57

Slip modulus $k_{Ff,Rd}$ calculation according to NEN-EN 1995-1-1:2005								
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9 & $c_i \cdot$ 1,2 = 1,12)	408,80	478,24	505,12	579,04	700,00	517,44	562,24	604,80
$v_{Ff,Rd}$ [mm]	5,9E-01	8,6E-01	8,8E-01	7,7E-01	8,4E-01	7,3E-01	4,4E-01	6,8E-01
$v_{i,mod}$ [mm]	5,9E-01	8,6E-01	8,8E-01	7,7E-01	8,4E-01	7,3E-01	4,9E-01	6,8E-01
k_u(Based on $F_{f,Rd}$, F_{01})	6,9E+02	5,5E+02	5,7E+02	7,5E+02	8,3E+02	7,1E+02	1,1E+03	8,8E+02

Comparison:								
K_{ser} / k_s	1,06	1,47	1,47	1,21	1,23	1,14	0,67	1,03
K_u / k_s	0,71	0,98	0,98	0,81	0,82	0,76	0,44	0,69
K_{ser} / k_u	1,06	1,47	1,47	1,21	1,23	1,14	0,76	1,03
K_u / k_u	0,71	0,98	0,98	0,81	0,82	0,76	0,51	0,69
F_{04} / $F_{f,Rd}$	1,05	1,00	0,91	1,00	0,93	1,09	0,90	0,97

diameter / length ratio fastener [-]	0,05 (11.1)	0,06 (1.2)	0,06 (6.1)	0,05 (6.2)	0,05 (6.3)	0,05 (5.1)	0,04 (3.5)	0,05 (9.1)

Fastener tests:								
Material:	Plywood	Plywood	Plywood	Plywood	Plywood	Plywood	Plywood	Plywood
Material thickness t: [mm]	9,5	9	9	9	9	12	8	9,5
Fastener properties: N = nail, Scr = Screw, St = Staple	N Ø2,87x50	Scr Ø4,2x35 (high edge distances)	Scr Ø4,2x35 (// to edge)	Scr Ø4,2x35 (┴ to edge)	N Ø2,9x48	N Ø2,9x48	N Ø2,13 x 50,1	N Ø3,1x63,5 (f_u = 770 Mpa)
Density ρ of material:	genomen: $\rho_k = 400$ en: $\rho_m = 460$ Spruce Plywood	genomen: $\rho_k = 400$ en: $\rho_m = 460$	genomen: $\rho_k = 400$ en: $\rho_m = 460$	genomen: $\rho_k = 400$ en: $\rho_m = 460$	genomen: $\rho_k = 460$ en: $\rho_m = 520$ Douglas-fir Plywood	genomen: $\rho_k = 460$ en: $\rho_m = 520$ Douglas-fir Plywood	genomen: $\rho_k = 460$ en: $\rho_m = 520$	$\rho_k = 420$ en: $\rho_m = 450$ (Canadian Softwood Plywood) CSP Grade CSA 0151-M
Remark:	S-P-F framing lumber 38 x 89 mm Based on 6 specimens	3-layer, 3-ply Framing members: 45 x 90 ungraded 1 specimen	3-layer, 3-ply Framing members: 45 x 90 ungraded Mean of 2 specimen	3-layer, 3-ply Framing members: 45 x 90 ungraded Mean of 2 specimen	Mean of 5 tests Douglas Fir framing members MOE = 110000 kgf / cm^2	Mean of 5 tests Douglas Fir framing members MOE = 110000 kgf / cm^3	Mean of 6 tests Assymmetric Framing lumber 45 x 95 mm	Mean of 5 tests load applied // to surface veneer of the plywood Framing members: 38 x 89 mm Spruce Pine Fir MC framing and plywood ± 9%
Reference:	(1.3)	(8.5)	(8.3)	(8.4)	(2.1)	(2.2)	(15.6)	(4)

D = displacement [mm] L = load [N]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	3,0E-01	4,6E+02	5,0E-01	5,5E+02	5,0E-01	5,0E+02	5,0E-01	5,0E+02	1,0E+00	5,8E+02	1,0E+00	6,9E+02	0,0E+00	8,9E+01	1,9E-01	2,9E+02
	1,3E+00	7,9E+02	6,1E-01	6,0E+02	6,1E-01	6,0E+02	6,1E-01	6,0E+02	2,0E+00	9,4E+02	2,0E+00	1,1E+03	4,7E-01	3,6E+02	3,8E-01	3,9E+02
	2,5E+00	1,1E+03	1,0E+00	7,0E+02	1,0E+00	7,1E+02	1,0E+00	7,7E+02	3,0E+00	1,2E+03	3,0E+00	1,3E+03	2,3E+00	5,4E+02	7,5E-01	5,1E+02
	3,8E+00	1,2E+03	2,0E+00	8,5E+02	2,0E+00	9,6E+02	2,0E+00	1,0E+03	4,0E+00	1,3E+03	4,0E+00	1,5E+03	4,2E+00	7,1E+02	1,5E+00	6,6E+02
	5,0E+00	1,3E+03	4,0E+00	1,1E+03	3,0E+00	1,1E+03	3,0E+00	1,2E+03	5,0E+00	1,4E+03	5,0E+00	1,6E+03	5,8E+00	8,0E+02	3,0E+00	8,2E+02
	6,3E+00	1,3E+03	6,0E+00	1,2E+03	4,0E+00	1,1E+03	3,9E+00	1,2E+03	6,0E+00	1,4E+03	6,0E+00	1,6E+03	8,8E+00	8,9E+02	6,0E+00	1,0E+03
	7,5E+00	1,2E+03	8,0E+00	1,3E+03	5,0E+00	1,2E+03	4,2E+00	1,1E+03	7,0E+00	1,5E+03	7,0E+00	1,7E+03	1,4E+01	8,3E+02	9,0E+00	1,1E+03
	8,8E+00	1,1E+03	1,0E+01	1,4E+03	6,0E+00	1,3E+03	6,0E+00	1,1E+03	8,0E+00	1,5E+03	8,0E+00	1,7E+03	1,5E+01	7,1E+02	1,1E+01	1,1E+03
	1,0E+01	1,1E+03	1,2E+01	1,4E+03	8,1E+00	1,5E+03	7,7E+00	1,1E+03	9,0E+00	1,5E+03	9,0E+00	1,7E+03			1,2E+01	1,1E+03
	1,1E+01	1,0E+03	1,4E+01	1,3E+03	8,3E+00	1,1E+03	8,7E+00	7,5E+02	1,0E+01	1,5E+03	1,0E+01	1,7E+03			1,5E+01	1,1E+03
	1,3E+01	9,5E+02	1,6E+01	1,2E+03	8,8E+00	9,8E+02	1,2E+01	6,4E+02	1,5E+01	1,5E+03	1,5E+01	1,7E+03				
	1,4E+01	9,1E+02	1,8E+01	1,1E+03	9,0E+00	6,6E+02	1,3E+01	2,0E+02	2,0E+01	1,5E+03	2,0E+01	1,7E+03				
	1,5E+01	8,6E+02	2,0E+01	9,8E+02	1,1E+01	6,3E+02	1,8E+01	0,0E+00	2,5E+01	1,4E+03	2,5E+01	1,7E+03				
	1,6E+01	8,3E+02	2,2E+01	7,9E+02	1,5E+01	4,2E+02										
	1,7E+01	7,7E+02	2,4E+01	6,0E+02	1,8E+01	0,0E+00										

Slip modulus k_s calculation according to NEN-EN 26891								
F_{est} [N]	1,3E+03	1,4E+03	1,5E+03	1,2E+03	1,5E+03	1,7E+03	8,9E+02	1,1E+03
F_{04} [N]	5,1E+02	5,5E+02	6,0E+02	5,0E+02	6,0E+02	6,7E+02	3,6E+02	4,6E+02
F_{01} [N]	1,3E+02	1,4E+02	1,5E+02	1,2E+02	1,5E+02	1,7E+02	8,9E+01	1,1E+02
v_{04} [mm]	4,4E-01	5,0E-01	6,1E-01	5,0E-01	1,1E+00	9,8E-01	4,7E-01	6,0E-01
v_{01} [mm]	8,3E-02	1,3E-01	1,5E-01	1,2E-01	2,6E-01	2,4E-01	0,0E+00	7,3E-02
$v_{i,mod}$ [mm]	4,8E-01	5,0E-01	6,1E-01	5,0E-01	1,1E+00	9,8E-01	6,3E-01	7,0E-01
k_s (Based on F_{04} , F_{01}) [N/mm]	1,1E+03	1,1E+03	9,8E+02	1,0E+03	5,7E+02	6,9E+02	5,6E+02	6,6E+02

Slip modulus K_{ser} calculation according to *NEN-EN 1995-1-1*								
$\rho_{m,1}$ [kg/m^3] strength class C18	380,00	380,00	380,00	380,00	380,00	380,00	380,00	380,00
$\rho_{m,1}$ [kg/m^3] strength class C24	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00
$\rho_{m,2}$ [kg/m^3]	460,00	460,00	460,00	460,00	520,00	520,00	520,00	450,00
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C18)	418,09	418,09	418,09	418,09	444,52	444,52		413,52
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)	439,55	439,55	439,55	439,55	467,33	467,33	467,33	434,74
d [mm]	2,87	2,77 d_{ef}=0,66*d	2,77 d_{ef}=0,66*d	2,77 d_{ef}=0,66*d	2,90	2,90	2,13	3,10
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C18)	662,35	1029,58	1029,58	1029,58	732,21	732,21	0,00	692,97
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)	713,99	1109,83	1109,83	1109,83	789,29	789,29	616,63	746,99
		K_{ser} [N/mm] = $\rho_m^{1,5} d/23$ (screw)						

Slip modulus K_u calculation according to *NEN-EN 1995-1-1*								
$K_u = 2/3 \cdot K_{ser}$	475,99	739,89	739,89	739,89	526,19	526,19	411,08	497,99

Slip modulus $k_{Ff,Rd}$ calculation according to NEN-EN 1995-1-1:2005								
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9 & $c_i \cdot$ 1,2 = 1,12)	448,00	435,68 based on d_{ef}	435,68 based on d_{ef}	435,68 based on d_{ef}	467,04	506,24	302,40	567,84
$v_{Ff,Rd}$ [mm]	2,9E-01	4,0E-01	4,4E-01	4,4E-01	8,0E-01	7,3E-01	3,8E-01	9,2E-01
$v_{i,mod}$ [mm]	2,9E-01	4,0E-01	4,4E-01	4,4E-01	8,0E-01	7,3E-01	5,4E-01	1,1E+00
k_u(Based on $F_{f,Rd}$, F_{01})	1,5E+03	1,1E+03	1,0E+03	1,0E+03	5,8E+02	6,9E+02	5,6E+02	5,3E+02

Comparison:								
K_{ser} / k_s	0,67	1,01	1,13	1,11	1,39	1,14	1,09	1,13
K_u / k_s	0,44	0,68	0,76	0,74	0,93	0,76	0,73	0,76
K_{ser} / k_u	0,46	1,01	1,11	1,11	1,36	1,14	1,09	1,40
K_u / k_u	0,31	0,67	0,74	0,74	0,90	0,76	0,73	0,93
$F_{04} / F_{f,Rd}$	1,14	1,27	1,38	1,14	1,29	1,33	1,18	0,81

diameter / length ratio fastener [-]	0,06 (1.3)	0,08 (8.5)	0,08 (8.3)	0,08 (8.4)	0,06 (2.1)	0,06 (2.2)	0,04 (15.6)	0,05 (4)

Fastener tests:				
Material:	Particle Board		Particle Board	
Material thickness t: [mm]	10		12	
Fastener properties: N = nail, Scr = Screw, St = Staple	N Ø2,5x50 (smooth nail)		N Ø2,00 x 49,4 (smoot nail with rectangular cross section)	
Density ρ of material:	$\rho_k = \rho_m$ = 650 kg/m^3		$\rho_k = \rho_m$ = 674 kg/m^3	
Remark:	SPF (Spruce-Pine-Fir) framing lumber 45 x 150 mm Mean of 3 specimen		Assymetric Framing lumber 45x95 mm Mean of 6 specimen	
Reference:	(3.3)		(15.1)	
D = displacement [mm]	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	2,0E-01	2,9E+02	1,0E-02	5,0E+01
	4,0E-01	3,8E+02	2,8E-02	1,0E+02
	6,0E-01	4,5E+02	1,1E-01	2,0E+02
	1,2E+00	5,8E+02	3,9E-01	3,0E+02
	1,8E+00	6,3E+02	5,3E-01	3,4E+02
	2,4E+00	6,7E+02	8,8E-01	4,0E+02
	3,0E+00	7,0E+02	1,3E+00	4,4E+02
	3,6E+00	7,3E+02	2,5E+00	4,9E+02
	4,2E+00	7,4E+02	3,4E+00	5,0E+02
	4,8E+00	7,6E+02	4,4E+00	4,8E+02
	5,4E+00	7,8E+02	5,0E+00	4,6E+02
	6,0E+00	7,9E+02	7,5E+00	3,8E+02
	6,6E+00	7,9E+02	1,0E+01	3,5E+02
	7,2E+00	7,9E+02		
	7,8E+00	7,8E+02		
	8,4E+00	7,5E+02		
	9,0E+00	7,0E+02		
	9,4E+00	6,8E+02		

Slip modulus k_s calculation according to NEN-EN 26891		
F_{est} [N]	7,9E+02	5,0E+02
F_{04} [N]	3,2E+02	2,0E+02
F_{01} [N]	7,9E+01	5,0E+01
v_{04} [mm]	2,6E-01	1,1E-01
v_{01} [mm]	5,5E-02	1,0E-02
$v_{i,mod}$ [mm]	2,7E-01	1,3E-01
k_s (Based on F_{04} , F_{01}) [N/mm]	1,2E+03	1,6E+03

Slip modulus K_{ser} calculation according to *NEN-EN 1995-1-1*		
$\rho_{m,1}$ [kg/m^3] strength class C18	380,00	380,00
$\rho_{m,1}$ [kg/m^3] strength class C24	420,00	420,00
$\rho_{m,2}$ [kg/m^3]	650,00	674,00
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C18)	496,99	506,08
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)	522,49	532,05
d [mm]	2,50	2,00
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C18)	768,69	660,75
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)	828,61	712,25

Slip modulus K_u calculation according to *NEN-EN 1995-1-1*		
$K_u = 2/3 \cdot K_{ser}$	552,41	474,84

Slip modulus $k_{Ff,Rd}$ calculation according to NEN-EN 1995-1-1:2005		
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9 & $c_i \cdot$ 1,2 = 1,12)	405,44	359,52
$v_{Ff,Rd}$ [mm]	4,6E-01	6,0E-01
$v_{i,mod}$ [mm]	5,0E-01	6,8E-01
k_u(Based on $F_{f,Rd}$, F_{01})	8,1E+02	5,3E+02

Comparison:		
K_{ser} / k_s	0,72	0,45
K_u / k_s	0,48	0,30
K_{ser} / k_u	1,02	1,35
K_u / k_u	0,68	0,90
F_{04} / $F_{f,Rd}$	0,78	0,56

diameter / length ratio fastener [-]	0,05 (3.3)	0,04 (15)

Fastener tests:								
Material:	GKB	GKB	GKB	GKB	GKB	GKB	GKB	GKB
Material thickness t: [mm]	12	12,5	12,5	13	12,5	12,5	12,5	12,5
Fastener properties: N = nail, Scr = Screw, St = Staple	N Ø2,34x40 (GN40)	N Ø2,2x45	Scr Ø3,35x45	Scr Ø3,9x41 Essve drywall screw	N Ø2,45 x 34,5	N Ø2,45 x 34,5	Scr Ø3,00 x 38	Scr Ø3,00 x 38
Density ρ of material:		Gyproc gipskartonplaat	Gyproc gipskartonplaat	Danogips Normal DN13 / GKB/1200	Gyproc Normal GN	Gyproc Normal GN	Gyproc Normal GN	Gyproc Normal GN
Remark:	S-P-F framing lumber 38 x 89 mm Based on 6 specimens	Assymetric (langs (ABA)) Mean of 10 tests framing lumber 45 x 45 mm	Assymetric (langs (ABA)) Mean of 10 tests framing lumber 45 x 45 mm	Mean of 2 tests (// to edge) (edges lined with paper) Framing members: 45 x 90 ungraded	Assymetric Framing lumber 45x95 mm Mean of 6 specimen (edges lined with paper)	Assymetric Framing lumber 45x95 mm Mean of 6 specimen (sawn edge)	Assymetric Framing lumber 45x95 mm Mean of 6 specimen (edges lined with paper)	Assymetric Framing lumber 45x95 mm Mean of 6 specimen (sawn edge)
Reference:	(1.1)	(7.2)	(7.4)	(8.1)	(15.3)	(15.2)	(15.4)	(15.5)

Reference:	(1.1)		(7.2)		(7.4)		(8.1)		(15.3)		(15.2)		(15.4)		(15.5)	
D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	3,3E-01	1,6E+02	1,0E-02	4,5E+01	1,0E-02	6,6E+01	1,0E+00	3,6E+02	0,0E+00	5,6E+01	0,0E+00	4,3E+01	2,6E-02	9,0E+01	1,8E-02	6,5E+01
	1,3E+00	2,6E+02	2,0E-02	7,4E+01	2,0E-02	1,1E+02	2,0E+00	4,8E+02	5,3E-02	1,1E+02	2,3E-01	1,7E+02	1,6E-01	3,6E+02	3,1E-01	2,6E+02
	2,5E+00	3,5E+02	3,0E-02	9,4E+01	3,0E-02	1,3E+02	3,0E+00	5,5E+02	2,7E-01	2,3E+02	8,6E-01	2,6E+02	5,8E-01	5,4E+02	7,8E-01	3,9E+02
	3,8E+00	4,1E+02	4,0E-02	1,1E+02	4,0E-02	1,6E+02	3,5E+00	5,6E+02	9,4E-01	3,4E+02	2,7E+00	3,4E+02	1,3E+00	7,2E+02	1,8E+00	5,2E+02
	5,0E+00	4,5E+02	1,0E-01	1,5E+02	5,0E-02	1,9E+02	4,0E+00	5,6E+02	3,3E+00	4,5E+02	4,3E+00	3,9E+02	2,3E+00	8,1E+02	2,7E+00	5,8E+02
	6,3E+00	4,6E+02	2,0E-01	1,9E+02	6,0E-02	2,2E+02	4,5E+00	5,7E+02	4,9E+00	5,1E+02	8,2E+00	4,3E+02	4,4E+00	8,5E+02	5,0E+00	6,5E+02
	7,5E+00	4,6E+02	3,0E-01	2,3E+02	8,0E-02	2,5E+02	5,0E+00	5,8E+02	8,5E+00	5,6E+02	1,1E+01	3,9E+02	6,2E+00	9,0E+02	7,7E+00	5,8E+02
	8,8E+00	4,6E+02	3,3E-01	2,4E+02	9,0E-02	2,8E+02	8,0E+00	5,9E+02	1,3E+01	5,1E+02	1,3E+01	3,4E+02	7,1E+00	8,1E+02	8,7E+00	4,6E+02
	1,0E+01	4,5E+02	3,4E-01	2,4E+02	1,0E-01	3,0E+02	9,0E+00	5,5E+02	1,4E+01	4,5E+02			8,8E+00	7,1E+02		
	1,1E+01	4,4E+02	3,5E-01	2,5E+02	1,1E-01	3,2E+02	1,2E+01	4,9E+02								
	1,3E+01	4,3E+02	4,0E-01	2,6E+02	1,2E-01	3,4E+02	1,4E+01	3,3E+02								
	1,4E+01	4,1E+02	5,0E-01	2,8E+02	1,3E-01	3,5E+02	1,4E+01	3,0E+02								
	1,5E+01	3,7E+02	5,5E-01	3,0E+02	1,4E-01	3,7E+02	1,8E+01	1,7E+02								
	1,6E+01	3,3E+02	5,8E-01	3,1E+02	1,5E-01	3,9E+02	2,0E+01	0,0E+00								
	1,8E+01	2,4E+02	7,0E-01	3,3E+02	2,0E-01	4,5E+02										
	1,8E+01	2,0E+02	1,0E+00	3,8E+02	2,5E-01	4,9E+02										
			1,3E+00	4,2E+02	3,0E-01	5,1E+02										
			1,6E+00	4,4E+02	4,0E-01	5,6E+02										
			2,0E+00	4,8E+02	5,0E-01	6,0E+02										
			2,1E+00	4,9E+02	6,0E-01	6,3E+02										
			2,1E+00	5,0E+02	8,0E-01	6,5E+02										
			2,2E+00	5,0E+02	1,0E+00	6,8E+02										
			2,9E+00	5,5E+02	1,4E+00	7,2E+02										
			5,0E+00	6,0E+02	2,0E+00	7,6E+02										
					2,5E+00	7,8E+02										
					3,0E+00	8,0E+02										
					3,5E+00	8,2E+02										

Slip modulus k_s calculation according to NEN-EN 26891								
F_{est} [N]	4,6E+02	6,0E+02	8,2E+02	5,9E+02	5,6E+02	4,3E+02	9,0E+02	6,5E+02
F_{04} [N]	1,8E+02	2,4E+02	3,3E+02	2,4E+02	2,3E+02	1,7E+02	3,6E+02	2,6E+02
F_{01} [N]	4,6E+01	6,0E+01	8,2E+01	5,9E+01	5,6E+01	4,3E+01	9,0E+01	6,5E+01
v_{04} [mm]	5,3E-01	3,3E-01	1,1E-01	6,6E-01	2,7E-01	2,3E-01	1,6E-01	3,1E-01
v_{01} [mm]	9,3E-02	1,5E-02	1,4E-02	1,6E-01	0,0E+00	0,0E+00	2,6E-02	1,8E-02
$v_{i,mod}$ [mm]	5,8E-01	4,3E-01	1,3E-01	6,6E-01	3,5E-01	3,1E-01	1,8E-01	3,9E-01
k_s (Based on F_{04} , F_{01}) [N/mm]	3,2E+02	5,6E+02	2,5E+03	3,6E+02	6,4E+02	5,5E+02	2,0E+03	6,7E+02

Slip modulus K_{ser} calculation according to *NEN-EN 1995-1-1*								
$\rho_{m,1}$ [kg/m^3] strength class C18								
$\rho_{m,1}$ [kg/m^3] strength class C24								
$\rho_{m,2}$ [kg/m^3]								
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C18)								
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)								
d [mm]								
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C18)								
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)								

Slip modulus K_u calculation according to *NEN-EN 1995-1-1*								
$K_u = 2/3 \cdot K_{ser}$								

Slip modulus $k_{Ff,Rd}$ calculation according to NEN-EN 1995-1-1:2005								
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9 & $c_i \cdot$ 1,2 = 1,12)	200,48	188,16	187,04 based on d_{ef}	236,32 based on d_{ef}	217,28	217,28	165,76 based on d_{ef}	165,76 based on d_{ef}
$v_{Ff,Rd}$ [mm]	7,0E-01	1,9E-01	4,9E-02	6,5E-01	2,5E-01	5,7E-01	6,3E-02	1,7E-01
$v_{i,mod}$ [mm]	7,8E-01	2,5E-01	6,2E-02	6,5E-01	3,4E-01	7,1E-01	8,1E-02	2,5E-01
k_u(Based on $F_{f,Rd}$, F_{01})	2,6E+02	7,4E+02	3,0E+03	3,6E+02	6,4E+02	3,0E+02	2,0E+03	6,7E+02

(k_{mod} = 0,85)

Comparison:								
K_{ser} / k_s								
K_u / k_s								
K_{ser} / k_u								
K_u / k_u								
F_{04} / $F_{f,Rd}$	0,92	1,28	1,76	1,01	1,04	0,79	2,18	1,56

diameter / length ratio fastener [-]	0,06 (1.1)	0,05 (7.2)	0,05 (7.4)	0,06 (8.1)	0,07 (15.3)	0,07 (15.3)	0,05 (15.4)	0,05 (15.5)

Fastener tests:												
Material:	Gypsum Fiber Board		Gypsum Fiber Board		Gypsum Fiber Board		Gypsum Fiber Board		Gypsum Fiber Board		Gypsum Fiber Board	
Material thickness t: [mm]	15		15		15		15		15		12,5	
Fastener properties: N = nail, Scr = Screw, St = Staple	Staple Ø1,34x1,62x47 (NIKO staple)		Staple Ø1,5x50		N Ø2,5x65 (Rillenagel)		Staple Ø1,8x65		N Ø2,5x55 (Smooth nail)		Staple Ø1,4 x 1,6 x 60	
Density ρ of material:												
Remark:	SPF (Spruce-Pine-Fir) framing lumber 40 x 100 mm Mean of 3 specimens		Mean of 5 tests, mean of 4 fastener slices, symmetric test		Mean of 5 tests, mean of 4 fastener slices, symmetric test		Mean of 5 tests, mean of 4 fastener slices, symmetric test		Mean of 5 tests, mean of 4 fastener slices, symmetric test		Mean of 2 tests FERMACELL	
Reference:	(3.1)		(12.1)		(12.2)		(13.1)		(13.2)		(14)	
D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	2,0E-01	3,4E+02	6,3E-02	1,0E+02	1,3E-01	1,8E+02	1,3E-01	3,0E+02	1,3E-01	2,7E+02	2,8E-01	1,4E+02
	4,0E-01	4,4E+02	1,3E-01	1,4E+02	2,5E-01	2,5E+02	2,5E-01	3,8E+02	2,5E-01	3,5E+02	8,9E-01	4,0E+02
	6,0E-01	5,0E+02	2,5E-01	1,8E+02	5,0E-01	3,4E+02	5,0E-01	4,7E+02	5,0E-01	4,3E+02	1,4E+00	5,5E+02
	9,0E-01	6,0E+02	5,0E-01	2,5E+02	1,0E+00	4,8E+02	1,0E+00	6,1E+02	1,0E+00	5,4E+02	2,5E+00	9,5E+02
	1,2E+00	7,0E+02	1,3E+00	4,6E+02	2,5E+00	7,4E+02	3,0E+00	1,1E+03	2,0E+00	6,7E+02	6,9E+00	1,4E+03
	1,8E+00	8,2E+02	1,5E+00	5,1E+02	6,5E+00	1,3E+03	4,0E+00	1,3E+03	2,5E+00	7,2E+02		
	2,4E+00	9,3E+02	1,8E+00	5,5E+02	8,5E+00	1,6E+03			3,0E+00	7,6E+02		
	3,0E+00	9,8E+02							3,5E+00	7,8E+02		
	3,6E+00	1,0E+03							4,0E+00	7,9E+02		
	4,2E+00	1,0E+03							4,5E+00	8,0E+02		
	4,8E+00	1,0E+03										
	5,4E+00	1,0E+03										
	6,0E+00	1,0E+03										
	6,6E+00	9,9E+02										
	7,2E+00	9,6E+02										
	7,8E+00	9,3E+02										
	8,4E+00	9,1E+02										
	8,7E+00	8,9E+02										

Slip modulus k_s calculation according to NEN-EN 26891						
F_{est} [N]	1,0E+03	5,5E+02	1,6E+03	1,3E+03	8,0E+02	1,4E+03
F_{04} [N]	4,1E+02	2,2E+02	6,2E+02	5,1E+02	3,2E+02	5,7E+02
F_{01} [N]	1,0E+02	5,5E+01	1,6E+02	1,3E+02	8,0E+01	1,4E+02
v_{04} [mm]	3,4E-01	3,9E-01	1,8E+00	6,4E-01	2,1E-01	1,4E+00
v_{01} [mm]	6,0E-02	3,3E-02	1,1E-01	5,3E-02	3,7E-02	2,8E-01
$v_{i,mod}$ [mm]	3,8E-01	4,8E-01	2,3E+00	7,8E-01	2,3E-01	1,5E+00
k_s (Based on F_{04} , F_{01}) [N/mm]	1,1E+03	4,6E+02	2,7E+02	6,5E+02	1,4E+03	4,0E+02

Slip modulus K_{ser} calculation according to *NEN-EN 1995-1-1*						
$\rho_{m,1}$ [kg/m³] strength class C18						
$\rho_{m,1}$ [kg/m³] strength class C24						
$\rho_{m,2}$ [kg/m³]						
ρ_m [kg/m³] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C18)						
ρ_m [kg/m³] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)						
d [mm]						
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C18)						
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)						

Slip modulus K_u calculation according to *NEN-EN 1995-1-1*						
$K_u = 2/3 \cdot K_{ser}$						

Slip modulus $k_{Ff,Rd}$ calculation according to NEN-EN 1995-1-1:2005						
$F_{f,Rd}$ [N] (framing lumber: C24) ($\gamma_M = 1,3$ & $k_{mod} = 0,9$ & $c_i \cdot 1,2 = 1,12$)	443,52 (beide benen	456,96 (beide benen	432,32	611,52 (beide benen	432,32	445,76 (beide benen
$v_{Ff,Rd}$ [mm]	4,2E-01	1,2E+00	8,4E-01	1,0E+00	5,0E-01	8,9E-01
$v_{i,mod}$ [mm]	4,6E-01	1,4E+00	1,1E+00	1,2E+00	5,7E-01	9,0E-01
k_u(Based on $F_{f,Rd}$, F_{01})	9,6E+02	3,4E+02	3,8E+02	5,1E+02	7,6E+02	5,0E+02

($k_{mod} = 0,85$)

Comparison:						
K_{ser} / k_s						
K_u / k_s						
K_{ser} / k_u						
K_u / k_u						
F_{04} / $F_{f,Rd}$	0,92	0,48	1,44	0,83	0,74	1,29

diameter / length ratio fastener [-]	0,06 (3.1)	0,06 (12.1)	0,04 (12.2)	0,06 (13.1)	0,05 (13.2)	0,05 (14)

Reference list belonging to the calculation sheet and column charts

(1) (Yasumura & Kawai, 1997)
(2) (Komatsu et al., 1999)
(3) (Dujic & Zarnic, 2003)
(4) (Karacabeyli & Ceccotti, 1996)
(5) (Christovasilis et al., 2007)
(6) (Richard et al., 2002)
(7) (Bouw- en woningtoezicht Rotterdam, 1989)
(8) (Andreasson, 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)

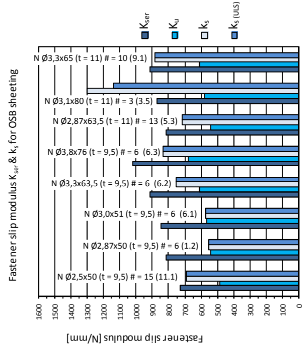
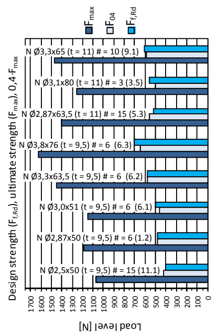
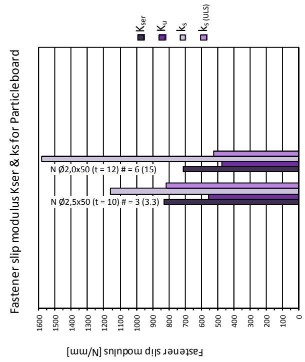
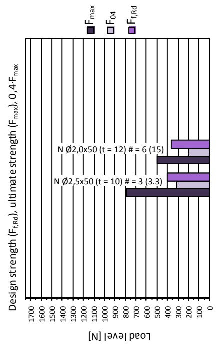
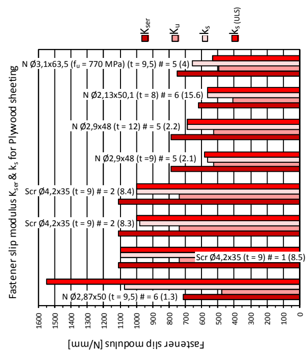
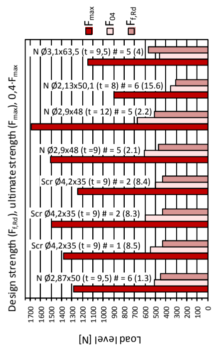
The literature can be found in the sources displayed on page 105.

Appendix II Comparison of K_{ser} and k_s

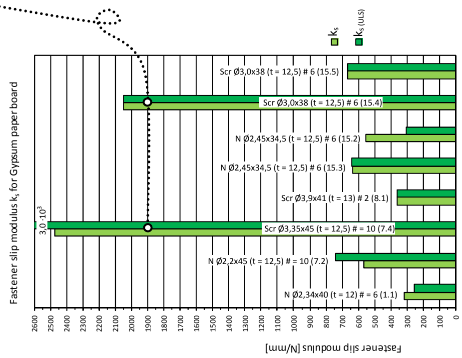
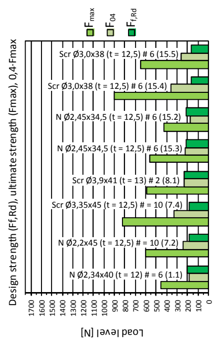
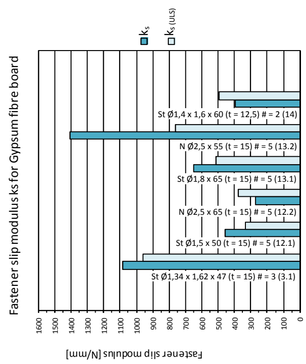
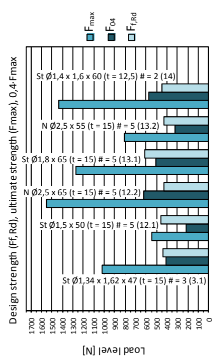
In §2.3 and §2.4 the results were presented of a comparison between the experimentally determined fastener slip-modulus k_s and $k_{s(ULS)}$, and the calculated fastener slip-modulus K_{ser} and K_u. The comparison was based on the column charts on this page.

The upper charts show K_{ser}, K_u, k_s, and k_u for each test of successively OSB, particleboard, plywood, gypsum paperboard, and gypsum fibreboard. The bottom charts show the ultimate load reached in the test (F_{max}), 40% of F_{max} (F_{04}), and the calculated design strength of the sheathing-to-timber connection ($F_{f,Rd}$). The experimentally determined values were based on test data found in literature, and calculated according to the standard NEN-EN 26891. The calculated values were based on the equations for K_{ser}, and the design rules for the fastener strength in timber-frame shear-walls $F_{f,Rd}$, from Eurocode 5: NEN-EN 1995-1-1.

Next to the bars, reference is made to the configuration of the fastener, and the sources. For example:
nail Ø2,87x63,5 (t = 11) # = 13 (5.3) means: a nail fastener is used, the thickness of the sheathing is 11 mm, the test is an average of 13 specimen (#), digit 5 in (5.3) is a reference in the reference list on page 126, digit 3 means, this tests concerns the third test in the publication referred to by digit 5.

(!) For comment see page 30.

The bar charts are based on the calculations that are shown on the pages before (Appendix I).

Appendix III Determining steel-to-timber fastener strength

In this appendix the calculations are shown which are basis for the characteristic fastener strength $F_{v,Rk}$ as stated in paragraph 3.1. The values are given in the table below, and were calculated using NEN-EN 1995-1-1, with a steel thickness of 3,0 mm.

Fastener:	nail Ø4,11 x 89 (smooth)	nail Ø4 x 40 (annular)	nail Ø4 x 75 (annular)	screw Ø5 x 50
Characteristic strength $F_{v,Rk}$ [kN]:	1,48	1,32	1,47	1,34

table 24: Characteristic values for the steel-to-timber fastener strength

See the next pages for the calculation of the characteristic fastener strength $F_{v,Rk}$ for:

- nail Ø4,11 x 89 (smooth)
- nail Ø4 x 40 (annular)
- nail Ø4 x 75 (annular)
- screw Ø5 x 50

nail Ø4,11 x 89 (smooth)

VERBINDING STAAL-OP-HOUT VOLGENS NEN-EN 1995-1-1

Invoer:

Verbindingsmiddel:

diameter d:	4,11	[mm]		Invullen
effectieve diameter d_{eff}:	4,11	[mm]	(Voor nagel: d_{eff} = d, voor schroef d_{eff} = 0,66 · d invullen)	Overnemen
diameter nagelkop d_h:		[mm]		Uitkomst
lengte:	89	[mm]		
karakteristieke waarde van het vloeimoment $M_{y,Rk}$:	7100,05	[Nmm]	(Wanneer niet gespecificeerd zie berekening hieronder)	
treksterkte f_u:	600	[N/mm^2]	(Minimaal 600 N/mm^2)	
stuiksterkte $f_{h,k}$:	18,78	[N/mm^2]	(Zie berekening hieronder)	

Hout:

karakteristieke dichtheid ρ_k:	350	[kg/m^3]	(Standaard 320 kg/m^3 voor C18)
dikte t_1:	128	[mm]	(Dikte van de houten stijl / regel)

Staalplaat:

dikte t_2: 3 [mm]

Aanvullend:

Beperking van de bijdrage van het koordeffect: 15 %

Ronde nagels:	15 %
Vierkante en geprofileerde nagels:	25 %
Andere nagels:	50 %
Schroeven:	100 %
Bouten:	25 %
Stiften:	0 %

Uittrekkracht $F_{ax,Rk}$: 824,59 [N] (Zie berekening hieronder)

Controleren:

- Doortrekken van de kop	(Zie berekening hieronder)
- Blokafschuiving	(Niet opgenomen in dit Excelblad)
- Doorsnede staal / hout	(Niet opgenomen in dit Excelblad)
- Treksterkte van de schroef en de kop	(Niet opgenomen in dit Excelblad)

Berekende waarden:

- Karakteristieke waarde van het vloeimoment:

$$M_{y,Rk} = \begin{cases} 0{,}3 f_u d^{2,6} & \text{voor ronde nagels} \\ 0{,}45 f_u d^{2,6} & \text{voor vierkante en geprofileerde nagels} \end{cases}$$

= 7100,05 [Nmm]
= 10650,07 [Nmm]

- Karakteristieke stuiksterkte in hout of LVL:

  - zonder voorgeboorde gaten

$$f_{h,k} = 0{,}082 \rho_k d^{-0,3}$$ = 18,78 [N/mm^2]

  - met voorgeboorde gaten

$$f_{h,k} = 0{,}082(1 - 0{,}01d)\rho_k$$ = 27,52 [N/mm^2]

- Karakteristieke waarde van de uittreksterkte voor nagels:

$$f_{ax,k} = 20 \cdot 10^{-6} \rho_k^2$$ = 2,45 [N/mm^2] (Hechtlengte tenminste 12 d)
(Nagels in kops hout worden niet in staat geacht axiale krachten over te dragen)

$$f_{head,k} = 70 \cdot 10^{-6} \rho_k^2$$ = 8,58 [N/mm^2]

$$F_{ax,Rk} \le f_{head,k} d_h^2$$ ≤ 0,00 [N] (Doortrekken van de kop)

- Karakteristieke uittrek kracht voor nagels:

Voor gladde nagels:

t_{pen} = 81,89 [mm] (t_{pen} Is de hechtlengte in het element die de puntzijde bevat)

8 · d = 32,88 [mm] (Vereist is dat: $t_{pen} \ge 8d$)

$$F_{ax,Rk} = min\begin{cases} f_{ax,k} d t_{pen} \\ f_{ax,k} dt + f_{head,k} d_h^2 \end{cases}$$ = 824,59 [N] (wanneer 8d ≤ t_{pen} ≤ 12d; $F_{ax,Rk}$ vermenigvuldigen met (t_{pen}/4d)-2)

Uitkomsten:

- Voor een dunne staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min\begin{cases} 0{,}4 f_{h,k} t_1 d \\ 1{,}15\sqrt{2M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4} \end{cases}$$

(Johansen deel + Koordeffect)

= min { 2528,298 (a); 1234,883 (b) } = 1234,88 [N]

- Voor een dikke staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min\begin{cases} f_{h,k} t_1 d \\ f_{h,k} t_1 d\left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right] + \frac{F_{ax,Rk}}{4} \\ 2{,}3\sqrt{M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4} \end{cases}$$

(Johansen deel + Koordeffect)

= min { 6320,745 (c); 6522,756 (d); 1733,579 (e) } = 1733,58 [N]

0,5625·d	1297,22
0,625·d	1359,557
0,6875·d	1421,894
0,75·d	1484,231
0,8125·d	1546,568
0,875·d	1608,905
0,9375·d	1671,242

Te hanteren uitkomsten:

- Dikke plaat:	$t_2 \ge d$
- Dunne plaat:	$t_2 \le 0{,}5 \cdot d$
- Interpoleren:	$0{,}5 \cdot d \le t_2 \le d$
t_2 =	3,00
d =	4,11
0,5 · d =	2,06

nail Ø4 x 40 (annular)

VERBINDING STAAL-OP-HOUT VOLGENS NEN-EN 1995-1-1

Invoer:

Verbindingsmiddel:

diameter d:	4	[mm]		
effectieve diameter d_{eff}:	4	[mm]	(Voor nagel: d_{eff} = d, voor schroef d_{eff} = 0,66 · d invullen)	
diameter nagelkop d_h:		[mm]		
lengte:	40	[mm]		
karakteristieke waarde van het vloeimoment $M_{y,Rk}$:	6616,50	[Nmm]	(Wanneer niet gespecificeerd zie berekening hieronder)	
treksterkte f_u:	600	[N/mm^2]	(Minimaal 600 N/mm^2)	
stuiksterkte $f_{h,k}$:	18,93	[N/mm^2]	(Zie berekening hieronder)	

Legenda	
	Invullen
	Overnemen
	Uitkomst

Hout:

karakteristieke dichtheid ρ_k:	350	[kg/m^3]	(Standaard 320 kg/m^3 voor C18)
dikte t_1:	128	[mm]	(Dikte van de houten stijl / regel)

Staalplaat:

dikte t_2: 3 [mm]

Aanvullend:

Beperking van de bijdrage van het koordeffect: 50 %

Ronde nagels:	15 %
Vierkante en geprofileerde nagels:	25 %
Andere nagels:	50 %
Schroeven:	100 %
Bouten:	25 %
Stiften:	0 %

Uittrekkracht $F_{ax,Rk}$: 20,21 [N] (Zie berekening hieronder)

Controleren:

- Doortrekken van de kop	(Zie berekening hieronder)
- Blokafschuiving	(Niet opgenomen in dit Excelblad)
- Doorsnede staal / hout	(Niet opgenomen in dit Excelblad)
- Treksterkte van de schroef en de kop	(Niet opgenomen in dit Excelblad)

Berekende waarden:

- Karakteristieke waarde van het vloeimoment:

$$M_{y,Rk} = \begin{cases} 0{,}3 f_u d^{2,6} & \text{voor ronde nagels} \\ 0{,}45 f_u d^{2,6} & \text{voor vierkante en geprofileerde nagels} \end{cases}$$

= 6616,50 [Nmm] (voor ronde nagels)
= 9924,75 [Nmm] (voor vierkante en geprofileerde nagels)

- Karakteristieke stuiksterkte in hout of LVL:

- zonder voorgeboorde gaten

$$f_{h,k} = 0{,}082 \rho_k d^{-0,3}$$ = 18,93 [N/mm^2]

- met voorgeboorde gaten

$$f_{h,k} = 0{,}082(1 - 0{,}01d)\rho_k$$ = 27,55 [N/mm^2]

- Karakteristieke waarde van de uittreksterkte voor nagels:

$$f_{ax,k} = 20 \cdot 10^{-6} \rho_k^2$$ = 2,45 [N/mm^2] (Hechtlengte tenminste 12 d)

(Nagels in kops hout worden niet in staat geacht axiale krachten over te dragen)

$$f_{head,k} = 70 \cdot 10^{-6} \rho_k^2$$ = 8,58 [N/mm^2]

$$F_{ax,Rk} \le f_{head,k} d_h^2$$ ≤ 0,00 [N] (Doortrekken van de kop)

- Karakteristieke uittrek kracht voor nagels:

Voor gladde nagels:

t_{pen}	=	33,00 [mm]	(t_{pen} Is de hechtlengte in het element die de puntzijde bevat)
8 · d	=	32,00 [mm]	(Vereist is dat: $t_{pen} \ge 8d$)

$$F_{ax,Rk} = min\begin{cases} f_{ax,k} d t_{pen} \\ f_{ax,k} d t + f_{head,k} d_h^2 \end{cases}$$ = 20,21 [N] (wanneer $8d \le t_{pen} \le 12d$; $F_{ax,Rk}$ vermenigvuldigen met $(t_{pen}/4d)$-2)

Uitkomsten:

- Voor een dunne staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min\begin{cases} 0{,}4 f_{h,k} t_1 d \\ \underbrace{1{,}15\sqrt{2 M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 999,504 (a); 1153,677 (b) } = 999,50 [N]

- Voor een dikke staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min\begin{cases} f_{h,k} t_1 d \\ \underbrace{f_{h,k} t_1 d \left[\sqrt{2 + \frac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right]}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \\ \underbrace{2{,}3\sqrt{M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 2498,76 (c); 2874,426 (d); 1630,5 (e) } = 1630,50 [N]

0,5625·d	1078,378
0,625·d	1157,253
0,6875·d	1236,127
0,75·d	1315,002
0,8125·d	1393,876
0,875·d	1472,751
0,9375·d	1551,625

Te hanteren uitkomsten:

- Dikke plaat:	$t_2 \ge d$
- Dunne plaat:	$t_2 \le 0{,}5 \cdot d$
- Interpoleren:	$0{,}5 \cdot d \le t_2 \le d$
t_2 =	3,00
d =	4,00
0,5 · d =	2,00

nail Ø4 x 75 (annular)

VERBINDING STAAL-OP-HOUT VOLGENS NEN-EN 1995-1-1

Invoer:

Verbindingsmiddel:				
diameter d:	4	[mm]		Invullen
effectieve diameter d_{eff}:	4	[mm]	(Voor nagel: $d_{eff} = d$, voor schroef $d_{eff} = 0{,}66 \cdot d$ invullen)	Overnemen
diameter nagelkop d_h:		[mm]		Uitkomst
lengte:	75	[mm]		
karakteristieke waarde van het vloeimoment $M_{y,Rk}$:	6616,50	[Nmm]	(Wanneer niet gespecificeerd zie berekening hieronder)	
treksterkte f_u:	600	[N/mm^2]	(Minimaal 600 N/mm^2)	
stuiksterkte $f_{h,k}$:	18,93	[N/mm^2]	(Zie berekening hieronder)	
Hout:				
karakteristieke dichtheid ρ_k:	350	[kg/m^3]	(Standaard 320 kg/m^3 voor C18)	
dikte t_1:	128	[mm]	(Dikte van de houten stijl / regel)	
Staalplaat:				
dikte t_2:	3	[mm]		
Aanvullend:				
Beperking van de bijdrage van het koordeffect:	50	%		

Ronde nagels:	15 %
Vierkante en geprofileerde nagels:	25 %
Andere nagels:	50 %
Schroeven:	100 %
Bouten:	25 %
Stiften:	0 %

Uittrekkracht $F_{ax,Rk}$: 666,40 [N] (Zie berekening hieronder)

Controleren:

- Doortrekken van de kop	(Zie berekening hieronder)
- Blokafschuiving	(Niet opgenomen in dit Excelblad)
- Doorsnede staal / hout	(Niet opgenomen in dit Excelblad)
- Treksterkte van de schroef en de kop	(Niet opgenomen in dit Excelblad)

Berekende waarden:

- Karakteristieke waarde van het vloeimoment:

$$M_{y,Rk} = \begin{cases} 0{,}3 f_u d^{2,6} & \text{voor ronde nagels} \\ 0{,}45 f_u d^{2,6} & \text{voor vierkante en geprofileerde nagels} \end{cases}$$

= 6616,50 [Nmm]
= 9924,75 [Nmm]

- Karakteristieke stuiksterkte in hout of LVL:

- zonder voorgeboorde gaten

$$f_{h,k} = 0{,}082 \rho_k d^{-0{,}3}$$ = 18,93 [N/mm^2]

- met voorgeboorde gaten

$$f_{h,k} = 0{,}082 (1 - 0{,}01 d) \rho_k$$ = 27,55 [N/mm^2]

- Karakteristieke waarde van de uittreksterkte voor nagels:

$$f_{ax,k} = 20 \cdot 10^{-6} \rho_k^2$$ = 2,45 [N/mm^2] (Hechtlengte tenminste 12 d)
(Nagels in kops hout worden niet in staat geacht axiale krachten over te dragen)

$$f_{head,k} = 70 \cdot 10^{-6} \rho_k^2$$ = 8,58 [N/mm^2]

$$F_{ax,Rk} \le f_{head,k} d_h^2$$ ≤ 0,00 [N] (Doortrekken van de kop)

- Karakteristieke uittrek kracht voor nagels:

Voor gladde nagels:

t_{pen} = 68,00 [mm] (t_{pen} Is de hechtlengte in het element die de puntzijde bevat)

$8 \cdot d$ = 32,00 [mm] (Vereist is dat: $t_{pen} \ge 8d$)

$$F_{ax,Rk} = \min \begin{cases} f_{ax,k} d t_{pen} \\ f_{ax,k} d t + f_{head,k} d_h^2 \end{cases}$$ = 666,40 [N] (wanneer $8d \le t_{pen} \le 12d$; $F_{ax,Rk}$ vermenigvuldigen met $(t_{pen}/4d)$-2)

Uitkomsten:

- Voor een dunne staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min \begin{cases} 0{,}4 f_{h,k} t_1 d \\ \underbrace{1{,}15 \sqrt{2 M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 2059,584 (a); 1234,451 (b) } = 1234,45 [N]

0,5625·d	1294,054
0,625·d	1353,657
0,6875·d	1413,259
0,75·d	1472,862
0,8125·d	1532,465
0,875·d	1592,068
0,9375·d	1651,671

Te hanteren uitkomsten:

- Dikke plaat:	$t_2 \ge d$
- Dunne plaat:	$t_2 \le 0{,}5 \cdot d$
- Interpoleren:	$0{,}5 \cdot d \le t_2 \le d$
t_2 =	3,00
d =	4,00
0,5 · d =	2,00

- Voor een dikke staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d \\ \underbrace{f_{h,k} t_1 d \left[\sqrt{2 + \frac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right]}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \\ \underbrace{2{,}3 \sqrt{M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 5148,96 (c); 5423,318 (d); 1711,273 (e) } = 1711,27 [N]

screw Ø5 x 50

VERBINDING STAAL-OP-HOUT VOLGENS NEN-EN 1995-1-1

Invoer:

Verbindingsmiddel:				
diameter d:	5	[mm]		Invullen
effectieve diameter d_{eff}:	3,3	[mm]	(Voor nagel: d_{eff} = d, voor schroef d_{eff} = 0,66 · d invullen)	Overnemen
diameter nagelkop d_h:		[mm]		Uitkomst
lengte:	50	[mm]		
karakteristieke waarde van het vloeimoment $M_{y,Rk}$:	4012,44	[Nmm]	(Wanneer niet gespecificeerd zie berekening hieronder)	
treksterkte f_u:	600	[N/mm²]	(Minimaal 600 N/mm²)	
stuiksterkte $f_{h,k}$:	20,06	[N/mm²]	(Zie berekening hieronder)	
Hout:				
karakteristieke dichtheid ρ_k:	350	[kg/m³]	(Standaard 320 kg/m³ voor C18)	
dikte t_1:	128	[mm]	(Dikte van de houten stijl / regel)	
Staalplaat:				
dikte t_2:	3	[mm]		

Aanvullend:
Beperking van de bijdrage van het koordeffect: 100 %

Ronde nagels:	15 %
Vierkante en geprofileerde nagels:	25 %
Andere nagels:	50 %
Schroeven:	100 %
Bouten:	25 %
Stiften:	0 %

Uittrekkracht $F_{ax,Rk}$: 4245,22 [N] (Zie berekening hieronder)

Controleren:

- Doortrekken van de kop	(Zie berekening hieronder)
- Blokafschuiving	(Niet opgenomen in dit Excelblad)
- Doorsnede staal / hout	(Niet opgenomen in dit Excelblad)
- Treksterkte van de schroef en de kop	(Niet opgenomen in dit Excelblad)

Berekende waarden:

- Karakteristieke waarde van het vloeimoment:

$$M_{y,Rk} = \begin{cases} 0{,}3 f_u d^{2{,}6} & \text{voor ronde nagels} \\ 0{,}45 f_u d^{2{,}6} & \text{voor vierkante en geprofileerde nagels} \end{cases}$$

= 4012,44 [Nmm]
= 6018,66 [Nmm]

- Karakteristieke stuiksterkte in hout of LVL:

  - zonder voorgeboorde gaten

$$f_{h,k} = 0{,}082 \rho_k d^{-0{,}3}$$ = 20,06 [N/mm²]

  - met voorgeboorde gaten

$$f_{h,k} = 0{,}082(1 - 0{,}01d)\rho_k$$ = 27,75 [N/mm²]

- Karakteristieke waarde van de uittreksterkte voor schroeven:

t_{pen}	=	42,00 [mm]	(t_{pen} Is de hechtlengte in het element die de puntzijde bevat)
6 · d	=	30,00 [mm]	(Vereist is dat: $t_{pen} \geq 6d$)
$f_{ax,k} = 3{,}6 \cdot 10^{-3} \rho_k^{1{,}5}$	=	23,57 [N/mm²]	
$f_{head,k} = 70 \cdot 10^{-6} \rho_k^2$	=	8,58 [N]	(limiet voor de uittrekkracht is: $f_{head,k} d_h^2$)

- Karakteristieke uittrek kracht voor schroeven:

$$F_{ax,Rk} = (\pi d l_{ef})^{0{,}8} f_{ax,k}$$ = 4245,22 [N] (l_{ef} is de lengte van het deel van de schroefdraad in het deel waar de punt zich bevindt min een schroefdiameter)

$$F_{ax,Rk} \leq f_{head,k} d_h^2$$ ≤ 0,00 [N] ($l_{ef} \geq 6d$) (Doortrekken van de kop)

Uitkomsten:

- Voor een dunne staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min \begin{cases} 0{,}4 f_{h,k} t_1 d & \\ \underbrace{1{,}15\sqrt{2 M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 1112,126 (a); 1899,489 (b) } = **1112,13** [N]

- Voor een dikke staalplaat in een enkelsnedige verbinding:

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d & \\ \underbrace{f_{h,k} t_1 d\left[\sqrt{2 + \frac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right]}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \\ \underbrace{2{,}3\sqrt{M_{y,Rk} f_{h,k} d}}_{\text{Johansen deel}} + \underbrace{\frac{F_{ax,Rk}}{4}}_{\text{Koordeffect}} \end{cases}$$

= min { 2780,316 (c); 4026,539 (d); 2246,676 (e) } = **2246,68** [N]

0,5625·d	1253,945
0,625·d	1395,764
0,6875·d	1537,582
0,75·d	1679,401
0,8125·d	1821,22
0,875·d	1963,039
0,9375·d	2104,857

Te hanteren uitkomsten:

- Dikke plaat:	$t_2 \geq d$
- Dunne plaat:	$t_2 \leq 0{,}5 \cdot d$
- Interpoleren:	$0{,}5 \cdot d \leq t_2 \leq d$
t_2 =	3,00
d =	5,00
0,5 · d =	2,50

Appendix IV Calculated and test-based hold-down stiffness

Type of hold-down:	Simpson Strong-Tie HTT16 hold-down	Rotho Blaas GmbH Rothofixing WHT 80/620						
Fastener properties:	18 x nail Ø4,11 x 89	50 x nail Ø4,0 x 40	30 x nail Ø4,0 x 40	35 x nail Ø4,0 x 40	40 x nail Ø4,0 x 40	40 x nail Ø4,0 x 40	20 x nail Ø4,0 x 75	25 x nail Ø4,0 x 75
Density of the wood ρ [kg/m³]:	Unknown Spruce Pine Fir	421	419	407	418	438	423	401
Reference:	(2)	(1.1)	(1.2)	(1.3)	(1.4)	(1.5)	(1.6)	(1.7)
Failure mode:	Unknown	Rupture of the steel cross section	Nail fasteners pulled out	Pulling out of fasteners followed by rupture of a timber lammella	Nail fasteners pulled out	Pulling out of fasteners followed by rupture of steel cross section	Pulling out of fasteners followed by rupture of nail head near a knot	Nail fasteners pulled out

D = displacement [mm] L = load [kN]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
	1,27	13,91	0,20	10,25	0,40	7,41	0,25	8,31	0,20	8,85	0,30	9,67	0,45	7,24	0,27	8,77
	2,54	22,40	1,00	33,85	1,00	20,00	1,00	25,38	1,00	26,67	1,00	21,54	1,00	15,35	1,00	21,74
	5,08	29,41	1,35	40,98	1,59	29,64	1,49	33,23	1,50	35,38	2,24	38,67	2,65	28,95	2,30	35,09
(!) See page 136 for the references.	7,62	31,54	3,00	65,90	3,00	44,10	3,00	52,31	3,00	55,90	3,00	47,95	3,00	31,97	3,00	42,71
	10,16	32,59	5,00	92,56	5,50	65,64	5,00	72,31	5,50	77,69	5,65	80,26	6,45	56,68	7,00	70,95
	22,61	30,52	6,28	102,46	7,30	72,31	9,00	83,08	9,75	88,46	10,00	95,41	6,70	55,24	11,75	87,72
	25,40	28,22	6,18	86,26	10,00	74,10	9,50	75,90	12,30	86,56	10,70	96,67	8,40	62,15	11,85	85,68
	29,34	26,49			11,00	70,51	10,60	81,28	14,00	75,90			8,65	60,46	14,00	83,99
	28,96	8,25			12,50	64,87	13,70	71,69					14,40	72,38	15,65	79,64
													15,50	65,98		
													17,20	62,40		
Rerence in original report:			4_40_1		4_40_2		4_40_3		4_40_4		4_40_5		4_75_1		4_75_2	

Test-based determinitaion of the hold-down stiffness according to NEN-EN 26891								
F_{max} [kN]	32,59	102,46	74,10	83,08	88,46	96,67	72,38	87,72
F_{04} [kN]	13,04	40,98	29,64	33,23	35,38	38,67	28,95	35,09
F_{01} [kN]	3,26	10,25	7,41	8,31	8,85	9,67	7,24	8,77
v_{04} [mm]	0,94	1,35	1,59	1,49	1,50	2,24	2,65	2,30
v_{01} [mm]	0,14	0,20	0,40	0,25	0,20	0,30	0,45	0,27
$v_{i,mod}$ [mm]	1,07	1,53	1,59	1,66	1,74	2,59	2,93	2,71
$K_{hd;test}$ (Based on F_{04}, F_{01}) [kN/mm]	12,22	26,82	18,60	20,02	20,34	14,93	9,87	12,96
							#VERW!	

Slip-modulus of the fastener (K_{ser}) according to NEN-EN 1995-1-1								
$\rho_{m,1}$ [kg/m³] strength class C24	420,00	420,000	420,00	420,00	420,00	420,00	420,00	420,00
d [mm]	4,11	4,000	4,00	4,00	4,00	4,00	4,00	4,00
K_{ser} [kN/mm] = $\rho_m^{1,5} \cdot d^{0,8}/30$ (C24)	0,89	0,870	0,87	0,87	0,87	0,87	0,87	0,87
$2 \cdot K_{ser}$ [kN/mm]	1,78	1,740	1,74	1,74	1,74	1,74	1,74	1,74
n = number of fasteners [-]	18	50	30	35	40	40	20	25
$n \cdot K_{ser}$ [kN/mm]	32,00	86,98	52,19	60,88	69,58	69,58	34,79	43,49

Comparison test-based $K_{hd;test}$ and analytically derived K_{hd}:								
$n \cdot K_{ser} / K_{hd;test}$ [-]	2,62	3,24	2,81	3,04	3,42	4,66	3,52	3,35

Calculation of the hold-down stiffness with an analytical aproach								
$F_{v,Rk}$ per fastener [kN]	1,48	1,83	1,83	1,83	1,83	1,83	2,50	2,50
$F_{v,Rd}$ per fastener [kN] $k_{mod} = 0,9$ $\gamma_M = 1,3$	1,02	1,27	1,27	1,27	1,27	1,27	1,73	1,73
$F_{hd,Rd} = n \cdot F_{v,Rd}$ [kN]	18,44	63,35	38,01	44,34	50,68	50,68	34,62	43,27

Calculation of the cross-sectional stiffness of the steel and timber elements:								
E_{steel} [N/mm²]	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00
A_{steel} [mm²]	193,04 = (63,5 x 3,04)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)	240,00 = (80,0 x 3,0)
l_1 (steel) [mm]	111,13	415,00	415,00	415,00	415,00	415,00	415,00	415,00
$E \cdot A_{steel}/l_1$ [kN/mm]	364,78	121,45	121,45	121,45	121,45	121,45	121,45	121,45
$A_{steel;foot}$ [mm²]	347,47 = (63,5+50,8) x 3,04	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0
l_{foot} [mm]	173,04	150,00	150,00	150,00	150,00	150,00	150,00	150,00
$E \cdot A_{foot}/l_{foot}$ [kN/mm]	421,69	228,20	228,20	228,20	228,20	228,20	228,20	228,20
E_{timber} [N/mm²] (C24)	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00
A_{timber} [mm²]	10146,00 = (114,00 x 89,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)	16384,00 = (128,00 x 128,00)
l_2 (timber) [mm]	111,13	205,00	205,00	205,00	205,00	205,00	205,00	205,00
$E \cdot A_{timber}/l_2$ [kN/mm]	1004,33	879,14	879,14	879,14	879,14	879,14	879,14	879,14

Calulation of the equivalent stiffness of the hold-down:								
K_{hd} [kN/mm]	26,77	39,60	30,38	33,14	35,55	35,55	23,53	27,21

Calculation of the hole-clearance reduction:								
d_{hole} [mm]	5	5,00	5,00	5,00	5,00	5,00	5,00	5,00
$d_{fastener}$ [mm]	4,11	4,00	4,00	4,00	4,00	4,00	4,00	4,00
$K_{hd;reduced}$ [kN/mm]	13,99	26,70	20,09	22,11	23,67	24,36	16,73	19,61

Comparison between k_{hd} and $k_{hd;test}$:								
$K_{hd;reduced} / K_{hd;test}$ [-]	1,14	1,00	1,08	1,10	1,16	1,63	1,70	1,51

Type of hold-down:																
Fastener properties:	25 x nail Ø4,0 x 75		25 x nail Ø4,0 x 75		22 x nail Ø4,0 x 75		30 x screw Ø5,0 x 50		27 x screw Ø5,0 x 50		25 x screw Ø5,0 x 50		25 x screw Ø5,0 x 50		22 x screw Ø5,0 x 50	
Density of the wood ρ [kg/m³]:	452		431		421		423		387		411		418		434	
Reference:	(1.8)		(1.9)		(1.10)		(1.11)		(1.12)		(1.13)		(1.14)		(1.15)	
Failure mode:	Nail fasteners pulled out		Rupture of steel cross section		Nail fasteners pulled out		Screw fasteners pulled out		Pulling out of fasteners followed by yielding of the cross-section		Screw fasteners pulled out		Screw fasteners pulled out		Screw fasteners pulled out	
D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:	D:	L:
L = load [kN]	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
	0,26	9,78	0,35	9,78	0,25	8,95	0,03	9,82	0,02	9,97	0,02	9,69	0,02	7,80	0,02	8,66
	1,00	24,30	1,00	20,97	1,00	19,95	0,68	39,28	0,90	39,90	0,85	38,77	0,37	31,20	0,33	34,64
	2,05	39,12	2,33	39,12	2,13	35,80	1,00	46,80	1,00	40,41	1,00	40,66	1,00	42,20	1,00	46,55
(!) See page 136 for the references.	3,00	52,43	3,00	47,06	3,00	46,29	3,00	69,16	3,00	61,59	3,00	62,81	3,00	61,89	3,00	65,12
	7,00	84,91	5,50	76,73	6,00	72,53	6,00	87,06	5,50	76,98	7,00	84,40	6,50	74,58	5,50	75,60
	11,55	97,80	8,00	94,73	9,00	88,24	9,35	98,21	8,00	87,88	10,00	92,58	10,00	78,01	8,00	80,72
	12,50	90,64	8,65	97,80	9,15	88,75	9,55	96,93	11,50	97,32	14,30	95,57	14,00	77,49	12,00	82,86
	14,70	73,25	8,65	96,78	9,40	87,37	9,80	97,44	14,90	99,74	14,40	96,93	16,30	77,65	16,80	86,60
					9,45	88,59	12,75	97,34	15,50	95,65	15,00	90,69	17,00	74,42	16,80	85,32
					9,90	89,51	14,50	94,37	16,50	91,97						
					10,00	87,98	15,80	93,61	16,40	90,00						
Rerence in original report:	4_75_3		4_75_4		4_75_5		5_50_1		5_50_2		5_50_3		5_50_4		5_50_5	

Test-based determinitaion of the hold-down stiffness according to NEN-EN 26891								
F_{max} [kN]	97,80	97,80	89,51	98,21	99,74	96,93	78,01	86,60
F_{04} [kN]	39,12	39,12	35,80	39,28	39,90	38,77	31,20	34,64
F_{01} [kN]	9,78	9,78	8,95	9,82	9,97	9,69	7,80	8,66
v_{04} [mm]	2,05	2,33	2,13	0,68	0,90	0,85	0,37	0,33
v_{01} [mm]	0,26	0,35	0,25	0,03	0,02	0,02	0,02	0,02
$v_{i,mod}$ [mm]	2,39	2,64	2,51	0,87	1,17	1,11	0,47	0,41
$K_{hd;test}$ (Based on F_{04} , F_{01}) [kN/mm]	16,39	14,82	14,28	45,33	34,00	35,03	66,87	83,81

Slip-modulus of the fastener (K_{ser}) according to NEN-EN 1995-1-1								
$\rho_{m,1}$ [kg/m³] strength class C24	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00
d [mm]	4,00	4,00	4,00	3,30 d_{eff} = 0,66 · d	3,30 d_{eff} = 0,66 · d	3,30 d_{eff} = 0,66 · d	3,30 d_{eff} = 0,66 · d	3,30 d_{eff} = 0,66 · d
K_{ser} [kN/mm] = $\rho_m^{1,5} \cdot d^{0,8}/30$ (C24)	0,87	0,87	0,87	1,23	1,23	1,23	1,23	1,23
$2 \cdot K_{ser}$ [kN/mm]	1,74	1,74	1,74	2,47	2,47	2,47	2,47	2,47
n = number of fasteners [-]	25	25	22	30	27	25	25	22
$n \cdot K_{ser}$ [kN/mm]	43,49	43,49	38,27	74,10	66,69	61,75	61,75	54,34

Comparison test-based $K_{hd;test}$ and analytically derived K_{hd}:								
$n \cdot K_{ser} / K_{hd;test}$ [-]	2,65	2,93	2,68	1,63	1,96	1,76	0,92	0,65

Calculation of the hold-down stiffness with an analytical aproach								
$F_{v,Rk}$ per fastener [kN]	2,50	2,50	2,50	2,51	2,51	2,51	2,51	2,51
$F_{v,Rd}$ per fastener [kN] k_{mod} = 0,9 γ_M = 1,3	1,73	1,73	1,73	1,74	1,74	1,74	1,74	1,74
$F_{hd,Rd} = n \cdot F_{v,Rd}$ [kN]	43,27	43,27	38,08	52,13	46,92	43,44	43,44	38,23

Calculation of the cross-sectional stiffness of the steel and timber elements:								
E_{steel} [N/mm²]	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00	210000,00
A_{steel} [mm²]	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)	240,00 =(80,0 x 3,0)
l_1 (steel) [mm]	415,00	415,00	415,00	415,00	415,00	415,00	415,00	415,00
$E \cdot A_{steel}/l_1$ [kN/mm]	121,45	121,45	121,45	121,45	121,45	121,45	121,45	121,45
$A_{steel;foot}$ [mm²]	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0	163,00 =(80,0+83,0) x 3,0
l_{foot} [mm]	150,00	150,00	150,00	150,00	150,00	150,00	150,00	150,00
$E \cdot A_{foot}/l_{foot}$ [kN/mm]	228,20	228,20	228,20	228,20	228,20	228,20	228,20	228,20
E_{timber} [N/mm²] (C24)	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00	11000,00
A_{timber} [mm²]	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)	16384,00 =(128,00 x 128,00)
l_2 (timber) [mm]	205,00	205,00	205,00	205,00	205,00	205,00	205,00	205,00
$E \cdot A_{timber}/l_2$ [kN/mm]	879,14	879,14	879,14	879,14	879,14	879,14	879,14	879,14

Calulation of the equivalent stiffness of the hold-down:								
K_{hd} [kN/mm]	27,21	27,21	25,07	36,70	34,78	33,39	33,39	31,10

Calculation of the hole-clearance reduction:								
d_{hole} [mm]	5,00	5,00	5,00	5,00	5,00	5,00	5,00	5,00
$d_{fastener}$ [mm]	4,00	4,00	4,00	5,00	5,00	5,00	5,00	5,00
$K_{hd;reduced}$ [kN/mm]	20,19	20,19	18,57	36,70	34,78	33,39	33,39	31,10

Comparison between k_{hd} and $k_{hd;test}$:								
$K_{hd;reduced} / K_{hd;test}$ [-]	1,23	1,36	1,30	0,81	1,02	0,95	0,50	0,37

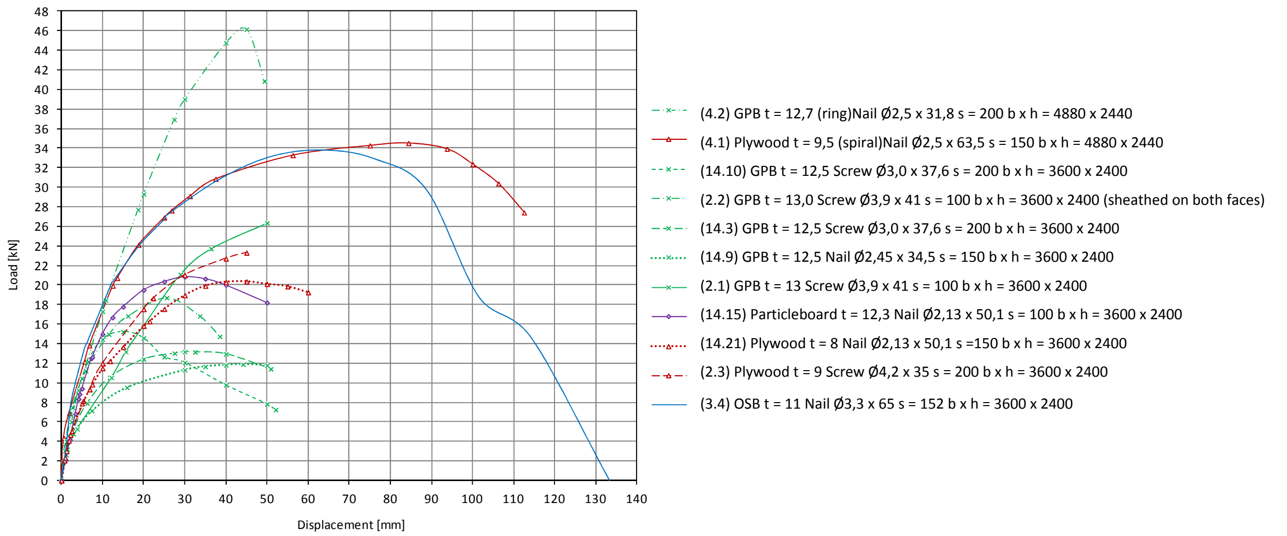
Reference list belonging to the calculation sheet and column charts

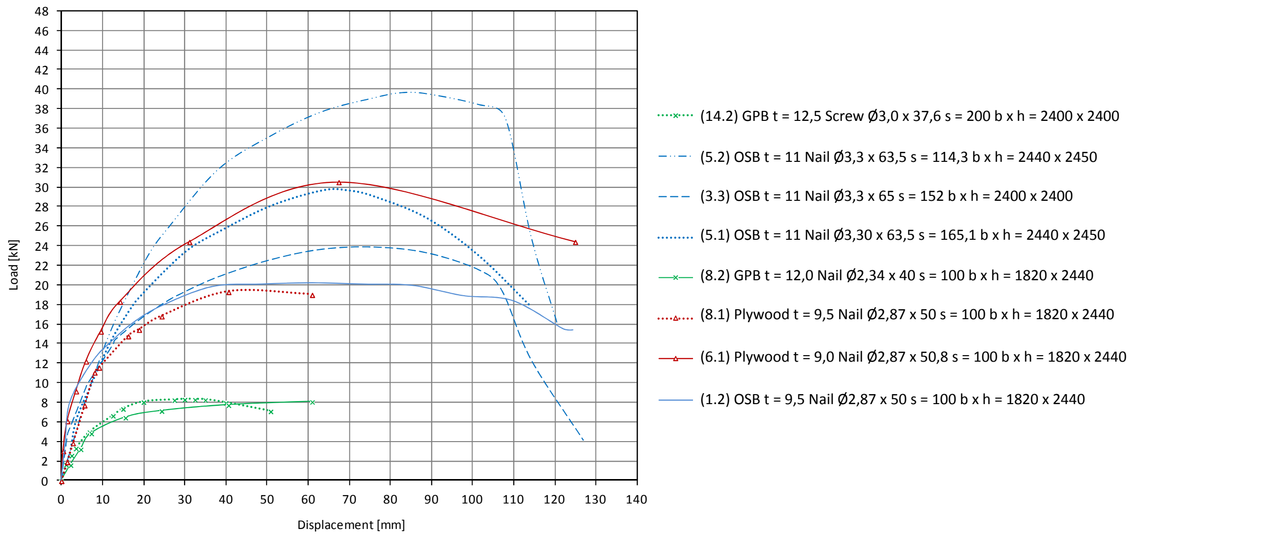
(1) (Rotho Blaas GmbH Rothofixing, 07-04-2010)
(2) (NAHB Research Center, 2003)

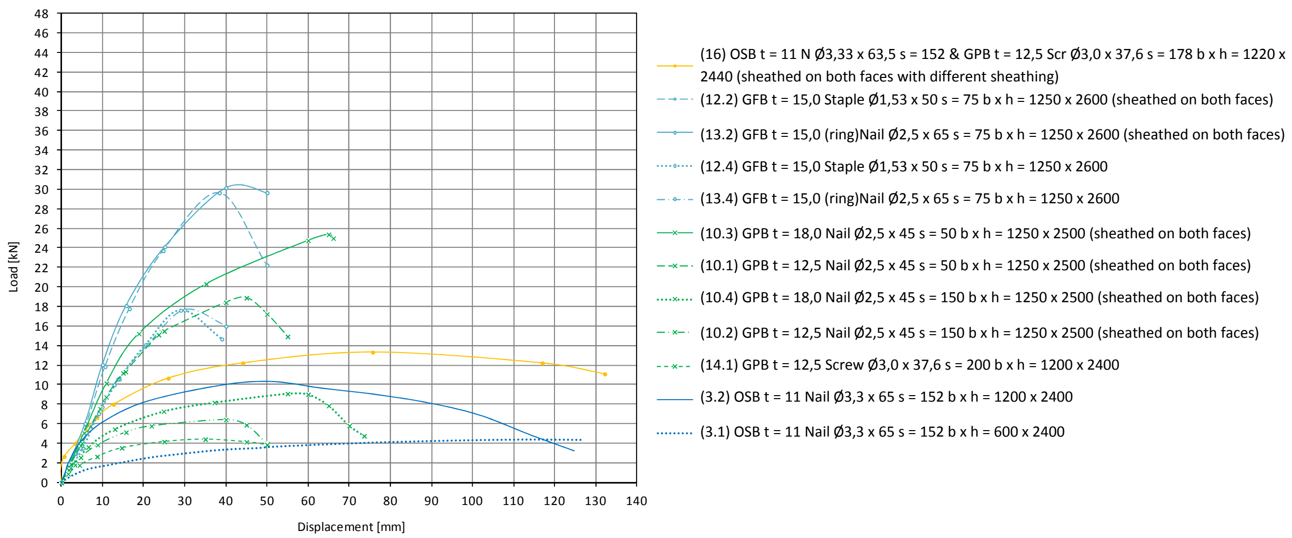
The literature can be found in the sources displayed on page 105.

Appendix V Load-displacement data racking tests

Below the load-displacement graphs are shown obtained from shear-wall tests found in the literature review.







On the next page reference is made to the source in which the graphs were found.

Reference list belonging to the load-displacement data

(1) (Yasumura & Kawai, 1997)
(2) (Andreasson, 2000)
(3) (Salenikovich, 2000)
(4) (Ni et al., 2010)
(5) (NAHB Research Center Inc., 2005)
(6) (Yasumura & Karacabeyli, 2007)
(8) (Yasumura, 1991)
(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002)
(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 04-10-2001)
(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)
(14) (Källsner, 1984)
(16) (Dolan & Toothman, 2003)

The literature can be found in the sources displayed on page 105.

Appendix VI Test-based shear-wall racking stiffness

Step 1): Load-displacement data shear-wall test				
Reference:	(3.1)	(3.2)	(1.2)	(5.1)
Material:	OSB	OSB	OSB	OSB
Material thickness t: [mm]	11	11	9,5	11
Fastener properties:	N Ø3,3x65 s = 152	N Ø3,3x65 s = 152	N Ø2,87x50 s = 100	N Ø3,3x63,5 s = 165,1
Diaphragm dimension b x h:	600x2400	1200x2400	1820 x 2440	2440 x 2450
Anchorage condition:	Provided with hold-down Simpson Strong-Tie HTT22	Provided with hold-down Simpson Strong-Tie HTT22	No anchorage	Provided with hold-down Simpson Strong-Tie HTT22
(As)symtric:	Assymetric	Assymetric	Assymetric	Assymetric
Remark:	mean of 2 tests framing elements: 38 x 89 mm double end studs	mean of 2 tests framing elements: 38 x 89 mm double end studs	Mean of 3 static tests (gecorrigeerd voor: sliding & tilting) echte schrank mechanisme framing elements: 38 x 89 mm double end studs	different spacing on edges, mean of 3 tests framing elements: 38 x 89 mm double end studs Slip of the bottom plate was subtracted from the global wall deformation.

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	1,6E+00	4,4E+02	1,0E+00	1,0E+03	5,2E-01	2,0E+03	2,1E+00	3,0E+03
	2,9E+00	7,3E+02	1,8E+00	2,1E+03	1,0E+00	4,0E+03	3,4E+00	5,9E+03
	3,7E+00	8,8E+02	4,7E+00	4,1E+03	2,2E+00	8,1E+03	5,8E+00	8,5E+03
	6,4E+00	1,3E+03	1,0E+01	6,2E+03	7,6E+00	1,2E+04	6,4E+00	8,9E+03
	1,1E+01	1,8E+03	2,1E+01	8,2E+03	1,2E+01	1,4E+04	9,4E+00	1,2E+04
	1,3E+01	1,9E+03	3,8E+01	9,8E+03	1,8E+01	1,6E+04	1,3E+01	1,5E+04
	1,9E+01	2,4E+03	5,1E+01	1,0E+04	2,4E+01	1,8E+04	1,7E+01	1,8E+04
	2,3E+01	2,6E+03	6,4E+01	9,6E+03	3,7E+01	2,0E+04	1,9E+01	1,9E+04
	2,5E+01	2,7E+03	7,6E+01	9,0E+03	4,9E+01	2,0E+04	2,5E+01	2,2E+04
	3,8E+01	3,3E+03	8,9E+01	8,1E+03	6,1E+01	2,0E+04	3,1E+01	2,4E+04
	4,8E+01	3,5E+03	1,0E+02	6,9E+03	7,3E+01	2,0E+04	3,8E+01	2,5E+04
	5,1E+01	3,6E+03	1,1E+02	4,8E+03	8,5E+01	2,0E+04	5,1E+01	2,8E+04
	6,4E+01	3,9E+03	1,2E+02	3,2E+03	9,8E+01	1,9E+04	6,4E+01	3,0E+04
	7,6E+01	4,1E+03	1,2E+02	3,2E+03	1,1E+02	1,8E+04	7,0E+01	3,0E+04
	8,9E+01	4,2E+03	1,2E+02	3,2E+03	1,2E+02	1,5E+04	7,6E+01	2,9E+04
	1,0E+02	4,3E+03	1,2E+02	3,2E+03	1,2E+02	1,5E+04	8,9E+01	2,7E+04
	1,1E+02	4,4E+03	1,2E+02	3,2E+03	1,2E+02	1,5E+04	1,0E+02	2,3E+04
	1,3E+02	4,3E+03	1,2E+02	3,2E+03			1,1E+02	1,8E+04

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	4,4E+03	1,0E+04	2,0E+04	3,0E+04
F_{04} [N] (40 % of F_{max})	1,8E+03	4,1E+03	8,1E+03	1,2E+04
F_{02} [N] (20% of F_{max})	8,8E+02	2,1E+03	4,0E+03	5,9E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	1,1E+01	4,7E+00	2,2E+00	9,4E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	3,7E+00	1,8E+00	1,0E+00	3,4E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	1,1E+02	7,2E+02	3,5E+03	9,9E+02
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	2,9E+00	3,4E+00	4,4E+00	4,9E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(3.3)	(5.2)	(3.4)	(6.1)
Material:	OSB	OSB	OSB	Plywood
Material thickness t: [mm]	11	11	11	9
Fastener properties:	N Ø3,3x65 s = 152	N Ø3,3x63,5 s = 114,3	N Ø3,3x65 s = 152	N Ø2,87x50,8 s = 100
Diaphragm dimension b x h:	2400x2400	2440 x 2450	3600x2400	1820 x 2440
Anchorage condition:	Provided with hold-down Simpson Strong-Tie HTT22	Provided with hold-down Simpson Strong-Tie HTT22	Provided with hold-down Simpson Strong-Tie HTT22	Provided with hold-down at both end studs of the wall
(As)symtric:	Assymetric	Assymetric	Assymetric	Assymetric
Remark:	mean of 2 tests framing elements: 38 x 89 mm double end studs	different spacing on edges, mean of 3 tests framing elements: 38 x 89 mm double end studs Slip of the bottom plate was subtracted from the global wall deformation.	mean of 2 tests framing elements: 38 x 89 mm double end studs	framing elements: 38 x 89 mm double end studs larch plywood (JAS Grade 2) (Load-displacement graph represents story drift. It is assumed that Yasumura distracted bottom rail slip from top rail displacement, similar to his other publications in which he called this the true shear deformation)

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	8,1E-01	2,4E+03	3,0E+00	4,0E+03	1,2E+00	3,4E+03	4,7E-01	3,1E+03
	1,6E+00	4,8E+03	5,7E+00	7,9E+03	2,0E+00	6,8E+03	1,5E+00	6,1E+03
	2,2E+00	5,4E+03	6,0E+00	8,0E+03	2,8E+00	8,9E+03	3,6E+00	9,2E+03
	6,1E+00	9,5E+03	7,9E+00	1,1E+04	5,7E+00	1,4E+04	6,0E+00	1,2E+04
	6,4E+00	9,8E+03	1,3E+01	1,6E+04	6,4E+00	1,4E+04	9,6E+00	1,5E+04
	1,3E+01	1,4E+04	1,9E+01	2,1E+04	1,2E+01	2,0E+04	1,4E+01	1,8E+04
	1,3E+01	1,4E+04	2,2E+01	2,4E+04	1,3E+01	2,0E+04	3,1E+01	2,4E+04
	1,9E+01	1,6E+04	2,5E+01	2,5E+04	1,9E+01	2,4E+04	6,7E+01	3,1E+04
	2,5E+01	1,8E+04	3,8E+01	3,2E+04	2,5E+01	2,7E+04	1,3E+02	2,4E+04
	2,9E+01	1,9E+04	5,1E+01	3,5E+04	2,5E+01	2,7E+04		
	3,8E+01	2,1E+04	6,4E+01	3,8E+04	3,8E+01	3,1E+04		
	5,1E+01	2,3E+04	7,6E+01	3,9E+04	5,1E+01	3,3E+04		
	6,4E+01	2,4E+04	8,3E+01	4,0E+04	6,4E+01	3,4E+04		
	7,6E+01	2,4E+04	8,9E+01	4,0E+04	7,6E+01	3,3E+04		
	8,9E+01	2,3E+04	1,0E+02	3,8E+04	8,9E+01	3,0E+04		
	1,0E+02	2,2E+04	1,1E+02	3,7E+04	1,0E+02	1,9E+04		
	1,1E+02	1,9E+04	1,1E+02	2,5E+04	1,1E+02	1,5E+04		
	1,1E+02	1,2E+04	1,2E+02	1,6E+04	1,3E+02	0,0E+00		
	1,3E+02	4,1E+03						

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	2,4E+04	4,0E+04	3,4E+04	3,1E+04
F_{04} [N] (40 % of F_{max})	9,5E+03	1,6E+04	1,4E+04	1,2E+04
F_{02} [N] (20% of F_{max})	4,8E+03	7,9E+03	6,8E+03	6,1E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	6,1E+00	1,3E+01	5,7E+00	6,0E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	1,6E+00	5,7E+00	2,0E+00	1,5E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	1,1E+03	1,1E+03	1,8E+03	1,3E+03
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	4,0E+00	6,5E+00	3,8E+00	6,7E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(8.1)	(2.3)	(14.21)	(4.1)
Material:	Plywood	Plywood	Plywood	Plywood
Material thickness t: [mm]	9,5	9	8	9,5
Fastener properties:	N Ø2,87x50 s=100 mm	Scr Ø4,2x35 s = 200	N Ø2,13x50,1 s = 150 mm	N Ø2,5x63,5 (spiral) s = 150
Diaphragm dimension b x h:	1820 x 2440	3600 x 2400	3600x2400	4880 x 2440
Anchorage condition:	2 tie-rods door wand	Partially anchored with steel angle bracket	Uplift prevented met testopstelling	Provided with hold-down -> no uplift
(As)symtric:	Assymetric	Assymetric	Assymetric	Assymetric
Remark:	framing elements: 38 x 89 mm Mean of 2 static tests Measured deformations were corrected for deflections due to hold-down strain, slip of the bottom rail, and compression perpendicular to grain.	framing elements: 45 x 90 mm An end stop at the bottom plate prevents the wall from sliding.	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.	framing elements: 38 x 89 mm double end studs stud spacing: 406 mm c.t.c.

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	1,4E+00	1,9E+03	8,8E-01	2,3E+03	7,4E-01	2,0E+03	5,7E-01	4,3E+03
	2,8E+00	3,9E+03	1,3E+00	3,0E+03	1,8E+00	4,1E+03	1,9E+00	6,9E+03
	5,6E+00	7,7E+03	2,2E+00	4,7E+03	2,5E+00	5,1E+03	5,7E+00	1,2E+04
	8,1E+00	1,1E+04	6,9E+00	9,3E+03	5,0E+00	7,9E+03	6,8E+00	1,4E+04
	9,2E+00	1,2E+04	1,0E+01	1,2E+04	5,3E+00	8,1E+03	1,3E+01	2,0E+04
	1,6E+01	1,5E+04	2,0E+01	1,8E+04	7,5E+00	9,8E+03	1,4E+01	2,1E+04
	1,9E+01	1,5E+04	2,2E+01	1,9E+04	1,0E+01	1,1E+04	1,9E+01	2,4E+04
	2,4E+01	1,7E+04	3,0E+01	2,1E+04	1,2E+01	1,2E+04	2,5E+01	2,7E+04
	4,1E+01	1,9E+04	4,0E+01	2,3E+04	1,5E+01	1,4E+04	2,7E+01	2,8E+04
	6,1E+01	1,9E+04	4,5E+01	2,3E+04	2,0E+01	1,6E+04	3,1E+01	2,9E+04
					2,1E+01	1,6E+04	3,8E+01	3,1E+04
					2,5E+01	1,8E+04	5,6E+01	3,3E+04
					3,0E+01	1,9E+04	7,5E+01	3,4E+04
					3,5E+01	2,0E+04	8,4E+01	3,5E+04
					4,0E+01	2,0E+04	9,4E+01	3,4E+04
					4,5E+01	2,0E+04	1,0E+02	3,2E+04
					5,0E+01	2,0E+04	1,1E+02	3,0E+04
					5,5E+01	2,0E+04	1,1E+02	2,7E+04
					6,0E+01	1,9E+04		

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	1,9E+04	2,3E+04	2,0E+04	3,5E+04
F_{04} [N] (40 % of F_{max})	7,7E+03	9,3E+03	8,1E+03	1,4E+04
F_{02} [N] (20% of F_{max})	3,9E+03	4,7E+03	4,1E+03	6,9E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	5,6E+00	6,9E+00	5,3E+00	6,8E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	2,8E+00	2,2E+00	1,8E+00	1,9E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	1,4E+03	1,0E+03	1,2E+03	1,4E+03
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	4,2E+00	2,6E+00	2,3E+00	2,8E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(14.15)	(14.1)	(10.2)	(10.4)
Material:	Particleboard	Gypsum paper board	Gypsum paper board	Gypsum paper board
Material thickness t: [mm]	12,3	12,5	12,5	18
Fastener properties:	N Ø2,13x50,1 s = 100 mm	Scr Ø3,0x37,6 s = 200 mm	N Ø2,5x45 s = 150 mm	N Ø2,5x45 s = 150 mm
Diaphragm dimension b x h:	3600x2400	1200x2400	1250x2500	1250x2500
Anchorage condition:	Uplift prevented met testopstelling	Uplift prevented met testopstelling	BMF-hold down anchorage	BMF-hold down anchorage
(As)symtric:			Symmetric	Symmetric
Remark:	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.	Knauf, mean of 3 tests framing elements: 40 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.	Knauf, mean of 3 tests framing elements: 40 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	1,0E+00	2,1E+03	1,6E+00	8,9E+02	1,8E+00	1,3E+03	3,3E+00	1,8E+03
	2,1E+00	4,2E+03	4,4E+00	1,8E+03	4,7E+00	2,6E+03	6,6E+00	3,6E+03
	3,4E+00	6,8E+03	8,7E+00	2,7E+03	8,8E+00	3,9E+03	1,3E+01	5,5E+03
	4,2E+00	8,3E+03	1,5E+01	3,5E+03	1,6E+01	5,1E+03	2,5E+01	7,3E+03
	4,4E+00	8,8E+03	2,5E+01	4,2E+03	2,2E+01	5,8E+03	3,7E+01	8,2E+03
	5,0E+00	9,4E+03	3,5E+01	4,4E+03	4,0E+01	6,4E+03	5,5E+01	9,1E+03
	7,3E+00	1,3E+04	4,5E+01	4,2E+03	4,5E+01	5,9E+03	6,0E+01	9,0E+03
	7,5E+00	1,3E+04	5,0E+01	3,9E+03	5,0E+01	4,0E+03	6,5E+01	7,9E+03
	1,0E+01	1,5E+04					7,0E+01	5,8E+03
	1,2E+01	1,7E+04					7,4E+01	4,8E+03
	1,5E+01	1,8E+04						
	2,0E+01	1,9E+04						
	2,5E+01	2,0E+04						
	3,0E+01	2,1E+04						
	3,5E+01	2,1E+04						
	4,0E+01	2,0E+04						
	5,0E+01	1,8E+04						

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	2,1E+04	4,4E+03	6,4E+03	9,1E+03
F_{04} [N] (40 % of F_{max})	8,3E+03	1,8E+03	2,6E+03	3,6E+03
F_{02} [N] (20% of F_{max})	4,2E+03	8,9E+02	1,3E+03	1,8E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	4,2E+00	4,4E+00	4,7E+00	6,6E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	2,1E+00	1,6E+00	1,8E+00	3,3E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	2,0E+03	3,2E+02	4,4E+02	5,6E+02
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	2,3E+00	1,5E+00	1,0E+00	1,5E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(10.1)	(10.3)	(8.2)	(14.2)
Material:	Gypsum paper board	Gypsum paper board	Gypsum paper board	Gypsum paper board
Material thickness t: [mm]	12,5	18	12	12,5
Fastener properties:	N Ø2,5x45 s = 50 mm	N Ø2,5x45 s = 50 mm	N Ø2,34x40 s=100 mm	Scr Ø3,0x37,6 s = 200 mm
Diaphragm dimension b x h:	1250x2500	1250x2500	1820 x 2440	2400x2400
Anchorage condition:	BMF-hold down anchorage	BMF-hold down anchorage	2 tie-rods door wand	Uplift prevented met testopstelling
(As)symtric:	Symmetric	Symmetric	Assymetric	
Remark:	Knauf, mean of 3 tests framing elements: 40 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.	Knauf, mean of 3 tests framing elements: 40 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.	framing elements: 38 x 89 mm Mean of 2 static tests Measured deformations were corrected for deflections due to hold-down strain, slip of the bottom rail, and compression perpendicular to grain.	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	2,4E+00	1,9E+03	5,5E+00	5,1E+03	2,3E+00	1,6E+03	1,1E+00	1,7E+03
	4,7E+00	3,8E+03	1,1E+01	1,0E+04	4,7E+00	3,2E+03	2,5E+00	2,6E+03
	5,0E+00	4,0E+03	1,9E+01	1,5E+04	7,3E+00	4,8E+03	3,6E+00	3,3E+03
	9,4E+00	7,6E+03	3,5E+01	2,0E+04	1,6E+01	6,5E+03	6,9E+00	5,0E+03
	1,1E+01	8,8E+03	6,0E+01	2,5E+04	2,4E+01	7,1E+03	1,3E+01	6,7E+03
	1,5E+01	1,1E+04	6,5E+01	2,5E+04	4,1E+01	7,7E+03	1,5E+01	7,3E+03
	1,6E+01	1,1E+04	6,6E+01	2,5E+04	6,1E+01	8,1E+03	2,0E+01	8,1E+03
	2,4E+01	1,5E+04					2,8E+01	8,3E+03
	2,5E+01	1,6E+04					3,0E+01	8,3E+03
	4,0E+01	1,8E+04					3,3E+01	8,3E+03
	4,5E+01	1,9E+04					3,5E+01	8,3E+03
	5,0E+01	1,7E+04					4,0E+01	8,0E+03
	5,5E+01	1,5E+04					5,1E+01	7,1E+03

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	1,9E+04	2,5E+04	8,1E+03	8,3E+03
F_{04} [N] (40 % of F_{max})	7,6E+03	1,0E+04	3,2E+03	3,3E+03
F_{02} [N] (20% of F_{max})	3,8E+03	5,1E+03	1,6E+03	1,7E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	9,4E+00	1,1E+01	4,7E+00	3,6E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	4,7E+00	5,5E+00	2,3E+00	1,1E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	8,0E+02	9,3E+02	6,6E+02	6,9E+02
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	3,0E+00	4,1E+00	1,8E+00	1,4E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(2.1)	(14.9)	(14.3)	(2.2)
Material:	Gypsum paper board	Gypsum paper board	Gypsum paper board	Gypsum paper board
Material thickness t: [mm]	13	12,5	12,5	13
Fastener properties:	Scr Ø3,9x41 s = 100	N Ø2,45x34,5 s = 150 mm	Scr Ø3,0x37,6 s = 200 mm	Scr Ø3,9x41 s = 100
Diaphragm dimension b x h:	3600 x 2400	3600x2400	3600x2400	3600 x 2400
Anchorage condition:	Partially anchored with steel angle bracket	Uplift prevented met testopstelling	Uplift prevented met testopstelling	Partially anchored with steel angle bracket
(As)symtric:	Assymetric			Symmetric
Remark:	framing elements: 45 x 90 mm An end stop at the bottom plate prevents the wall from sliding.	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.	framing elements: 45 x 95 mm Slip of bottom plate preventd by test-setup.	framing elements: 45 x 90 mm Slip of bottom plate preventd by test-setup.

D = displacement [mm] L = load [N]	D:	L:	D:	L:	D:	L:	D:	L:
	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	1,2E+00	2,6E+03	1,0E+00	2,4E+03	1,1E+00	2,6E+03	1,3E+00	4,0E+03
	1,4E+00	3,2E+03	2,9E+00	4,8E+03	3,0E+00	5,3E+03	1,6E+00	4,6E+03
	3,8E+00	5,3E+03	7,4E+00	7,1E+03	6,3E+00	7,9E+03	2,5E+00	6,0E+03
	1,2E+01	1,1E+04	1,6E+01	9,5E+03	1,2E+01	1,1E+04	4,1E+00	9,2E+03
	1,6E+01	1,3E+04	3,0E+01	1,1E+04	2,0E+01	1,2E+04	5,0E+00	1,0E+04
	2,0E+01	1,6E+04	3,5E+01	1,2E+04	2,8E+01	1,3E+04	1,0E+01	1,7E+04
	2,9E+01	2,1E+04	4,0E+01	1,2E+04	3,0E+01	1,3E+04	1,1E+01	1,8E+04
	3,6E+01	2,4E+04	4,4E+01	1,2E+04	3,3E+01	1,3E+04	1,9E+01	2,8E+04
	5,0E+01	2,6E+04	5,0E+01	1,2E+04	4,0E+01	1,3E+04	2,0E+01	2,9E+04
					5,1E+01	1,1E+04	2,7E+01	3,7E+04
							3,0E+01	3,9E+04
							4,0E+01	4,5E+04
							4,5E+01	4,6E+04
							4,9E+01	4,1E+04

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	2,6E+04	1,2E+04	1,3E+04	4,6E+04
F_{04} [N] (40 % of F_{max})	1,1E+04	4,8E+03	5,3E+03	1,8E+04
F_{02} [N] (20% of F_{max})	5,3E+03	2,4E+03	2,6E+03	9,2E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	1,2E+01	2,9E+00	3,0E+00	1,1E+01
v_{02} [mm] (Deformation at 20% F_{max} load-level)	3,8E+00	1,0E+00	1,1E+00	4,1E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	6,3E+02	1,3E+03	1,4E+03	1,4E+03
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	2,9E+00	1,3E+00	1,5E+00	2,6E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(14.10)	(4.2)	(13.4)	(12.4)
Material:	Gypsum paper board	Gypsum paper board	Gympsum Fibre Board	Gympsum Fiber Board
Material thickness t: [mm]	12,5	12,7	15	15
Fastener properties:	Scr Ø3,0x37,6 s = 200 mm	N Ø2,5x31,8 (ring) s = 200	N Ø2,5x65 (Rillenagel) s = 75 mm	Staple Ø1,53x50 s = 75 mm
Diaphragm dimension b x h:	3600x2400	4880 x 2440	1250x2600	1250x2600
Anchorage condition:	Uplift prevented met testopstelling	Provided with hold-down -> no uplift	Volledig verankerd met hold-down	Volledig verankerd met hold-down
(As)symtric:		Assymetric	Assymetric	Assymetric
Remark:	Zelfde als 14.3, maar dan met afgewerkte / opgevulde plaatnaden	framing elements: 38 x 89 mm double end studs stud spacing: 406 mm c.t.c.	Enkelzijdig beplaat mean of 3 tests framing elements: 45 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.	Enkelzijdig beplaat mean of 3 tests framing elements: 45 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.

D = displacement [mm] L = load [N]	D:	L:	D:	L:	D:	L:	D:	L:
	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	8,3E-01	3,0E+03	4,6E-01	1,9E+03	2,3E+00	1,8E+03	5,2E+00	3,5E+03
	2,2E+00	6,1E+03	9,4E-01	3,7E+03	4,5E+00	3,5E+03	9,0E+00	7,1E+03
	4,1E+00	9,1E+03	2,7E+00	7,5E+03	9,1E+00	7,1E+03	1,4E+01	1,1E+04
	6,6E+00	1,2E+04	5,9E+00	1,1E+04	9,9E+00	7,7E+03	2,1E+01	1,4E+04
	1,0E+01	1,4E+04	1,2E+01	1,5E+04	1,3E+01	1,0E+04	2,9E+01	1,8E+04
	1,5E+01	1,5E+04	1,6E+01	1,7E+04	1,4E+01	1,1E+04	3,9E+01	1,5E+04
	2,0E+01	1,5E+04	2,6E+01	1,9E+04	2,1E+01	1,4E+04		
	2,5E+01	1,3E+04	3,4E+01	1,7E+04	3,0E+01	1,8E+04		
	3,0E+01	1,2E+04	3,9E+01	1,5E+04	4,0E+01	1,6E+04		
	4,0E+01	9,8E+03						
	5,0E+01	7,8E+03						
	5,2E+01	7,2E+03						

If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	1,5E+04	1,9E+04	1,8E+04	1,8E+04
F_{04} [N] (40 % of F_{max})	6,1E+03	7,5E+03	7,1E+03	7,1E+03
F_{02} [N] (20% of F_{max})	3,0E+03	3,7E+03	3,5E+03	3,5E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	2,2E+00	2,7E+00	9,1E+00	9,0E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	8,3E-01	9,4E-01	4,5E+00	5,2E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	2,2E+03	2,1E+03	7,8E+02	9,2E+02
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	1,7E+00	1,5E+00	5,7E+00	5,6E+00

Step 1): Load-displacement data shear-wall test				
Reference:	(13.2)		(12.2)	
Material:	Gympsum Fiber Board		Gympsum Fiber Board	
Material thickness t: [mm]	15		15	
Fastener properties:	N Ø2,5x65 (Rillenagel) s = 75 mm		Staple Ø1,53x50 s = 75 mm	
Diaphragm dimension b x h:	1250x2600		1250x2600	
Anchorage condition:	Volledig verankerd met hold-down		Volledig verankerd met hold-down	
(As)symtric:	Symmetric		Symmetric	
Remark:	Dubbelzijdig beplaat mean of 3 tests framing elements: 45 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.		Dubbelzijdig beplaat mean of 3 tests framing elements: 45 x 100 mm Slip of bottom plate prevented by a load cell measuring the horizontal force in the bottom rail.	
D = displacement [mm]	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	5,8E+00	6,0E+03	3,5E+00	3,0E+03
If no special remark is made, assymetric means that only one side of the panel is sheeted with board material. Symmetric means that both sides of the panel are sheeted with the same configuration.	1,0E+01	1,2E+04	6,1E+00	5,9E+03
	1,6E+01	1,8E+04	1,1E+01	1,2E+04
	2,5E+01	2,4E+04	1,7E+01	1,8E+04
	4,0E+01	3,0E+04	2,5E+01	2,4E+04
	5,0E+01	3,0E+04	3,8E+01	3,0E+04
	5,0E+01	3,0E+04	5,0E+01	2,2E+04

Step 2): Determination of the test-based shear-wall stiffness according to NEN-EN 594		
F_{max} [N] (Ultimate load in the test)	3,0E+04	3,0E+04
F_{04} [N] (40 % of F_{max})	1,2E+04	1,2E+04
F_{02} [N] (20% of F_{max})	6,0E+03	5,9E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	1,0E+01	1,1E+01
v_{02} [mm] (Deformation at 20% F_{max} load-level)	5,8E+00	6,1E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	1,4E+03	1,3E+03
f_{04} [kN/m] F_{04} / ($n_{panels} \cdot n_{sheets} \cdot b_1$) (distributed load on the sheet at 40% F_{max})	4,8E+00	4,7E+00

Reference list belonging to the load-displacement data

(1) (Yasumura & Kawai, 1997)
(2) (Andreasson, 2000)
(3) (Salenikovich, 2000)
(4) (Ni et al., 2010)
(5) (NAHB Research Center Inc., 2005)
(6) (Yasumura & Karacabeyli, 2007)
(8) (Yasumura, 1991)
(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002)
(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 04-10-2001)
(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)
(14) (Källsner, 1984)
(16) (Dolan & Toothman, 2003)

The literature can be found in the sources displayed on page 105.

Appendix VII Calculated shear-wall racking stiffness

Reference:	(3.1)		(3.2)		(1.2)		(5.1)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	1		1		2		2	
h_1 [mm] (element height)	2400,00		2400,00		2440,00		2450,00	
b_1 [mm] (width panel)	600,00		1200,00		910,00		1220,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		1		1	
t [mm] (sheeting thickness)	11,00		11,00		9,50		11,00	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	3400,00		3400,00		3400,00		3400,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	1080,00		1080,00		1080,00		1080,00	
b_2 [mm] (stud & rail cross-sectional width)	38,00		38,00		38,00		38,00	
h_2 [mm] (stud & rail cross-sectional height)	89,00		89,00		89,00		89,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	2		2		2		2	
s [mm] (fastener spacing along perimeter)	152,0		152,0		100,0		165,1	
K_{ser} [N/mm] (fastener slip-modulus)	912,85		912,85		816,38		912,85	
K_{hd} [N/mm] (hold-down stiffness)	10794,65		12037,53		- - -		11368,39	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	1,2E+04		1,2E+04		- - -		1,2E+04	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,9	43%	3,4	47%	4,0	72%	5,3	61%
u_{sh} [mm] (shear of the sheeting)	0,6	5%	0,7	9%	1,1	19%	1,0	12%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,9	8%	0,5	7%	0,5	9%	0,4	5%
u_{hd} [mm] (hold-down anchorage)	2,6	23%	1,4	19%	0,0	0%	1,1	12%
u_c [mm] (compression perpendicular to grain)	2,3	20%	1,3	18%	0,0	0%	1,0	11%
u_Σ [mm] (total deformation element at load level F_{04})	11,2	100%	7,4	100%	5,5	100%	8,7	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	1,6E+02		5,6E+02		1,5E+03		1,4E+03	
Step 6): Comparison								
R_{test} / R_Σ [-]	0,72		1,29		2,38		0,73	
v_{04} / u_Σ [-]	1,02		0,64		0,40		1,08	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,9	43%	3,4	47%	4,0	72%	5,3	61%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	2,2	19%	0,8	11%	1,9	35%	1,2	14%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,9	8%	0,5	7%	0,5	9%	0,4	5%
u_{hd} [mm] (hold-down anchorage)	2,6	23%	1,4	19%	0,0	0%	1,1	12%
u_c [mm] (compression perpendicular to grain)	2,3	20%	1,3	18%	0,0	0%	1,0	11%
u_Σ [mm] (total deformation element at load level F_{04})	12,8	100%	7,5	100%	6,4	100%	8,9	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	1,4E+02		5,5E+02		1,3E+03		1,3E+03	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	0,82		1,31		2,77		0,74	
v_{04} / u_Σ [-]	0,89		0,63		0,34		1,05	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	3,95		7,89		9,10		7,39	
c_i [-] (Reduction factor for slender sheeted walls)	0,50		1,00		0,75		1,00	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	540,00		540,00		427,00		540,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1,2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	1278,95		5115,79		6956,04		9537,65	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	1,37		0,80		1,16		1,24	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	1,91		0,78		0,37		1,56	
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	947,37		3789,47		5152,62		7064,93	
F_{04} / $F_{v,Rd\,(SLS)}$	1,85		1,09		1,56		1,68	
	(3) (Salenikovich, 2000)		(3) (Salenikovich, 2000)		(1) (Yasumura & Kawai, 1997)		(5) (NAHB Research Center Inc., 2005)	

Reference:	(3.3)		(5.2)		(3.4)		(6.1)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	2		2		3		2	
h_1 [mm] (element height)	2400,00		2450,00		2400,00		2440,00	
b_1 [mm] (width panel)	1200,00		1220,00		1200,00		910,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		1		1	
t [mm] (sheeting thickness)	11,00		11,00		11,00		9,00	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	3400,00		3400,00		3400,00		6000,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	1080,00		1080,00		1080,00		350,00	
b_2 [mm] (stud & rail cross-sectional width)	38,00		38,00		38,00		38,00	
h_2 [mm] (stud & rail cross-sectional height)	89,00		89,00		89,00		89,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	2		2		2		2	
s [mm] (fastener spacing along perimeter)	152,0		114,3		152,0		100,0	
K_{ser} [N/mm] (fastener slip-modulus)	912,85		912,85		912,85		713,99	
K_{hd} [N/mm] (hold-down stiffness)	13237,26		13607,91		12771,94		24634,51	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	1,2E+04		1,2E+04		1,2E+04		1,2E+04	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,0	60%	4,9	53%	3,8	66%	6,9	45%
u_{sh} [mm] (shear of the sheeting)	0,8	12%	1,3	15%	0,8	13%	5,2	33%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,3	5%	0,5	6%	0,2	3%	0,7	5%
u_{hd} [mm] (hold-down anchorage)	0,7	11%	1,2	13%	0,5	8%	0,9	6%
u_c [mm] (compression perpendicular to grain)	0,8	12%	1,3	14%	0,5	9%	1,8	12%
u_Σ [mm] (total deformation element at load level F_{04})	6,6	100%	9,2	100%	5,7	100%	15,5	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	1,4E+03		1,7E+03		2,4E+03		7,9E+02	
Step 6): Comparison								
R_{test} / R_Σ [-]	0,73		0,66		0,77		1,71	
v_{04} / u_Σ [-]	0,93		1,37		1,00		0,39	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,0	60%	4,9	53%	3,8	66%	6,9	45%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	1,0	14%	1,6	17%	0,9	16%	1,8	11%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,3	5%	0,5	6%	0,2	3%	0,7	5%
u_{hd} [mm] (hold-down anchorage)	0,7	11%	1,2	13%	0,5	8%	0,9	6%
u_c [mm] (compression perpendicular to grain)	0,8	12%	1,3	14%	0,5	9%	1,8	12%
u_Σ [mm] (total deformation element at load level F_{04})	6,7	100%	9,5	100%	5,8	100%	12,1	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	1,4E+03		1,7E+03		2,3E+03		1,0E+03	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	0,75		0,68		0,79		1,33	
v_{04} / u_Σ [-]	0,91		1,33		0,98		0,50	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	7,89		10,67		7,89		9,10	
c_i [-] (Reduction factor for slender sheeted walls)	1,00		1,00		1,00		0,75	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	540,00		540,00		540,00		397,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1,2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	10231,58		13776,61		15347,37		6467,33	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	0,93		1,15		0,88		1,89	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	1,02		2,11		0,95		1,00	
$F_{v,Rd (SLS)} = F_{v,Rd}$ /1,35	7578,95		10204,90		11368,42		4790,61	
F_{04} / $F_{v,Rd (SLS)}$	1,26		1,55		1,19		2,55	
	(3) (Salenikovich, 2000)		(5) (NAHB Research Center Inc., 2005)		(3) (Salenikovich, 2000)		(6) (Yasumura & Karacabeyli, 2007)	

Reference:	(8.1)		(2.3)		(14.21)		(4.1)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	2		3		3		4	
h_1 [mm] (element height)	2440,00		2400,00		2400,00		2440,00	
b_1 [mm] (width panel)	910,00		1200,00		1200,00		1220,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		1		1	
t [mm] (sheeting thickness)	9,50		9,00		8,00		9,50	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	6000,00		6000,00		6000,00		6000,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	350		350,00		350,00		350	
b_2 [mm] (stud & rail cross-sectional width)	38,00		45,00		45,00		38	
h_2 [mm] (stud & rail cross-sectional height)	89,00		90,00		95,00		89	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		1		1		2	
s [mm] (fastener spacing along perimeter)	100,0		200,0		150,0		150,0	
K_{ser} [N/mm] (fastener slip-modulus)	713,99		1110,63		562,45		639,35	
K_{hd} [N/mm] (hold-down stiffness)	---		1637,63		9262,50		14048,87	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	---		8,8E+03		9,3E+03		1,2E+04	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,4	52%	2,8	35%	3,6	55%	4,0	59%
u_{sh} [mm] (shear of the sheeting)	3,1	37%	2,0	25%	1,9	30%	2,1	31%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,9	11%	0,2	3%	0,2	3%	0,1	2%
u_{hd} [mm] (hold-down anchorage)	0,0	0%	2,5	32%	0,4	6%	0,2	4%
u_c [mm] (compression perpendicular to grain)	0,0	0%	0,5	6%	0,4	6%	0,3	4%
u_Σ [mm] (total deformation element at load level F_{04})	8,4	100%	8,0	100%	6,5	100%	6,7	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	9,2E+02		1,2E+03		1,2E+03		2,1E+03	
Step 6): Comparison								
R_{test} / R_Σ [-]	1,49		0,85		0,93		0,68	
v_{04} / u_Σ [-]	0,67		0,86		0,82		1,02	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	4,4	52%	2,8	35%	3,6	55%	4,0	59%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	1,1	13%	0,4	5%	0,4	6%	0,5	7%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,9	11%	0,2	3%	0,2	3%	0,1	2%
u_{hd} [mm] (hold-down anchorage)	0,0	0%	2,5	32%	0,4	6%	0,2	4%
u_c [mm] (compression perpendicular to grain)	0,0	0%	0,5	6%	0,4	6%	0,3	4%
u_Σ [mm] (total deformation element at load level F_{04})	6,3	100%	6,5	100%	5,0	100%	5,1	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	1,2E+03		1,4E+03		1,6E+03		2,7E+03	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	1,12		0,69		0,71		0,52	
v_{04} / u_Σ [-]	0,89		1,06		1,06		1,34	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	9,10		6,00		8,00		8,13	
c_i [-] (Reduction factor for slender sheeted walls)	0,75		1,00		1,00		1,00	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	400,00		389,00		255,00		334,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1,2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	6516,20		8402,40		7344,00		13039,36	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	1,19		1,11		1,11		1,06	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	0,94		1,14		0,89		1,13	
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	4826,81		6224,00		5440,00		9658,79	
F_{04} / $F_{v,Rd\,(SLS)}$	1,60		1,50		1,50		1,43	
	(8) (Yasamura, 1991)		(2) (Andreasson, 2000)		(14) (Källsner, 1984)		(4) (Ni, Shim, Karacabeyli, 2010)	

Reference:	(14.15)		(14.1)		(10.2)		(10.4)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	3		1		1		1	
h_1 [mm] (element height)	2400,00		2400,00		2500,00		2500,00	
b_1 [mm] (width panel)	1200,00		1200,00		1250,00		1250,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		2		2	
t [mm] (sheeting thickness)	12,30		12,50		12,50		18,00	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	2000,00		1100,00		1100,00		1100,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	960,00		700,00		700,00		700,00	
b_2 [mm] (stud & rail cross-sectional width)	45,00		45,00		40,00		40,00	
h_2 [mm] (stud & rail cross-sectional height)	95,00		95,00		100,00		100,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		1		1		1	
s [mm] (fastener spacing along perimeter)	100,0		200,0		150,0		150,0	
K_{ser} [N/mm] (fastener slip-modulus)	728,96		6,7E+02		5,6E+02		5,6E+02	
K_{hd} [N/mm] (hold-down stiffness)	9262,50		9262,50		1,5E+04		1,9E+04	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	9,3E+03		9,3E+03		9,1E+03		9,1E+03	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,9	57%	2,7	54%	1,6	38%	2,3	40%
u_{sh} [mm] (shear of the sheeting)	0,5	14%	0,4	8%	0,3	7%	0,3	5%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,2	6%	0,4	7%	0,6	14%	0,8	14%
u_{hd} [mm] (hold-down anchorage)	0,4	12%	0,8	15%	0,7	16%	0,8	13%
u_c [mm] (compression perpendicular to grain)	0,4	12%	0,8	15%	1,1	26%	1,6	28%
u_Σ [mm] (total deformation element at load level F_{04})	3,4	100%	5,0	100%	4,3	100%	5,8	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	2476,82		3,6E+02		5,9E+02		6,3E+02	
Step 6): Comparison								
R_{test} / R_Σ [-]	0,81		0,90		0,74		0,89	
v_{04} / u_Σ [-]	1,24		0,89		1,08		1,13	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,9	57%	2,7	54%	1,6	38%	2,3	40%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	0,8	25%	1,0	19%	0,7	16%	0,7	12%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,2	6%	0,4	7%	0,6	14%	0,8	14%
u_{hd} [mm] (hold-down anchorage)	0,4	12%	0,8	15%	0,7	16%	0,8	13%
u_c [mm] (compression perpendicular to grain)	0,4	12%	0,8	15%	1,1	26%	1,6	28%
u_Σ [mm] (total deformation element at load level F_{04})	3,7	100%	5,5	100%	4,7	100%	6,2	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	2,2E+03		3,2E+02		5,4E+02		5,9E+02	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	0,90		1,00		0,81		0,96	
v_{04} / u_Σ [-]	1,12		0,80		0,99		1,06	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	12,00		6,00		8,33		8,33	
c_i [-] (Reduction factor for slender sheeted walls)	1,00		1,00		1,00		1,00	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	343,00		152,00		199,00		254,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1{,}2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	14817,60		1094,40		3980,00		5080,00	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	0,56		1,62		0,65		0,72	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	0,70		0,73		0,78		1,10	
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	10976,00		810,67		2948,15		3762,96	
F_{04} / $F_{v,Rd\,(SLS)}$	0,76		2,19		0,87		0,97	
	(14) (Källsner, 1984)		(14) (Källsner, 1984)		(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-		(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-	

Reference:	(10.1)		(10.3)		(8.2)		(14.2)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	1		1		2		2	
h_1 [mm] (element height)	2500,00		2500,00		2440,00		2400,00	
b_1 [mm] (width panel)	1250,00		1250,00		910,00		1200,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	2		2		1		1	
t [mm] (sheeting thickness)	12,50		18,00		12,00		12,50	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	1100,00		1100,00		1100,00		1100,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	700,00		700,00		700,00		700,00	
b_2 [mm] (stud & rail cross-sectional width)	40,00		40,00		38,00		45,00	
h_2 [mm] (stud & rail cross-sectional height)	100,00		100,00		89,00		95,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		1		1		1	
s [mm] (fastener spacing along perimeter)	50,0		50,0		100,0		200,0	
K_{ser} [N/mm] (fastener slip-modulus)	5,6E+02		5,6E+02		4,4E+02		6,7E+02	
K_{hd} [N/mm] (hold-down stiffness)	2,9E+04		3,4E+04		- - -		9262,50	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	9,1E+03		9,1E+03		- - -		9,3E+03	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,6	19%	2,2	20%	3,0	77%	2,5	66%
u_{sh} [mm] (shear of the sheeting)	0,9	10%	0,8	7%	0,5	13%	0,4	10%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	1,7	20%	2,3	21%	0,4	10%	0,2	5%
u_{hd} [mm] (hold-down anchorage)	1,0	12%	1,2	11%	0,0	0%	0,4	10%
u_c [mm] (compression perpendicular to grain)	3,3	39%	4,5	41%	0,0	0%	0,4	10%
u_Σ [mm] (total deformation element at load level F_{04})	8,5	100%	10,9	100%	3,9	100%	3,8	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	8,9E+02		9,3E+02		8,4E+02		8,8E+02	
Step 6): Comparison								
R_{test} / R_Σ [-]	0,91		1,01		0,79		0,78	
v_{04} / u_Σ [-]	1,10		1,00		1,22		0,94	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,6	19%	2,2	20%	3,0	77%	2,5	66%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	2,0	24%	1,9	17%	1,9	49%	0,9	24%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	1,7	20%	2,3	21%	0,4	10%	0,2	5%
u_{hd} [mm] (hold-down anchorage)	1,0	12%	1,2	11%	0,0	0%	0,4	10%
u_c [mm] (compression perpendicular to grain)	3,3	39%	4,5	41%	0,0	0%	0,4	10%
u_Σ [mm] (total deformation element at load level F_{04})	9,7	100%	12,1	100%	5,3	100%	4,3	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	7,8E+02		8,4E+02		6,1E+02		7,8E+02	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	1,03		1,11		1,07		0,89	
v_{04} / u_Σ [-]	0,97		0,91		0,90		0,83	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	25,00		25,00		9,10		6,00	
c_i [-] (Reduction factor for slender sheeted walls)	1,00		1,00		0,75		1,00	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	199,00		254,00		179,00		152,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1,2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	11940,00		15240,00		2916,00		2188,80	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	0,63		0,67		1,11		1,52	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	1,57		1,83		0,79		0,59	
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	8844,44		11288,89		2160,00		1621,33	
F_{04} / $F_{v,Rd\,(SLS)}$	0,86		0,90		1,50		2,05	
	(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-		(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-		(8) (Yasamura, 1991)		(14) (Källsner, 1984)	

Reference:	(2.1)		(14.9)		(14.3)		(2.2)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	3		3		3		3	
h_1 [mm] (element height)	2400,00		2400,00		2400,00		2400,00	
b_1 [mm] (width panel)	1200,00		1200,00		1200,00		1200,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		1		2	
t [mm] (sheeting thickness)	13,00		12,50		12,50		13,00	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	1100,00		1100,00		1100,00		1100,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	700,00		700,00		700,00		700,00	
b_2 [mm] (stud & rail cross-sectional width)	45,00		45,00		45,00		45,00	
h_2 [mm] (stud & rail cross-sectional height)	90,00		95,00		95,00		90,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		1		1		1	
s [mm] (fastener spacing along perimeter)	100,0		150,0		200,0		100,0	
K_{ser} [N/mm] (fastener slip-modulus)	6,7E+02		4,4E+02		6,7E+02		6,7E+02	
K_{hd} [N/mm] (hold-down stiffness)	1,6E+03		9262,50		9262,50		1,6E+03	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	8,8E+03		9,3E+03		9,3E+03		8,8E+03	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	2,6	37%	2,7	74%	2,6	72%	2,3	25%
u_{sh} [mm] (shear of the sheeting)	0,8	11%	0,4	10%	0,4	11%	0,7	7%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,3	4%	0,1	3%	0,1	3%	0,4	5%
u_{hd} [mm] (hold-down anchorage)	2,9	41%	0,2	6%	0,3	7%	5,0	53%
u_c [mm] (compression perpendicular to grain)	0,5	8%	0,2	6%	0,3	7%	0,9	10%
u_Σ [mm] (total deformation element at load level F_{04})	7,0	100%	3,6	100%	3,7	100%	9,4	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	1,5E+03		1,3E+03		1440,66		2,0E+03	
Step 6): Comparison								
R_{test} / R_Σ [-]	0,42		0,96		0,94		0,70	
v_{04} / u_Σ [-]	1,73		0,80		0,83		1,15	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	2,6	37%	2,7	74%	2,6	72%	2,3	25%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	1,8	26%	0,9	24%	1,0	26%	1,6	17%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,3	4%	0,1	3%	0,1	3%	0,4	5%
u_{hd} [mm] (hold-down anchorage)	2,9	41%	0,2	6%	0,3	7%	5,0	53%
u_c [mm] (compression perpendicular to grain)	0,5	8%	0,2	6%	0,3	7%	0,9	10%
u_Σ [mm] (total deformation element at load level F_{04})	8,1	100%	4,1	100%	4,2	100%	10,3	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	1,3E+03		1,2E+03		1252,15		1,8E+03	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	0,49		1,10		1,08		0,77	
v_{04} / u_Σ [-]	1,50		0,70		0,72		1,05	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	12,00		8,00		6,00		12,00	
c_i [-] (Reduction factor for slender sheeted walls)	1,00		1,00		1,00		1,00	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	211,00		198,00		152,00		211,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1{,}2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	9115,20		5702,40		3283,20		18230,40	
$F_{04} / F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	1,16		0,83		1,60		1,01	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	2,03		0,48		0,50		1,79	
$F_{v,Rd\,(SLS)} = F_{v,Rd} / 1{,}35$	6752,00		4224,00		2432,00		13504,00	
$F_{04} / F_{v,Rd\,(SLS)}$	1,56		1,13		2,17		1,37	
	(2) (Andreasson, 2000)		(14) (Källsner, 1984)		(14) (Källsner, 1984)		(2) (Andreasson, 2000)	

Reference:	(14.10)		(4.2)		(13.4)		(12.4)	
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	1		4		1		1	
h_1 [mm] (element height)	2400,00		2440,00		2600,00		2600,00	
b_1 [mm] (width panel)	3600,00		1220,00		1250,00		1250,00	
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1		1		1		1	
t [mm] (sheeting thickness)	12,50		12,70		15,00		15,00	
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	1100,00		1100,00		3800,00		3800,00	
G_{mean} [N/mm^2] (Shear modulus sheet material)	700,00		700,00		1600,00		1600,00	
b_2 [mm] (stud & rail cross-sectional width)	45,00		38,00		45,00		45,00	
h_2 [mm] (stud & rail cross-sectional height)	95,00		89,00		100,00		100,00	
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00		11000,00		11000,00	
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		2		1		1	
s [mm] (fastener spacing along perimeter)	200,0		200,0		75,0		75,0	
K_{ser} [N/mm] (fastener slip-modulus)	6,7E+02		3,2E+02		5,4E+02		6,5E+02	
K_{hd} [N/mm] (hold-down stiffness)	9262,50		1,4E+04		2,0E+04		2,0E+04	
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	9,3E+03		1,2E+04		9,8E+03		9,8E+03	
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,7	59%	5,8	88%	4,8	41%	4,0	37%
u_{sh} [mm] (shear of the sheeting)	0,5	16%	0,4	6%	0,6	5%	0,6	6%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,1	5%	0,1	1%	1,6	14%	1,6	15%
u_{hd} [mm] (hold-down anchorage)	0,3	10%	0,1	2%	1,5	13%	1,5	14%
u_c [mm] (compression perpendicular to grain)	0,3	10%	0,2	2%	3,1	27%	3,1	29%
u_Σ [mm] (total deformation element at load level F_{04})	2,9	100%	6,6	100%	11,7	100%	10,9	100%
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	2,1E+03		1,1E+03		6,0E+02		6,5E+02	
Step 6): Comparison								
R_{test} / R_Σ [-]	1,02		1,87		1,30		1,43	
v_{04} / u_Σ [-]	0,78		0,41		0,77		0,83	
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}								
Effects on panel scale:								
u_f [mm] (fasteners along the perimeter)	1,7	59%	5,8	88%	4,8	41%	4,0	37%
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	0,5	18%	1,0	15%	1,0	9%	1,0	9%
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	0,1	5%	0,1	1%	1,6	14%	1,6	15%
u_{hd} [mm] (hold-down anchorage)	0,3	10%	0,1	2%	1,5	13%	1,5	14%
u_c [mm] (compression perpendicular to grain)	0,3	10%	0,2	2%	3,1	27%	3,1	29%
u_Σ [mm] (total deformation element at load level F_{04})	2,9	100%	7,2	100%	12,1	100%	11,3	100%
Step 5*): Additional calculations stiffness								
R_Σ [-]	2,1E+03		1,0E+03		5,8E+02		6,2E+02	
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	1,04		2,03		1,34		1,48	
v_{04} / u_Σ [-]	0,77		0,38		0,75		0,80	
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	18,00		6,10		16,67		16,67	
c_i [-] (Reduction factor for slender sheeted walls)	3,00		1,00		0,96		0,96	
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	152,00		201,00		386,00		422,00	
$F_{v,Rd} = (F_{f,Rd} \cdot 1{,}2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	9849,60		5885,28		7423,08		8115,38	
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	0,62		1,27		0,95		0,87	
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00		6,00	
v_{04} / Deformation limit	0,37		0,45		1,51		1,51	
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	7296,00		4359,47		5498,58		6011,40	
F_{04} / $F_{v,Rd\,(SLS)}$	0,83		1,71		1,28		1,17	
	(14) (Källsner, 1984)		(4) (Ni, Shim, Karacabeyli, 2010)		(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)		(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 04-10-2001)	

Reference:	(13.2)		(12.2)					
Step 3): Parameters of the wall								
n_{panels} [-] (number of panels in a wall element)	1		1					
h_1 [mm] (element height)	2600,00		2600,00					
b_1 [mm] (width panel)	1250,00		1250,00					
n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	2		2					
t [mm] (sheeting thickness)	15,00		15,00					
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	3800,00		3800,00					
G_{mean} [N/mm^2] (Shear modulus sheet material)	1600,00		1600,00					
b_2 [mm] (stud & rail cross-sectional width)	45,00		45,00					
h_2 [mm] (stud & rail cross-sectional height)	100,00		100,00					
E_2 [N/mm^2] (Young's modulus timber)	11000,00		11000,00					
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1		1					
s [mm] (fastener spacing along perimeter)	75,0		75,0					
K_{ser} [N/mm] (fastener slip-modulus)	5,4E+02		6,5E+02					
K_{hd} [N/mm] (hold-down stiffness)	2,3E+04		2,3E+04					
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	9,8E+03		9,8E+03					
Step 4): Calculated contributions to the deformation of the shear-wall with load F_{04}					**Mean values for all tests**			
Effects on panel scale:					mean	mean deviation	%	
u_f [mm] (fasteners along the perimeter)	4,1	28%	3,4	24%	3,3	1,1	48%	u_f [mm] (Fasteners along the perimeter)
u_{sh} [mm] (shear of the sheeting)	0,5	3%	0,5	4%	1,0	0,6	12%	u_{sh} [mm] (Shear of the sheeting)
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	2,7	18%	2,7	19%	0,7	0,6	8%	u_{str} [mm] (strain in leading & trailing stud)
u_{hd} [mm] (hold-down anchorage)	2,3	15%	2,2	16%	1,1	0,8	13%	u_{hd} [mm] (Hold-down anchorage)
u_c [mm] (compression perpendicular to grain)	5,4	36%	5,3	37%	1,3	1,2	15%	u_c [mm] (Compression perpendicular to grain)
u_Σ [mm] (total deformation element at load level F_{04})	15,0	100%	14,1	100%	7,42	2,92	100%	
Step 5): Calculated stiffness of the shear-wall								
R_Σ [-]	8,0E+02		8,4E+02		1,2E+03			
Step 6): Comparison								
R_{test} / R_Σ [-]	1,77		1,56		1,06	0,35		
v_{04} / u_Σ [-]	0,67		0,75		0,92	0,22		
Step 4*): Calculated contributions to the deformation of the shear-wall with load F_{04}					**Mean values for all tests**			
Effects on panel scale:					mean	mean deviation	%	
u_f [mm] (fasteners along the perimeter)	4,1	28%	3,4	24%	3,3	1,1	49%	u_f [mm] (Fasteners along the perimeter)
$u_{t\&c}$ [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)	0,9	6%	0,9	6%	1,1	0,4	17%	ut&c [mm] (sheeting deformation due to elongation and shortening of the tension and compression diagonal)
Effects on element scale:								
u_{str} [mm] (strain in leading & trailing stud)	2,7	18%	2,7	19%	0,7	0,6	8%	u_{str} [mm] (strain in leading & trailing stud)
u_{hd} [mm] (hold-down anchorage)	2,3	15%	2,2	16%	1,1	0,8	13%	u_{hd} [mm] (Hold-down anchorage)
u_c [mm] (compression perpendicular to grain)	5,4	36%	5,3	37%	1,3	1,2	15%	u_c [mm] (Compression perpendicular to grain)
u_Σ [mm] (total deformation element at load level F_{04})	15,4	100%	14,4	100%	7,56	2,98	100%	
Step 5*): Additional calculations stiffness								
R_Σ [-]	7,8E+02		8,2E+02		1,2E+03			
Step 6*): Additional calculations comparison								
R_{test} / R_Σ [-]	1,81		1,59		1,09	0,33		
v_{04} / u_Σ [-]	0,65		0,74		0,89	0,20		
Step 7): Design strenght of the timber-frame shear-wall $F_{v,Rd}$ according to NEN-EN 1995-1-1								
b/s [-] (Number of fasteners in horizontal framing elements)	16,67		16,67					
c_i [-] (Reduction factor for slender sheeted walls)	0,96		0,96					
$F_{f,Rd}$ [N] (framing lumber: C24) (γ_M = 1,3 & k_{mod} = 0,9)	386,00		422,00		330,50			
$F_{v,Rd} = (F_{f,Rd} \cdot 1{,}2) \cdot c_i \cdot b/s$ [N] (Design strength of the shear-wall)	14846,15		16230,77		8665,05			
F_{04} / $F_{v,Rd}$ [-] (Ratio 40% F_{max} / design strength)	0,81		0,73		1,04	0,26		
Deformation limit 1/500 * h (h=3000)	6,00		6,00		6,00	0,00		
v_{04} / Deformation limit	1,67		1,77		1,13	0,45		
$F_{v,Rd\,(SLS)} = F_{v,Rd}$ /1,35	10997,15		12022,79		6418,56	2941,07		
F_{04} / $F_{v,Rd\,(SLS)}$	1,10		0,99		1,40	0,35		
	(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)		(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 04-10-2001)					

Reference list belonging to the racking stiffness calculation

(1) (Yasumura & Kawai, 1997)
(2) (Andreasson, 2000)
(3) (Salenikovich, 2000)
(4) (Ni et al., 2010)
(5) (NAHB Research Center Inc., 2005)
(6) (Yasumura & Karacabeyli, 2007)
(8) (Yasumura, 1991)
(10) (LHT Labor für Holztechnik - Fachbereich Bauingenieurswesen Hildesheim, 29-08-2002)
(12) (VHT Versuchsanstalt fur holz- und trockenbau Darmstadt, 04-10-2001)
(13) (VHT Versuchsanstalt fur holz- und trockenbau Darmstad, 29-05-2002)
(14) (Källsner, 1984)
(16) (Dolan & Toothman, 2003)

The literature can be found in the sources displayed on page 105.

Appendix VIII Fastener slip-modulus in racking tests

This appendix contains the calculation of the values for K_{ser} that are used in the analytical calculation of the shear-wall stiffness. For gypsum-paper board, and gypsum-fibre board, use was made of the test-based slip-modulus shown in paragraph 2.4. For the wood-based panels the fastener slip-modulus can be calculated using the design rules given in Eurocode 5. This calculation can be found on the next page.

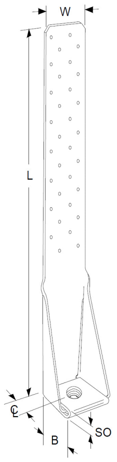
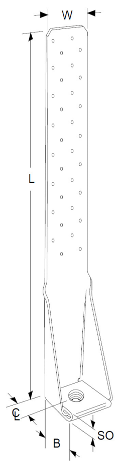
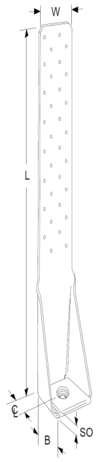
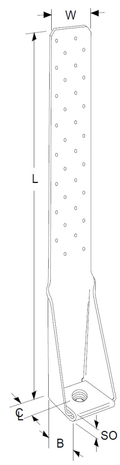
Slip modulus K_{ser} for nails (according to NEN-EN 1995-1-1)												
Reference to wall test:	(3.1)	(3.2)	(1.2)	(5.1)	(3.3)	(5.2)	(3.4)	(6.1)	(8.1)	(14.21)	(4.1)	(14.15)
Material:	OSB	OSB	OSB	OSB	OSB	OSB	OSB	Plywood	Plywood	Plywood	Plywood	Particleboard
Material thickness t: [mm]	11	11	9,5	11	11	11	11	9	9,5	8	9,5	12
Aplied fastener: N = nail, Scr = screw, St = staple	N Ø3,3x65 s = 152	N Ø3,3x65 s = 152	N Ø2,87x50 s = 100	N Ø3,3x63,5 s = 165,1	N Ø3,3x65 s = 152	N Ø3,3x63,5 s = 114,3	N Ø3,3x65 s = 152	N Ø2,87x50,8 s = 100	N Ø2,87x50 s=100 mm	N Ø2,13x50,1 s = 150 mm	N Ø2,5x63,5 (spiral) s = 150	N Ø2,13x50,1 s = 100 mm
$\rho_{m,1}$ [kg/m^3] strength class C24	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00	420,00
$\rho_{m,2}$ [kg/m^3]	550,00	550,00	550,00	550,00	550,00	550,00	550,00	460,00	460,00	460,00	460,00	650,00
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)	480,62	480,62	480,62	480,62	480,62	480,62	480,62	439,55	439,55	439,55	439,55	522,49
d [mm]	3,30	3,30	2,87	3,30	3,30	3,30	3,30	2,87	2,87	2,13	2,50	2,13
K_{ser} [N/mm] = $\rho_m^{1,5} d^{0,8}/30$ (C24)	912,85	912,85	816,38	912,85	912,85	912,85	912,85	713,99	713,99	562,45	639,35	728,96
$K_u = 2/3 \cdot K_{ser}$ [N/mm] (C24) =	608,57	608,57	544,25	608,57	608,57	608,57	608,57	475,99	475,99	374,97	426,23	485,97

Slip modulus K_{ser} for screws (according to NEN-EN 1995-1-1)	
Reference to wall test:	(2.3)
Material:	Plywood
Material thickness t: [mm]	9
Aplied fastener: N = nail, Scr = screw, St = staple	Scr Ø4,2x35 s = 200
$\rho_{m,1}$ [kg/m^3] strength class C24	420,000
$\rho_{m,2}$ [kg/m^3]	460,000
ρ_m [kg/m^3] = $\sqrt{\rho_{m,1} \cdot \rho_{m,2}}$ (C24)	439,545
d [mm] = 0,66 · d	2,77
K_{ser} [N/mm] = $\rho_m^{1,5} d/23$ (C24)	1110,63
$K_u = 2/3 \cdot K_{ser}$ [N/mm] (C24) =	740,42

Slip modulus k_s for gypsum paper-board (according to NEN-EN 26891)									
Reference to fastener test:	(1.1)	(7.2)	(7.4)	(8.1)	(15.3)	(15.2)	(15.4)	(15.5)	mean of (1.1) & (7.2)
Sheeting thickness t: [mm]	12,00	12,50	12,50	13,00	12,50	12,50	12,50	12,50	
Number of fasteners in test:	4	1	1	1	1	1	1	1	
Aplied fastener: N = nail, Scr = screw, St = staple	N Ø2,34x40	N Ø2,2x45	Scr Ø3,35x45	Scr Ø3,9x41	N Ø2,45 x 34,5	N Ø2,45 x 34,5	Scr Ø3,00 x 38	Scr Ø3,00 x 38	
Remark:	Framing timber member: SPF 38 x 89 mm Mean of 6 specimen (symmetric)	Framing timber member: 45 x 45 mm Mean of 10 specimen Gyproc gypsum paper board (// to edge lined with paper) (asymmetric)	Framing timber member: 45 x 45 mm Mean of 10 specimen Gyproc gypsum paper board (// to edge lined with paper) (asymmetric)	Framing timber member: 45 x 90 mm Mean of 2 specimen Danogips Normal DN13 / GKB/1200 gypsum paper board (// to edge lined with paper) (asymmetric)	Framing timber member: 45x95 mm Mean of 6 specimen Gyproc Normal GN (// to edge lined with paper) (asymmetric)	Framing timber member: 45x95 mm Mean of 6 specimen Gyproc Normal GN (// to sawn edge) (asymmetric)	Framing timber member: 45x95 mm Mean of 6 specimen Gyproc Normal GN (// to edge lined with paper) (asymmetric)	Framing timber member: 45x95 mm Mean of 6 specimen Gyproc Normal GN (// to sawn edge) (asymmetric)	This value is used for the analytical calculation of shear-wall stiffness tests (8.2) and (14.9). In these tests, either a rather short, or rather slender nail-fastener is aplied; and this is also the case in tests (1.1) and (7.2) <<---
k_s [N/mm]	3,2E+02	5,6E+02	2,5E+03	3,6E+02	6,4E+02	5,5E+02	2,0E+03	6,7E+02	4,4E+02

Slip modulus k_s for gypsum fibre-board (according to NEN-EN 26891)								
Reference to fastener test:	(3.1)	(12.1)	(12.2)	(13.1)	(13.2)	(14)	2 x (12.2)	mean of (3.1), (12.1), (13.1) & (14)
Sheeting thickness t: [mm]	15,00	15,00	15,00	15,00	15,00	12,50		
Number of fasteners in test:	4	4	4	4	4	12		
Aplied fastener: N = nail, Scr = screw, St = staple	Staple Ø1,34x1,62x47	St Ø1,53 x 50	N Ø2,5 x 65 (Rillenagel)	St Ø1,8 x 65	N Ø2,5 x 55	St Ø1,4 x 1,6 x 60		
Remark:	Framing timber member: SPF 40 x 100 mm Mean of 3 specimen (symmetric)	Framing timber member: 100 x 100 mm Mean of 5 specimen Rigidur RH gypsum fibre board (symmetric)	Framing timber member: 100 x 100 mm Mean of 5 specimen Rigidur RH gypsum fibre board (symmetric)	Framing timber member: 100 x 100 mm Mean of 5 specimen Rigidur RH gypsum fibre board (symmetric)	Framing timber member: 100 x 100 mm Mean of 5 specimen Rigidur RH gypsum fibre board (symmetric)	Framing timber member: 160 x 60 mm FERMACELL gypsum fibre board (symmetric)	In shear wall-tests (13.4) and (13.2), Ring-nails are aplied. Therefore, ringnail test (12.2) is chosen, and multiplied with 2 because the actual mean stiffness in a shear-wall will probably be higher than the mean behaviour of four fasteners in a test.	Mean value is taken of the staple-fastener tests. Because (3.1) provides a rather high value, therefore, the mean value of 5,1E+02 is lowered to 4,0E+02
k_s [N/mm]	1,1E+03	4,6E+02	2,7E+02	6,5E+02	1,4E+03	4,0E+02	5,4E+02	6,5E+02

Appendix IX Hold-down stiffness in racking tests

Hold-down stiffness in racking tests					
Type of hold-down:	Simpson Strong-Tie HTT22 hold-down	Simpson Strong-Tie HTT22 hold-down		Simpson Strong-Tie HTT22 hold-down	Simpson Strong-Tie HTT22 hold-down
Fastener properties:	32 x nail Ø3,8 x 82,6 &	32 x nail Ø3,8 x 82,6		32 x nail Ø3,33x83 &	32 x nail Ø3,8 x 82,6
Reference to test:	(3.1)	(3.2)	(1.2)	(5.1)	(3.3)
			Mean of 2 static tests (gecorrigeerd voor: sliding & tilting)		

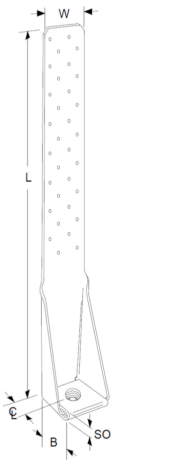
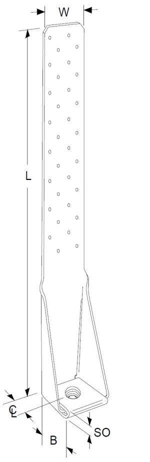
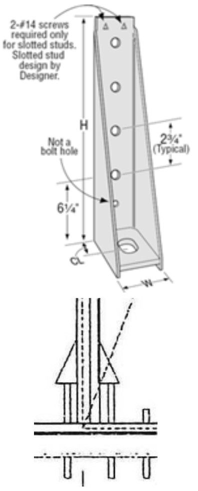
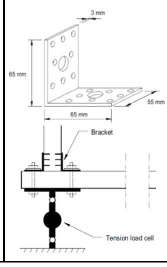
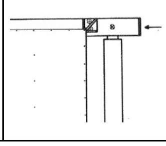
Slip-modulus of the fastener (K_{ser}) according to NEN-EN 1995-1-1					
$\rho_{m,1}$ [kg/m^3] strength class C24	420	420		420	420
d [mm]	3,80	3,80		3,33	3,80
K_{ser} [kN/mm] = $\rho_m^{1,5} \cdot d^{0,8}/30$ (C24)	0,83	0,83		0,75	0,83
$2 \cdot K_{ser}$ [kN/mm]	1,67	1,67		1,50	1,67
n = number of fasteners [-]	32	32		32	32
$n \cdot K_{ser}$ [kN/mm]	53,43	53,43		48,07	53,43

Calculation of the cross-sectional stiffness of the steel and timber elements:					
E_{steel} [N/mm^2]	210000	210000		210000	210000
A_{steel} [mm^2]	193,04 =(63,5 x 3,04)	193,04 =(63,5 x 3,04)		193,04 =(63,5 x 3,04	193,04 =(63,5 x 3,04
l_1 (steel) [mm]	200,03	200,03		200,03	200,03
$E \cdot A_{steel}/l_1$ [kN/mm]	202,67	202,67		202,67	202,67
$A_{steel;foot}$ [mm^2]	347,47 =(63,5+50,8) x 3,04	347,47 = (63,5+50,8) x 3,04		347,47 = (63,5+50,8) x 3,04	347,47 = (63,5+50,8) x 3,04
l_{foot} [mm]	147,64	147,64		147,64	147,64
$E \cdot A_{foot}/l_{foot}$ [kN/mm]	494,24	494,24		494,24	494,24
E_{timber} [N/mm^2] (C24)	11000	11000		11000	11000
A_{timber} [mm^2]	6764,00 =(76 x 89,00)	6764,00 =(76 x 89,00)		6764,00 =(76 x 89,00)	6764,00 =(76 x 89,00)
l_2 (timber) [mm]	200,03	200,03		200,03	200,03
$E \cdot A_{timber}/l_2$ [kN/mm]	371,97	371,97		371,97	371,97

Calulation of the equivalent stiffness of the hold-down:					
K_{hd} [kN/mm]	35,26	35,26		32,84	35,26

Load level on the wall:					
F_{04} [kN]:	1,75	4,11		11,86	9,54
$F_{v,Rd}$ [kN]:	1,28	5,12		9,54	10,23
Load level on the hold-down:					
F_{HD} [kN] = $F_{04} \cdot (h/b)$	7,00	8,23		11,91	9,54

Calculation of the reduced stiffness due to hole-clearance:					
d_{hole} [mm]	4,70	4,70		4,70	4,70
$d_{fastener}$ [mm]	3,80	3,80		3,33	3,80
$K_{hd;reduced}$ [N/mm] (F_{04})	10794,65	12037,53		11368,39	13237,26

Simpson Strong-Tie HTT22 hold-down	Simpson Strong-Tie HTT22 hold-down	Simpson Strong-Tie HDB 15 hold-down (2)				
32 x nail Ø3,33x83 & bolt	32 x nail Ø3,8 x 82,6	4 bolts Ø19 & bolt Ø25				
(5.2)	(3.4)	(6.1)	(8.1)	(2.3)	(14.21)	(4.1)
			Mean of 2 static tests (gecorrigeerd voor: sliding & tilting)	Partially anchored with steel angle bracket. Stiffness of the aplied bracket is determined with a test seperately. K_{hd} = 1637,63 N/mm 	Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	No specific information about the aplied hold-down is present in the report. Average value of all hold-down devices in the present calculation sheet is used.

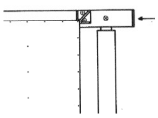
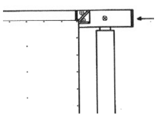
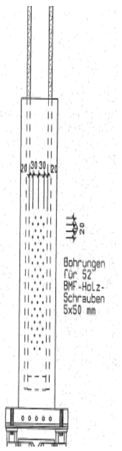
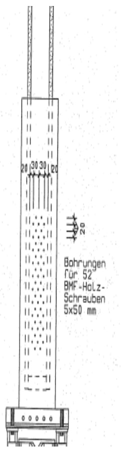
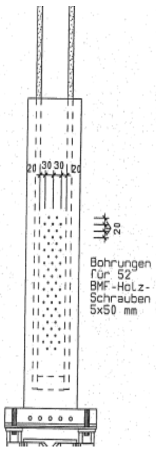
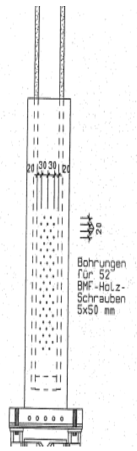
420	420	420				
3,33	3,80	19,00				
0,75	0,83	7,11				
1,50	1,67	14,22				
32	32	4				
48,07	53,43	56,88				

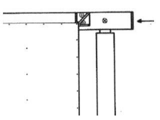
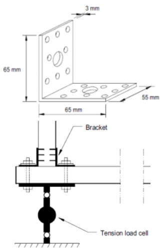
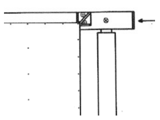
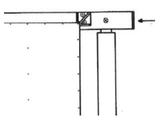
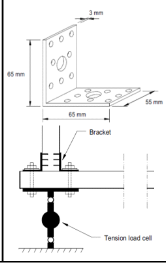
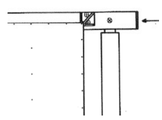
210000	210000	210000				
193,04 =(63,5 x3,04)	193,04 =(63,5 x3,04)					
200,03	200,03					
202,67	202,67					
347,47 =(63,5+50,8) x3,04	347,47 = (63,5+50,8) x3,04	627,19 =(61,91+69,85) x4,67				
147,64	147,64	368,30				
494,24	494,24	357,62				
11000	11000	11000				
6764,00 =(76 x 89,00)	6764,00 =(76 x 89,00)	6764,00 =(76 x 89,00)				
200,03	200,03	368,30				
371,97	371,97	202,02				

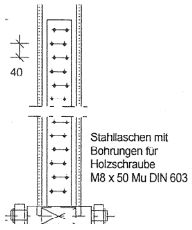
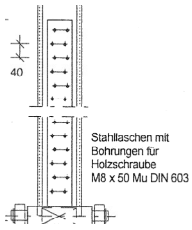
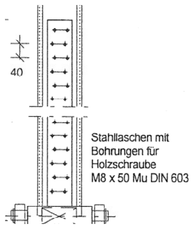
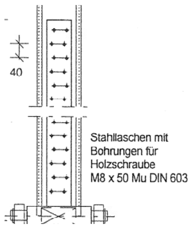
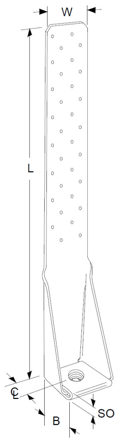
32,84	35,26	39,49				

15,85	13,52	12,21				
13,78	15,35	6,47				
15,92	9,01	16,37				

4,70	4,70	19,50				
3,33	3,80	19,00				
13607,91	12771,94	24634,51		1637,63	9262,50	14048,87

		Hold-down (BMF?)	Hold-down (BMF?)	Hold-down (BMF?)	Hold-down (BMF?)	
		52 x screw Ø5,0x50	52 x screw Ø5,0x50	52 x screw Ø5,0x50	52 x screw Ø5,0x50	
(14.15)	(14.1)	(10.2)	(10.4)	(10.1)	(10.3)	(8.2)
Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 					Mean of 2 static tests (gecorrigeerd voor: sliding & tilting)
		420	420	420	420	
		3,30 d_{eff} = 0,66	3,30 d_{eff} = 0,66	3,30 $d_{eff} = 0{,}66 \cdot d$	3,30 $d_{eff} = 0{,}66 \cdot d$	
		1,23	1,23	1,23	1,23	
		2,47	2,47	2,47	2,47	
		52	52	52	52	
		128,44	128,44	128,44	128,44	
		210000	210000	210000	210000	
		300,00 assumed	300,00 assumed	300,00 assumed	300,00 assumed	
		320,00 assumed	320,00 assumed	320,00 assumed	320,00 assumed	
		196,88	196,88	196,88	196,88	
		11000	11000	11000	11000	
		4000,00 = (40 x 100)	4000,00 = (40 x 100)	4000,00 = (40 x 100)	4000,00 = (40 x 100)	
		200,00	200,00	200,00	200,00	
		220,00	220,00	220,00	220,00	
		57,44	57,44	57,44	57,44	
		2,57	3,64	7,56	10,16	
		3,98	5,08	11,94	15,24	
		5,15	7,28	15,13	20,32	
		5,50	5,50	5,50	5,50	
		5,00	5,00	5,00	5,00	
9262,50	9262,50	15158,18	19327,96	29465,58	33651,25	

(14.2)	(2.1)	(14.9)	(14.3)	(2.2)	(14.10)	(4.2)
Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	Partially anchored with steel angle bracket. Stiffness of the aplied bracket is determined with a test seperately. K_{hd} = 1637,63 N/mm  	Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	Partially anchored with steel angle bracket. Stiffness of the aplied bracket is determined with a test seperately. K_{hd} = 1637,63 N/mm  	Uplift prevented by the test-rig. Assumed is that compression perpendicular to grain takes place below the compressed stud, and above the 'tensile' stud, in bottom-rail (sill) and top-rail. K_{hd} = 9262,50N/mm 	No specific information about the aplied hold-down is present in the report. Average value of all hold-down devices in the present calculation sheet is used.
9262,50	1637,63	9262,50	9262,50	1637,63	9262,50	14048,87

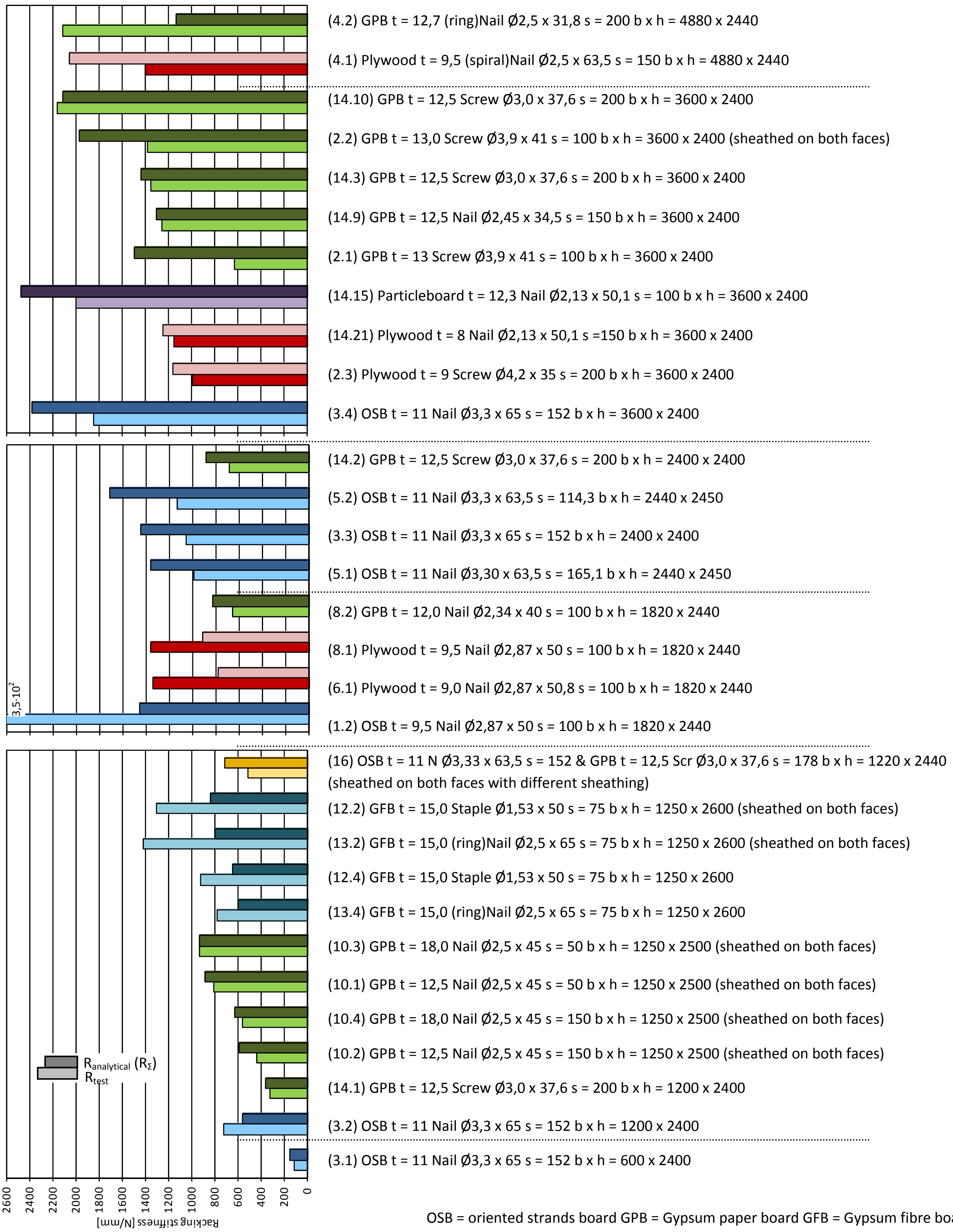
Hold-down (BMF?)	Hold-down (BMF?)	Hold-down (BMF?)	Hold-down (BMF?)	Simpson Strong-Tie HTT22 hold-down
54 x screw Ø8,0x50	54 x screw Ø8,0x50	54 x screw Ø8,0x50	54 x screw Ø8,0x50	32 x nail Ø3,8x82,6
(13.4)	(12.4)	(13.2)	(12.2)	(16)
				
420	420	420	420	420
5,28 $d_{eff} = 0,66 \cdot d$	5,28 $d_{eff} = 0,66 \cdot d$	5,28 $d_{eff} = 0,66 \cdot d$	5,28 $d_{eff} = 0,66 \cdot d$	3,80
1,98	1,98	1,98	1,98	0,83
3,95	3,95	3,95	3,95	1,67
54	54	54	54	32
213,40	213,40	213,40	213,40	53,43
210000	210000	210000	210000	210000
200,00 assumed	200,00 assumed	200,00 assumed	200,00 assumed	193,04 =(63,5 x 3,04)
657,50 assumed	657,50 assumed	657,50 assumed	657,50 assumed	200,03
63,88	63,88	63,88	63,88	202,67
				347,47 = (63,5+50,8) x 3,04
				147,64
				494,24
11000	11000	11000	11000	11000
4500,00 =(45 x 100)	4500,00 =(45 x 100)	4500,00 =(45 x 100)	4500,00 =(45 x 100)	6764,00 =(76 x 89,00)
657,50	657,50	657,50	657,50	200,03
75,29	75,29	75,29	75,29	371,97
29,74	29,74	29,74	29,74	35,26
7,06	7,05	12,07	11,86	5,34
7,42	8,12	14,85	16,23	
14,69	14,67	25,10	24,67	10,68
8,50	8,50	8,50	8,50	4,70
8,00	8,00	8,00	8,00	3,80
19747,90	19738,50	22944,67	22853,08	14181,01

References to literature:

(1) (Rotho Blaas GmbH Rothofixing, 07-04-2010)
(2) (NAHB Research Center, 2003)

Appendix X Comparison of test-based and calculated racking stiffness

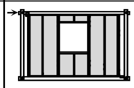
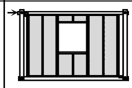
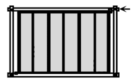
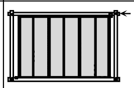
In the bar chart below the test-based racking stiffness (R_{test}), and calculated racking stiffness (R_{Σ}), of 31 shear-wall elements are shown.



OSB = oriented strands board GPB = Gypsum paper board GFB = Gypsum fibre board

Appendix XI Test-based racking stiffness perforated shear-walls

Below the load-displacement data of the perforated shear-wall elements in test (14.8) and test (14.15) are shown. The test-based stiffness of the perforated shear-wall elements is calculated with use of NEN-EN 594. Also the calculation of racking stiffness with use of the panel-area-ratio method is shown.

Load-displacement data perforated shear-wall test				
Reference:	(14.8)	(14.16)	(14.3)	(14.15)
Material:	GKB	Particleboard	GKB	Particleboard
Material thickness t: [mm]	12,5	12,3	12,5	12,3
Fastener properties:	Scr Ø3,0x37,6 s = 200 mm	N Ø2,13x50,1 s = 100 mm	Scr Ø3,0x37,6 s = 200 mm	N Ø2,13x50,1 s = 100 mm
Diaphragm dimension b x h:	3600x2400	3600x2400	3600x2400	3600x2400
Anchorage condition:	Uplift prevented met testopstelling	Uplift prevented met testopstelling	Uplift prevented met testopstelling	Uplift prevented met testopstelling
Perforation:	gap in center panel, 1200 x 1200 mm	gap in center panel, 1200 x 1200 mm	none	none
Figure:				
Remark:	Framing timber: 45 x 95 mm Sheeted on one face	Framing timber: 45 x 95 mm Sheeted on one face	Framing timber: 45 x 95 mm Sheeted on one face	Framing timber: 45 x 95 mm Sheeted on one face

D = displacement [mm]	D:	L:	D:	L:	D:	L:	D:	L:
L = load [N]	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00	0,0E+00
	4,1E-01	9,9E+02	1,5E+00	3,4E+03	1,1E+00	2,6E+03	1,0E+00	2,1E+03
	1,1E+00	2,0E+03	2,5E+00	4,3E+03	3,0E+00	5,3E+03	2,1E+00	4,2E+03
	3,6E+00	4,0E+03	4,5E+00	6,8E+03	6,3E+00	7,9E+03	3,4E+00	6,8E+03
	7,8E+00	6,0E+03	5,0E+00	7,4E+03	1,2E+01	1,1E+04	4,2E+00	8,3E+03
	1,3E+01	7,9E+03	7,5E+00	9,4E+03	2,0E+01	1,2E+04	4,4E+00	8,8E+03
	1,7E+01	8,9E+03	1,0E+01	1,1E+04	2,8E+01	1,3E+04	5,0E+00	9,4E+03
	2,9E+01	9,9E+03	1,5E+01	1,4E+04	3,0E+01	1,3E+04	7,3E+00	1,3E+04
	4,2E+01	8,9E+03	2,0E+01	1,6E+04	3,3E+01	1,3E+04	7,5E+00	1,3E+04
	4,6E+01	7,9E+03	2,8E+01	1,7E+04	4,0E+01	1,3E+04	1,0E+01	1,5E+04
	5,0E+01	6,9E+03	3,0E+01	1,7E+04	5,1E+01	1,1E+04	1,2E+01	1,7E+04
			3,3E+01	1,7E+04			1,5E+01	1,8E+04
			3,5E+01	1,7E+04			2,0E+01	1,9E+04
			4,0E+01	1,7E+04			2,5E+01	2,0E+04
			5,0E+01	1,6E+04			3,0E+01	2,1E+04
							3,5E+01	2,1E+04
							4,0E+01	2,0E+04
							5,0E+01	1,8E+04

Determination of the test-based shear-wall stiffness according to NEN-EN 594				
F_{max} [N] (Ultimate load in the test)	9,9E+03	1,7E+04	1,3E+04	2,1E+04
F_{04} [N] (40 % of F_{max})	4,0E+03	6,8E+03	5,3E+03	8,3E+03
F_{02} [N] (20% of F_{max})	2,0E+03	3,4E+03	2,6E+03	4,2E+03
v_{04} [mm] (Deformation at 40% F_{max} load-level)	3,6E+00	4,5E+00	3,0E+00	4,2E+00
v_{02} [mm] (Deformation at 20% F_{max} load-level)	1,1E+00	1,5E+00	1,1E+00	2,1E+00
R_{test} [N/mm] (Based on F_{04} , F_{02})	8,2E+02	1,1E+03	1,4E+03	2,0E+03

Parameters of the wall				
n_{panels} [-] (number of panels in a wall element)	3	3		
h_1 [mm] (element height)	2400,00	2400,00		
b_1 [mm] (width panel)	1200,00	1200,00		
n_{fullh} [-] (number of full height panels in a wall element)	2	2		
h_3 [mm] (height opening)	1200,00	1200,00		
b_3 [mm] (width opening)	1200,00	1200,00		

Determination of the shear-wall stiffness with use of the panel-area-ratio method				
$R_{analytical}$ [N/mm] (analytical value for tests (14.3), (14.10), & (14.15))	1,4E+03	2,2E+03		
α	0,17	0,17		
β	0,67	0,67		
r	0,80	0,80		
$R'_{panel\text{-}area\text{-}ratio}$ [N/mm]	8,2E+02	1,3E+03		
R_{test} / $R'_{panel\text{-}area\text{-}ratio}$ [-]	1,00	0,89		

n_{faces} [-] (1 = single side 2 = both sides (faces) sheeted)	1	1		
t [mm] (sheeting thickness)	12,5	12,3		
$E_{0/90,mean}$ [N/mm^2] (Young's modulus sheet material mean of 2 orthogonal directions)	1100	2000		
G_{mean} [N/mm^2] (Shear modulus sheet material)	700	960		
b_2 [mm] (stud & rail cross-sectional width)	45	45		
h_2 [mm] (stud & rail cross-sectional height)	95	95		
E_2 [N/mm^2] (Young's modulus timber)	11000	11000		
n_{studs} [-] (1 = single edge stud 2 = double edge studs)	1	1		
s [mm] (fastener spacing along perimeter)	200	100		
K_{ser} [N/mm] (fastener slip-modulus)	6,7E+02	728,96		
K_{hd} [N/mm] (hold-down stiffness)	9262,50	9262,50		
$K_{c,90}$ [N/mm] (compression perpendicular to grain)	9,3E+03	9,3E+03		

Appendix XII Analysis of perforated shear-walls

In this appendix the calculations are shown which are made to determine the stiffness of the diagonal elements used in the modelling of the perforated shear-walls as explained in chapter 6. As explained, two different approached can be followed. In this appendix the calculation of diagonal stiffness used in the models is shown for both approaches. In the first paragraph the calculations for the multi-panel models are shown in which the shear-wall is divided in a number of non-perforated parts. For each part a diagonal stiffness is determined using the analytical method explained in paragraph 4.8. In the second paragraph of this appendix the calculations for the equivalent-brace models are shown. In the equivalent-brace approach, the racking stiffness is calculated for the shear-wall as a whole, assuming no perforation. This racking stiffness is represented by an equivalent system of diagonals. The geometry of the perforation is made by removing specific diagonals. More explanation is given in the figures. The general properties of the shear-walls are given below.

b_2 [mm]	=	45	=	thickness of the timber framing elements
E_2 [N/mm^2]	=	11000	=	timber framing elements Young's modulus
h_2 [mm]	=	95	=	width of the timber framing elements
$K_{c,90}$ [N/mm]	=	$9{,}3 \cdot 10^3$	=	stiffness perpendicular to grain of the bottom rail beneath the compressed stud
K_{hd} [N/mm]	=	$9{,}3 \cdot 10^3$	=	stiffness of the hold-down connector

$$K_{c,90} = K_{hd} = 1{,}3 \cdot (b_2 + 30) \cdot h_2 = 1{,}3 \cdot (45 + 30) \cdot 95 = 9262{,}5 = 9{,}3 \cdot 10^3 \text{ [N/mm]}$$

XII.I Demonstration of multi-panel and equivalent-brace modelling method

First of all the multi-panel and equivalent-brace modelling approach are demonstrated with use of the geometry of the non-perforated timber-frame panel in shear-wall test (14.1).

Multi-panel approach shear-wall element test (14.1)

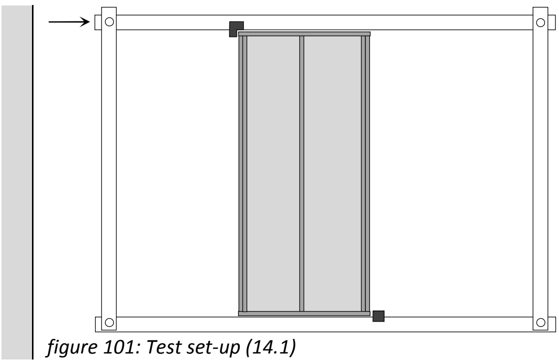
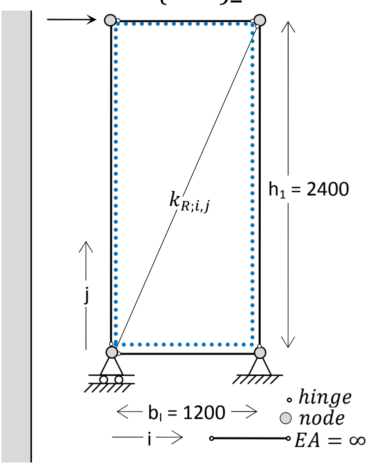


figure 101: Test set-up (14.1)

Test:	(14.1)
Sheathing material:	Gypsum paper board t = 12,5 mm (Gyproc)
Fastener:	Screw Ø3,0 x 37,6 s = 200 mm
K_{ser}:	667,62 N/mm
G_{mean}:	700 N/mm^2
Diaphragm dimensions:	1200 x 2400 mm
Perforation dimensions:	none
R_{test}:	320,78 N/mm
F_{04}:	1772,28 N

table 25: Properties of shear-wall in test (14.1)

Alternative (14.1)_A



In figure 102 a model is shown which represent the shear-wall in test (14.1). This is the model for which the analytical calculation method is made. The blue dotted line represents the fasteners present in the panel. The parameter $k_{R;i,j}$ will be determined below using the analytical method.

First of all, the fastener part will be analysed:

$$R_{f,Rd;i,j} = \frac{b_{i,j}}{2(1 + \frac{h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{2(1 + \frac{2400}{1200})} \cdot \frac{667{,}62}{200} = 667{,}62\ N/mm$$

Secondly, the shear contribution is analysed:

figure 102: Model test (14.1)

$$R_{G,Rd;i,j} = \frac{b_{i,j} \cdot t}{h_{i,j}} \cdot G_{mean} = \frac{1200 \cdot 12{,}5}{2400} \cdot 700 = 4375{,}00\ \mathrm{N/mm}$$

In the tests the uplift was prevented by the test-rig without hold-down anchor, but by means of compression on the top-rail. Consequently, the effect of hold-down slip is equal to the effect of compression perpendicular to grain below the compressed stud. This effect will be calculated for the element as a whole, and will be incorporated in the diagonal:

$$R_{hd,Rd;i,j} = \frac{{b_1}^2}{h_1^2} \cdot K_{hd} = R_{c,Rd;i,j} = \frac{{b_1}^2}{h_1^2} \cdot K_{c,90} = \frac{1200^2}{2400^2} \cdot 9262{,}50 = 2315{,}63\ N/mm$$

The effect of strain in the vertical studs due to the compressive and tensile force in the vertical studs will contribute to the stiffness of the shear-wall element as following:

$$R_{str,Rd;i,j} = \frac{b_2 \cdot h_2 \cdot {b_{\mathrm{i,j}}}^2}{\mathrm{h}_{\mathrm{i,j}}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{2400^3} \cdot 11000 = 4898{,}44\ N/mm$$

Combining all contributions calculated above, the stiffness of the shear wall can be calculated to be:

$$\begin{aligned} R_{Rd;i,j} &= \frac{1}{\frac{1}{R_{f,Rd;i,j}} + \frac{1}{R_{G,Rd;i,j}} + \frac{1}{R_{hd,Rd;i,j}} + \frac{1}{R_{c,Rd;i,j}} + \frac{1}{R_{str,Rd;i,j}}} \\ &= \frac{1}{\frac{1}{667{,}62} + \frac{1}{4375{,}00} + \frac{1}{2315{,}63} + \frac{1}{2315{,}63} + \frac{1}{4898{,}44}} = 357{,}87\ N/mm \end{aligned}$$

The stiffness of the diagonal can be calculated as following:

$$\mathrm{k}_{\mathrm{R_{Rd}};\mathrm{i,j}} = \mathrm{R}_{\mathrm{Rd;i,j}} \cdot \left(1 + \frac{{\mathrm{h}_{\mathrm{i,j}}}^2}{{\mathrm{b}_{\mathrm{i,j}}}^2}\right) = 357{,}87 \cdot \left(1 + \frac{2400^2}{1200^2}\right) = 1789{,}35$$

In the program for structural analysis timber C18 is used as a material for the diagonal brace. Young’s modulus of this material is E = 9000 N/mm2. Length of the brace is $l = \sqrt{1200^2 + 2400^2} = 2683{,}28\ \mathrm{mm}$. This gives the cross-sectional dimensions of the brace as:

$$\mathrm{A}_{\mathrm{i,j}} = \frac{\mathrm{k}_{\mathrm{R_{Rd}};\mathrm{i,j}} \cdot \mathrm{l}}{\mathrm{E}} = \frac{1789{,}35 \cdot 2683{,}28}{9000} = 533{,}48\ \mathrm{mm}^2 = 23{,}10 \times 23{,}10\ \mathrm{mm}$$

Alternative (14.1)_B

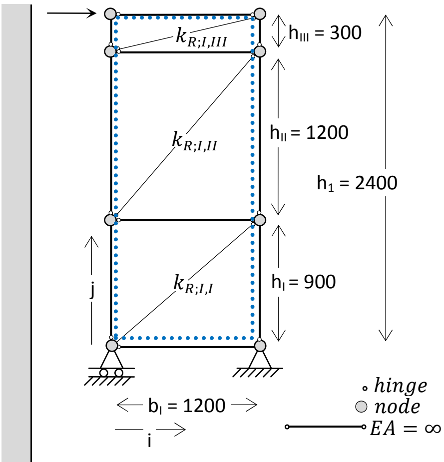


figure 103: Model test (14.1)

In figure 103 a second model is shown which represent the shear-wall in test (14.1). The parameters $k_{R;I,I}$, $k_{R;I,II}$, and $k_{R;I,III}$ will be determined below using the analytical method.

With respect to the fasteners, it can be seen that $k_{R;I,I}$ and $k_{R;I,III}$ will contain the effect of two vertical fastener rows, and one horizontal fastener row. $k_{R;I,II}$ will contain the effect of only two vertical fastener rows. Therefore, the equations have to be adapted as explained in paragraph 6.2.3 (page 96). The effect of fasteners for inclusion in $k_{R;I,I}$, $k_{R;I,II}$, and $k_{R;I,III}$, can be determined as following:

$$R_{f,Rd;I,I} = \frac{b_{I,I}}{(1 + \frac{2 \cdot h_{I,I}}{b_{I,I}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(1 + \frac{2 \cdot 900}{1200})} \cdot \frac{667,62}{200} = 1602,29\ N/mm$$

$$R_{f,Rd;I,II} = \frac{b_{I,II}}{\left(\frac{2 \cdot h_{I,II}}{b_{I,II}}\right)} \cdot \frac{K_{ser}}{s} = \frac{1200}{\left(\frac{2 \cdot 1200}{1200}\right)} \cdot \frac{667,62}{200} = 2002,86\ N/mm$$

$$R_{f,Rd;I,III} = \frac{b_{I,III}}{(1 + \frac{2 \cdot h_{I,III}}{b_{I,III}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(1 + \frac{2 \cdot 300}{1200})} \cdot \frac{667,62}{200} = 2670,48\ N/mm$$

The shear-contribution is calculated as following:

$$R_{G,Rd;I,I} = \frac{b_{I,I} \cdot t}{h_{I,I}} \cdot G_{mean} = \frac{1200 \cdot 12,5}{900} \cdot 700 = 11666,67\ \mathrm{N/mm}$$

$$R_{G,Rd;I,II} = \frac{b_{I,II} \cdot t}{h_{I,II}} \cdot G_{mean} = \frac{1200 \cdot 12,5}{1200} \cdot 700 = 8750,00\ \mathrm{N/mm}$$

$$R_{G,Rd;I,III} = \frac{b_{I,III} \cdot t}{h_{I,III}} \cdot G_{mean} = \frac{1200 \cdot 12,5}{300} \cdot 700 = 35000,00\ \mathrm{N/mm}$$

The effect of compression perpendicular to grain on the shear-wall panel as a whole, will be incorporated in the diagonal brace I,I:

$$R_{c,Rd;I,I} = \frac{{b_1}^2}{h_1^2} \cdot K_{c,90} = \frac{1200^2}{2400^2} \cdot 9262,50 = 2315,63\ N/mm$$

The effect of strain in the vertical studs due to the compressive and tensile force in the vertical studs will contribute to the stiffness of the shear-wall parts as following:

$$R_{str,Rd;I,I} = \frac{b_2 \cdot h_2 \cdot {b_{\mathrm{I,I}}}^2}{\mathrm{h}_{\mathrm{I,I}}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{900^3} \cdot 11000 = 92888,89\ N/mm$$

$$R_{str,Rd;I,I} = \frac{b_2 \cdot h_2 \cdot {b_{\mathrm{I,II}}}^2}{\mathrm{h}_{\mathrm{I,II}}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{1200^3} \cdot 11000 = 39187,50\ N/mm$$

$$R_{str,Rd;I,I} = \frac{b_2 \cdot h_2 \cdot {b_{\mathrm{I,II}}}^2}{\mathrm{h}_{\mathrm{I,III}}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{300^3} \cdot 11000 = 2508000{,}00\ N/mm$$

Combining all contributions calculated above, the stiffness of the shear-wall parts can be calculated to be:

$$R_{Rd;I,I} = \frac{1}{\frac{1}{1602{,}29} + \frac{1}{11666{,}67} + \frac{2}{2315{,}63} + \frac{1}{92888{,}89}} = 631{,}20\ N/mm$$

$$R_{Rd;I,II} = \frac{1}{\frac{1}{2002{,}86} + \frac{1}{8750{,}00} + \frac{1}{39187{,}50}} = 1564{,}72\ N/mm$$

$$R_{Rd;I,III} = \frac{1}{\frac{1}{2670{,}48} + \frac{1}{35000{,}00} + \frac{1}{2508000{,}00}} = 2478{,}72\ N/mm$$

The stiffness of the diagonals representing the shear-wall parts can be calculated as following:

$$\mathrm{k}_{\mathrm{R_{Rd;I,I}}} = 631{,}20 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 986{,}25$$

$$\mathrm{k}_{\mathrm{R_{Rd;I,II}}} = 1564{,}72 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 3129{,}44$$

$$\mathrm{k}_{\mathrm{R_{Rd;I,III}}} = 2478{,}72 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 2633{,}64$$

Length of the braces is given below:

$$\mathrm{l}_{\mathrm{I,I}} = \sqrt{1200^2 + 900^2} = 1500\ \mathrm{mm}$$

$$\mathrm{l}_{\mathrm{I,II}} = \sqrt{1200^2 + 1200^2} = 1697{,}06\ \mathrm{mm}$$

$$\mathrm{l}_{\mathrm{I,III}} = \sqrt{1200^2 + 300^2} = 1236{,}93\ \mathrm{mm}$$

This gives the cross-sectional dimensions of the brace as:

$$\mathrm{A}_{\mathrm{I,I}} = \frac{986{,}25 \cdot 1500}{9000} = 164{,}38\ \mathrm{mm}^2 = 12{,}82 \text{ x } 12{,}82\ \mathrm{mm}$$

$$\mathrm{A}_{\mathrm{I,II}} = \frac{3129{,}44 \cdot 1697{,}06}{9000} = 590{,}09\ \mathrm{mm}^2 = 24{,}29 \text{ x } 24{,}29\ \mathrm{mm}$$

$$\mathrm{A}_{\mathrm{I,III}} = \frac{2633{,}64 \cdot 1236{,}93}{9000} = 361{,}96\ \mathrm{mm}^2 = 19{,}03 \text{ x } 19{,}03\ \mathrm{mm}$$

Alternative (14.1)_C

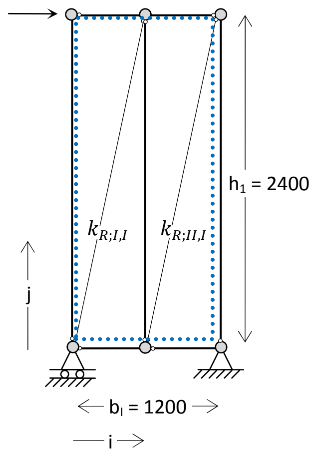


figure 104: Model test (14.1)

In figure 104 a third model is shown which represent the shear-wall in test (14.1). The parameters $k_{R;I,I}$ and $k_{R;I,II}$ will be determined below using the analytical method.

With respect to the fasteners, it can be seen that $k_{R;I,I}$ and $k_{R;II,I}$ will contain the effect of one vertical fastener row, and two horizontal fastener rows. Therefore, the equations have to be adapted. The effect of fasteners for inclusion in $k_{R;I,I}$, and $k_{R;II,I}$, can be determined as following:

$$R_{f,Rd;I,I} = R_{f,Rd;II,I} = \frac{b_{I,I}}{(2+\frac{h_{I,I}}{b_{I,I}})} \cdot \frac{K_{ser}}{s} = \frac{600}{(2+\frac{2400}{600})} \cdot \frac{667,62}{200} = 333,81\ N/mm$$

The shear-contribution is calculated as following:

$$R_{G,Rd;I,I} = R_{G,Rd;II,I} = \frac{b_{I,I} \cdot t}{h_{I,I}} \cdot G_{mean} = \frac{600 \cdot 12,5}{2400} \cdot 700 = 2187,50\ \mathrm{N/mm}$$

The effect of compression perpendicular to grain is present two times. The effect on the shear-wall panel as a whole, will be incorporated once in each brace:

$$R_{c,Rd;I,I} = R_{c,Rd;II,I} = 2315,63\ N/mm$$

The effect of strain in the vertical studs due to the compressive and tensile force in the vertical studs will contribute to the stiffness of the shear-wall parts as following:

$$R_{str,Rd;I,I} = R_{str,Rd;II,I} = \frac{b_2 \cdot h_2 \cdot b_{\mathrm{I,I}}{}^2}{\mathrm{h}_{\mathrm{I,I}}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 600^2}{2400^3} \cdot 11000 = 1224,61\ N/mm$$

Combining all contributions calculated above, the stiffness of the shear-wall parts can be calculated to be:

$$R_{Rd;I,I} = R_{Rd;II,I} = \frac{1}{\frac{1}{333,81} + \frac{1}{2187,50} + \frac{1}{2315,63} + \frac{1}{1224,61}} = 212,71\ N/mm$$

The stiffness of the diagonals representing the shear-wall parts can be calculated as following:

$$\mathrm{k}_{\mathrm{R_{Rd}};\mathrm{I,I}} = \mathrm{k}_{\mathrm{R_{Rd}};\mathrm{II,I}} = 212,71 \cdot \left(1 + \frac{2400^2}{600^2}\right) = 3616,07$$

Length of the braces is given below:

$$\mathrm{l}_{\mathrm{I,I}} = \mathrm{l}_{\mathrm{II,I}} = \sqrt{600^2 + 2400^2} = 2473,86\ \mathrm{mm}$$

This gives the cross-sectional dimensions of the brace as:

$$\mathrm{A}_{\mathrm{I,I}} = \mathrm{A}_{\mathrm{II,I}} = \frac{3616,07 \cdot 2473,86}{9000} = 993,96\ \mathrm{mm}^2 = 31,53\ \mathrm{x}\ 31,53\ \mathrm{mm}$$

Results

The three modelling approaches were analysed with use of a TechnosoftRaamwerken® calculation. Geometry and results are shown below.

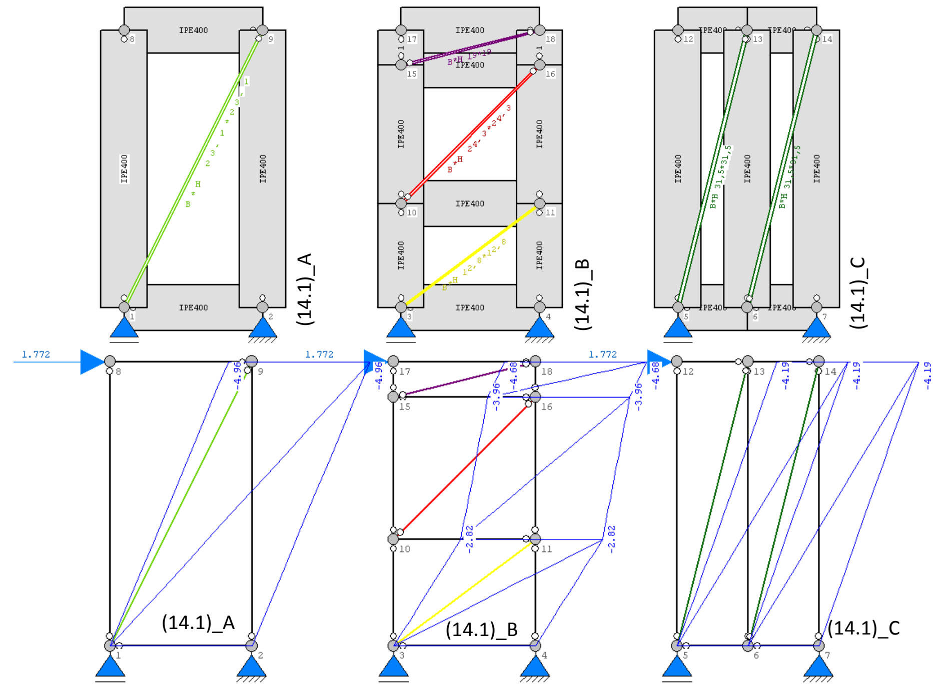


figure 105: Geometry and results of multi-panel modelling test (14.1) in framework program.

Alternative model for test (14.1)	**(A)**	**(B)**	**(C)**	
R_{model} [N/mm]	$\frac{1772}{4{,}96} = 357{,}26$ (111%)	$\frac{1772}{4{,}68} = 378{,}63$ (118%)	$\frac{1772}{4{,}19} = 422{,}91$ (132%)	R_{test} = 320,78 (100%) $R_{analytical}$ = 357,87 (112%)

table 26: Results of multi-panel modelling approaches test (14.1)

As can be seen from the table above, the analytically determined racking stiffness is 12% higher than the racking stiffness determined with use of test-based load-displacement data. Also the multi-panel modelling approach gives somewhat higher racking-stiffness, on one hand this can be explained by the use of the analytical calculation method, which result in a 12% higher racking stiffness in this case, but also the effect of the division into smaller panels turned out to given an additional increase in the model’s racking stiffness. The difference between $R_{analytical}$, and R_{model} of (14.1)_A can be explained by errors in the Technosoft® model due to rounding.

Equivalent-brace approach shear-wall element test (14.1)

From chapter 4 the analytical determined value for the racking stiffness of the shear-wall in tests (14.1) is known to be: $R_{analytical}$ = 357,87 N/mm. In the equivalent-brace models, the stiffness $k_{R;i,j}$ is chosen such that the combined effect of the braces gives the same stiffness to the shear-wall element as the analytically calculated value.

Alternative (14.1)_D

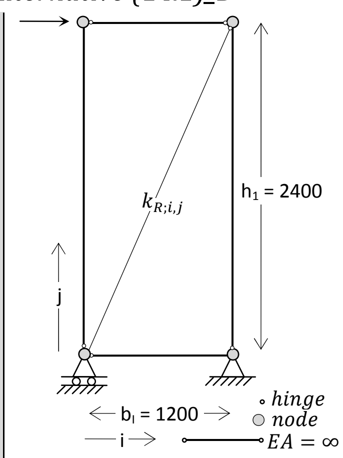


figure 106: Model test (14.1)

$$k_{R;i,j} = R_{analytical} \cdot \left(1 + \frac{{h_1}^2}{{b_1}^2}\right) = 357{,}87 \cdot \left(1 + \frac{2400^2}{1200^2}\right) = 1789{,}35\ N/mm$$

Length of the brace is:

$$\mathrm{l_{i,j}} = \sqrt{1200^2 + 2400^2} = 2683{,}28\ \mathrm{mm}$$

This gives the cross-sectional dimensions of the brace:

$$\mathrm{A_{i,j}} = \frac{1789{,}35 \cdot 2683{,}28}{9000} = 533{,}48\ \mathrm{mm^2} = 23{,}1\ \mathrm{x}\ 23{,}1\ \mathrm{mm}$$

Alternative (14.1)_E

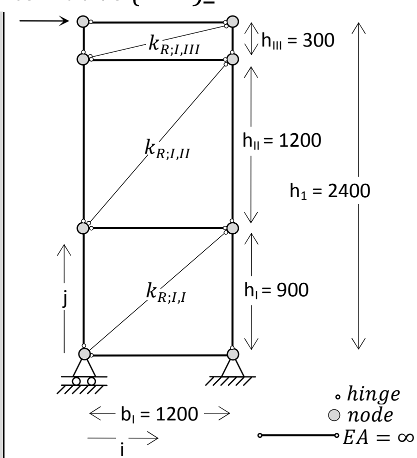


figure 107: Model test (14.1)

$$k_{R;j} = \frac{h}{h_j} \cdot R_{analytical} \cdot \left(1 + \frac{h_j^2}{b_1^2}\right)$$

$$k_{R;I,I} = \frac{2400}{900} \cdot 357{,}87 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 1491{,}13\ N/mm$$

$$k_{R;I,II} = \frac{2400}{1200} \cdot 357{,}87 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 1431{,}48\ N/mm$$

$$k_{R;I,III} = \frac{2400}{300} \cdot 357{,}87 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 3041{,}90\ N/mm$$

This gives the cross-sectional dimensions of the brace as:

$$A_{I,I} = \frac{1491{,}13 \cdot 1500}{9000} = 248{,}52\ mm^2 = 15{,}76\ x\ 15{,}76\ mm$$

$$A_{I,II} = \frac{1431{,}48 \cdot 1697{,}06}{9000} = 269{,}92\ mm^2 = 16{,}43\ x\ 16{,}43\ mm$$

$$A_{I,III} = \frac{3041{,}90 \cdot 1236{,}93}{9000} = 418{,}07\ mm^2 = 20{,}45\ x\ 20{,}45\ mm$$

Alternative (14.1)_F

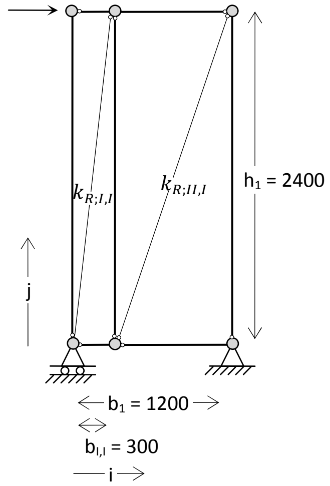


figure 108: Model test (14.1)

$$k_{R;j} = \frac{b_i}{b} \cdot R_{analytical} \cdot \left(1 + \frac{h_1^2}{b_i^2}\right)$$

$$k_{R;I,I} = \frac{300}{1200} \cdot 357{,}87 \cdot \left(1 + \frac{2400^2}{300^2}\right) = 5815{,}39$$

$$k_{R;II,I} = \frac{900}{1200} \cdot 357{,}87 \cdot \left(1 + \frac{2400^2}{900^2}\right) = 2177{,}04$$

Length of the braces is:

$$\mathrm{l}_{\mathrm{I,I}} = \sqrt{300^2 + 2400^2} = 2418{,}68 \text{ mm}$$

$$\mathrm{l}_{\mathrm{II,I}} = \sqrt{900^2 + 2400^2} = 2563{,}20 \text{ mm}$$

This results in:

$$A_{I,I} = \frac{5815{,}39 \cdot 2418{,}68}{9000} = 1562{,}84\ mm^2 = 39{,}53\ x\ 39{,}53\ mm$$

$$A_{II,I} = \frac{2177{,}04 \cdot 2563{,}20}{9000} = 620{,}02\ mm^2 = 24{,}90\ x\ 24{,}90\ mm$$

Results

The result of these three alternatives to the equivalent-brace method are shown below.

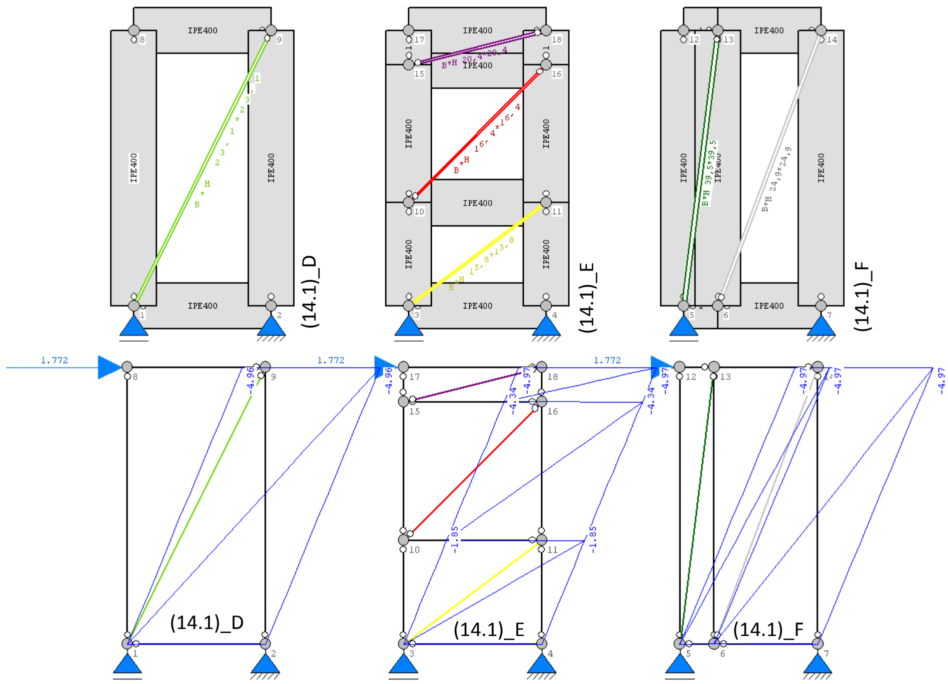


figure 109: Geometry and results of equivalent-brace modelling test (14.1) in framework program.

Alternative model for test (14.1)	**(D)**	**(E)**	**(F)**	R_{test} = 320,78 (100%) $R_{analytical}$ = 357,87 (112%)
R_{model} [N/mm]	$\frac{1772}{4{,}96} = 357{,}26$ (111%)	$\frac{1772}{4{,}97} = 356{,}54$ (111%)	$\frac{1772}{4{,}97} = 356{,}54$ (111%)	

table 27: Results of equivalent-brace modelling approaches test (14.1)

From figure 109 and figure 105 can be concluded that the equivalent-braces modelling approach does not affect the results for a non-perforated shear-wall element. In the following paragraphs will be analysed if the presented methods can be useful in modelling perforated shear-wall elements, this will give insight in the applicability of both possibilities.

XII.II Application of multi-panel and equivalent-brace modelling method

The perforated timber-frame shear-walls were tested in a series of tests, which also consist of non-perforated timber-frame shear-wall elements. These walls are useful for verification between perforated and non-perforated models.

Shear-wall element in test (14.3)

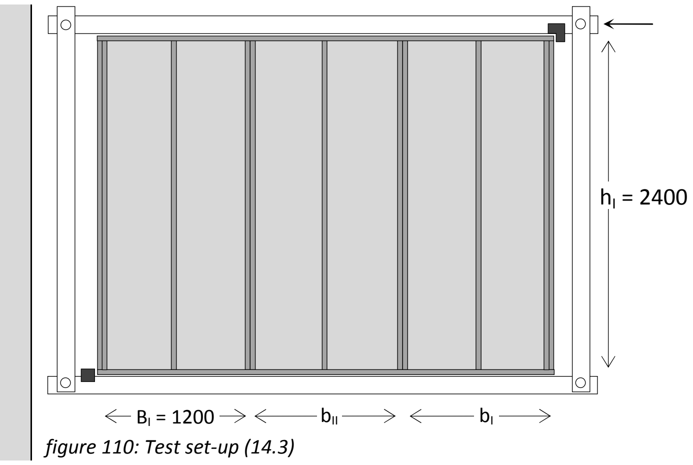


figure 110: Test set-up (14.3)

Test:	**(14.3)**
Sheathing material:	Gypsum paper board t = 12,5 mm (Gyproc)
Fastener:	Screw Ø3,0 x 37,6 s = 200 mm
Diaphragm dimensions:	3600 x 2400 mm
Perforation dimensions:	none
R_{test}:	1355,70 N/mm
F_{04}:	5266,84 N

table 28: Properties of shear-wall in test (14.3)

Multi-panel model (14.3)

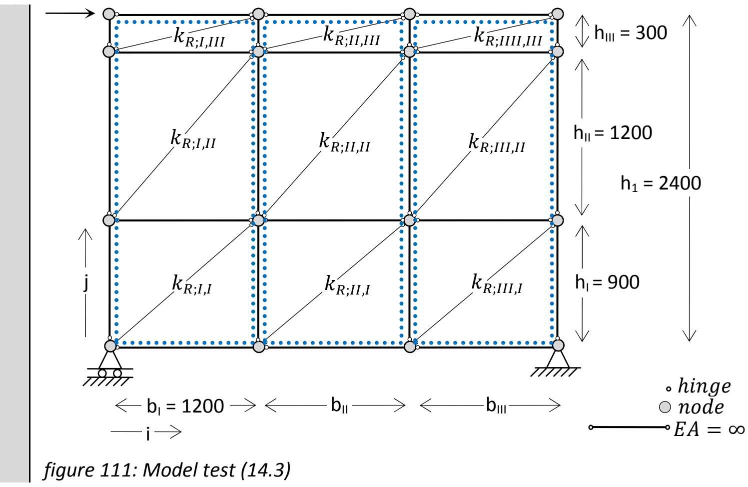


figure 111: Model test (14.3)

In figure 111 a model is shown which represent the shear-wall in test (14.3). This is the model for which the analytical calculation method is made. The blue dotted line represents the fasteners present in the panel. The parameter $k_{R;i,j}$ will be determined below using the analytical method.

First of all, the fastener part will be analysed:

In general:

$$R_{f,Rd;i,j} = \frac{b_{i,j}}{2(1+\frac{h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s}$$

In panel parts i,I one horizontal and two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{(1+\frac{2 \cdot h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(1+\frac{2 \cdot 900}{1200})} \cdot \frac{667{,}62}{200} = 1602{,}29\ N/mm$$

In panel parts i,II only two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{(\frac{2 \cdot h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(\frac{2 \cdot 1200}{1200})} \cdot \frac{667{,}62}{200} = 2002{,}86\ N/mm$$

In panel parts i,III one horizontal and two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{1200}{\left(1+\frac{2 \cdot 300}{1200}\right)} \cdot \frac{667{,}62}{200} = 2670{,}48\ N/mm$$

The shear contribution can be calculated as following:

$$R_{G,Rd;i,I} = \frac{b_{i,I} \cdot t}{h_{i,I}} \cdot G_{mean} = \frac{1200 \cdot 12{,}5}{900} \cdot 700 = 11666{,}67 \text{ N/mm}$$

$$R_{G,Rd;i,II} = \frac{b_{i,II} \cdot t}{h_{i,II}} \cdot G_{mean} = \frac{1200 \cdot 12{,}5}{1200} \cdot 700 = 8750{,}00 \text{ N/mm}$$

$$R_{G,Rd;i,III} = \frac{b_{i,III} \cdot t}{h_{i,III}} \cdot G_{mean} = \frac{1200 \cdot 12{,}5}{300} \cdot 700 = 35000{,}00 \text{ N/mm}$$

In the tests the uplift was prevented by the test-rig without hold-down anchor, but by means of compression on the top-rail. Consequently, the effect of hold-down slip is equal to the effect of compression perpendicular to grain below the compressed stud. This effect will be calculated for the element as a whole, and will be incorporated in the diagonals of element parts I,I and III,I:

$$R_{hd,Rd;I,I} = R_{c,Rd;III,I} = \frac{{b_1}^2}{h_1^2} \cdot K_{c,90} = \frac{3600^2}{2400^2} \cdot 9262{,}50 = 20840{,}63\ N/mm$$

The effect of strain in the vertical studs due to the compressive and tensile force in the vertical studs will contribute to the stiffness of the shear-wall element as following:

$$R_{str,Rd;i,I} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{I}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{900^3} \cdot 11000 = 92888{,}89\ N/mm$$

$$R_{str,Rd;i,II} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{II}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{1200^3} \cdot 11000 = 39187{,}50\ N/mm$$

$$R_{str,Rd;i,III} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{III}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{300^3} \cdot 11000 = 2508000{,}00\ N/mm$$

Combining all contributions calculated above, the stiffness of the panel parts $R_{Rd;i,j}$ can be calculated. This racking stiffness can be translated to a brace representing the panel part. The dimensions of the braces can be calculated as following:

<u>Parts I,I and III,I</u>

$$R_{Rd;I,I} = R_{Rd;III,I} = \frac{1}{\frac{1}{1602{,}29} + \frac{1}{20840{,}63} + \frac{1}{11666{,}67} + \frac{1}{92888{,}89}} = 1301{,}12\ N/mm$$

$$\mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{III,I}} = \mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{I,I}} = \mathrm{R}_{\mathrm{Rd;I,I}} \cdot \left(1 + \frac{{\mathrm{h_I}}^2}{{\mathrm{b_I}}^2}\right) = 1301{,}12 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 2033{,}00$$

E (C18) = 9000 N/mm^2

length of the brace is $l_{I,I} = l_{I,III} = \sqrt{1200^2 + 900^2} = 1500 \text{ mm}$

$$\mathrm{A_{I,I}} = \mathrm{A_{III,I}} = \frac{\mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{I,I}} \cdot \mathrm{l_{I,I}}}{\mathrm{E}} = \frac{2033{,}00 \cdot 1500{,}00}{9000} = 338{,}83 \text{ mm}^2 = 18{,}41 \text{ x } 18{,}41 \text{ mm}$$

<u>Part II,I</u>

$$R_{Rd;II,I} = \frac{1}{\frac{1}{1602{,}29} + \frac{1}{11666{,}67} + \frac{1}{92888{,}89}} = 1387{,}76\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd};II,I}} = 1387{,}76 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 2168{,}38\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{II,I}} = \frac{2168{,}38 \cdot 1500{,}00}{9000} = 361{,}39\ \mathrm{mm}^2 = 19{,}01\ \mathrm{x}\ 19{,}01\ \mathrm{mm}$$

<u>Part i,II</u>

$$R_{Rd;i,II} = \frac{1}{\frac{1}{2002{,}86} + \frac{1}{8750{,}00} + \frac{1}{39187{,}50}} = 1564{,}72\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd};i,II}} = 1564{,}72 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 3129{,}45\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{i,II}} = \frac{3129{,}45 \cdot 1697{,}06}{9000} = 590{,}10\ \mathrm{mm}^2 = 24{,}29\ \mathrm{x}\ 24{,}29\ \mathrm{mm}$$

<u>Part i,III</u>

$$R_{Rd;i,III} = \frac{1}{\frac{1}{2670{,}48} + \frac{1}{3500{,}00} + \frac{1}{2508000{,}00}} = 2478{,}72\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd};i,III}} = 2478{,}72 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 2633{,}64\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{i,III}} = \frac{2633{,}64 \cdot 1236{,}93}{9000} = 361{,}96\ \mathrm{mm}^2 = 19{,}03\ \mathrm{x}\ 19{,}03\ \mathrm{mm}$$

The multi-panel approach led to the determination of four different brace dimensions (as calculated above). The model is loaded with the 40% F_{max} load from the test: F_{04} = 5266,84 N

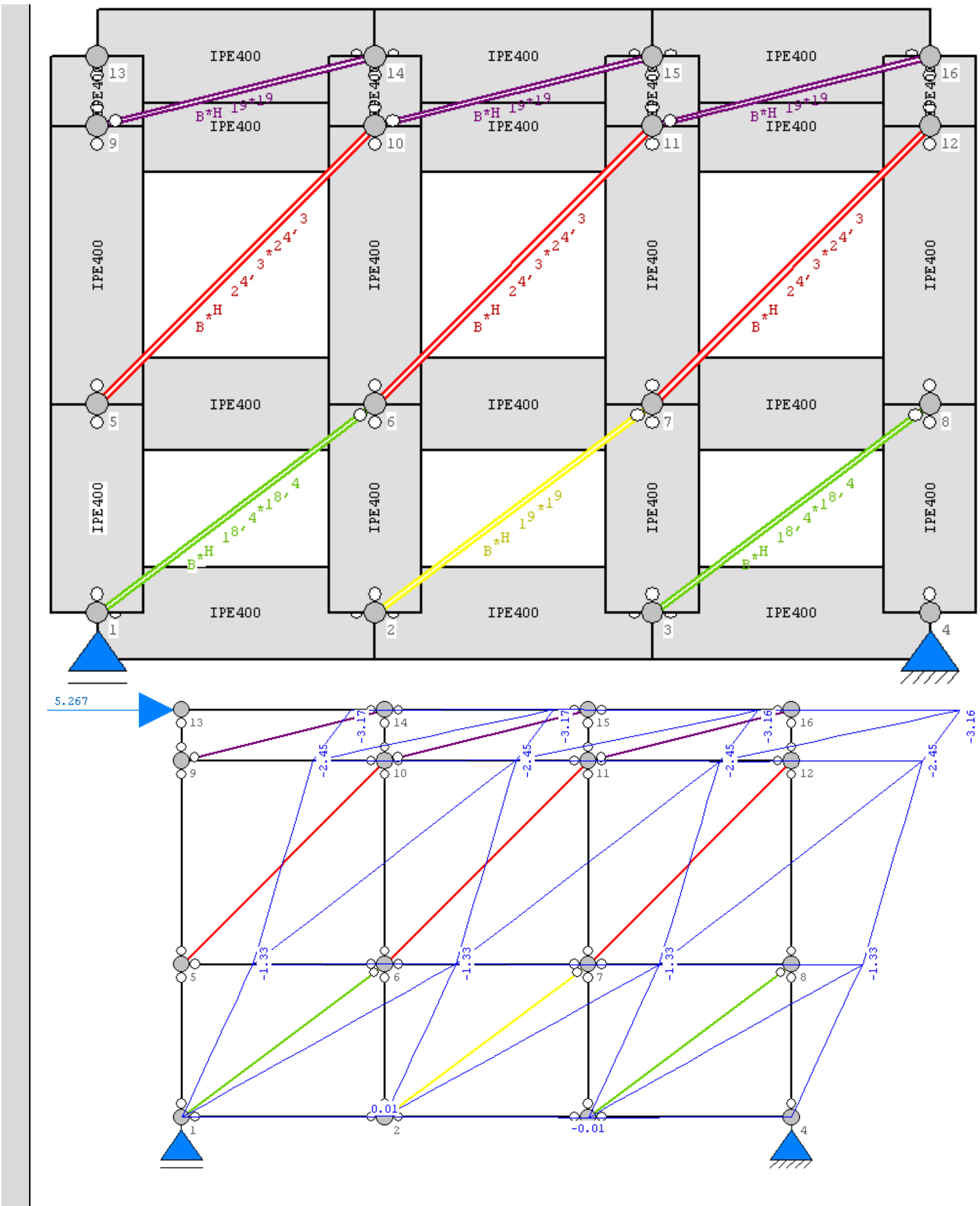


figure 112: Geometry and results of equivalent-brace modelling test (14.3) in framework program.

$$R_{\text{multi-panel}} = \frac{5267}{3{,}16} = 1666{,}77 \text{ N/mm}$$

Equivalent-brace model (14.3)

The procedure to determine the brace stiffness for the elements i,j with use of the equivalent-brace model was explained in paragraph 6.2.4 (page 98), and illustrated with test (14.1) earlier in this appendix. Based on the exercises with use of test (14.1) the following formula for the equivalent-brace stiffness was derived:

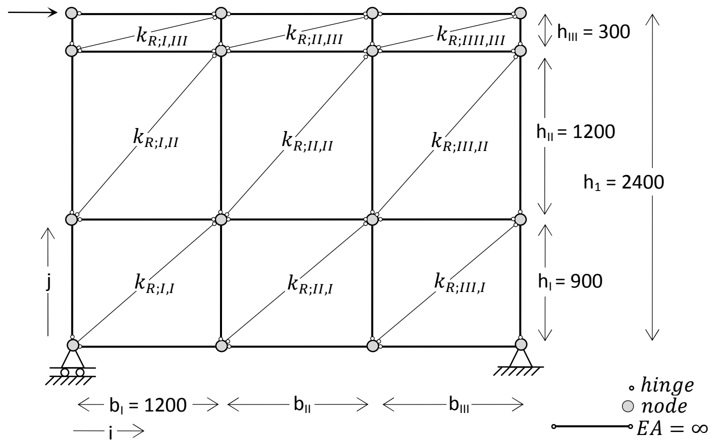


figure 113: Model test (14.3)

$$k_{i,j} = \frac{h_1}{h_i} \cdot \frac{b_i}{b_1} \cdot R \cdot (1 + \frac{h_i^2}{b_i^2})$$

in which R is the racking stiffness of the shear-wall element. R can be determined with use of the analytical method from paragraph 4.8.

From Appendix VII (page 149) is known that: $R_{analytical}$ for test (14.3) = 1440,66 N/mm.

<u>Brace I,I, II,I and III,I</u>

$$k_{i,I} = \frac{2400}{900} \cdot \frac{1200}{3600} \cdot 1440{,}66 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 2000{,}92\ N/mm$$

$$\mathrm{A_{i,I}} = \frac{2000{,}92 \cdot 1500{,}00}{9000} = 333{,}49\ \mathrm{mm}^2 = 18{,}26 \times 18{,}26\ \mathrm{mm}$$

<u>Brace I,II, II,II and III,II</u>

$$k_{i,II} = \frac{2400}{1200} \cdot \frac{1200}{3600} \cdot 1440{,}66 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 1920{,}88\ N/mm$$

$$\mathrm{A_{i,II}} = \frac{1920{,}88 \cdot 1697{,}06}{9000} = 362{,}20\ \mathrm{mm}^2 = 19{,}03 \times 19{,}03\ \mathrm{mm}$$

<u>Brace I,III, II,III and III,III</u>

$$k_{i,III} = \frac{2400}{300} \cdot \frac{1200}{3600} \cdot 1440{,}66 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 4081{,}87\ N/mm$$

$$\mathrm{A_{i,III}} = \frac{4081{,}87 \cdot 1236{,}93}{9000} = 561{,}00\ \mathrm{mm}^2 = 23{,}69 \times 23{,}69\ \mathrm{mm}$$

The equivalent-brace approach led to the determination of three different brace dimensions (as calculated above). The model is loaded with the 40% F_{max} load from the test: F_{04} = 5266,84 N

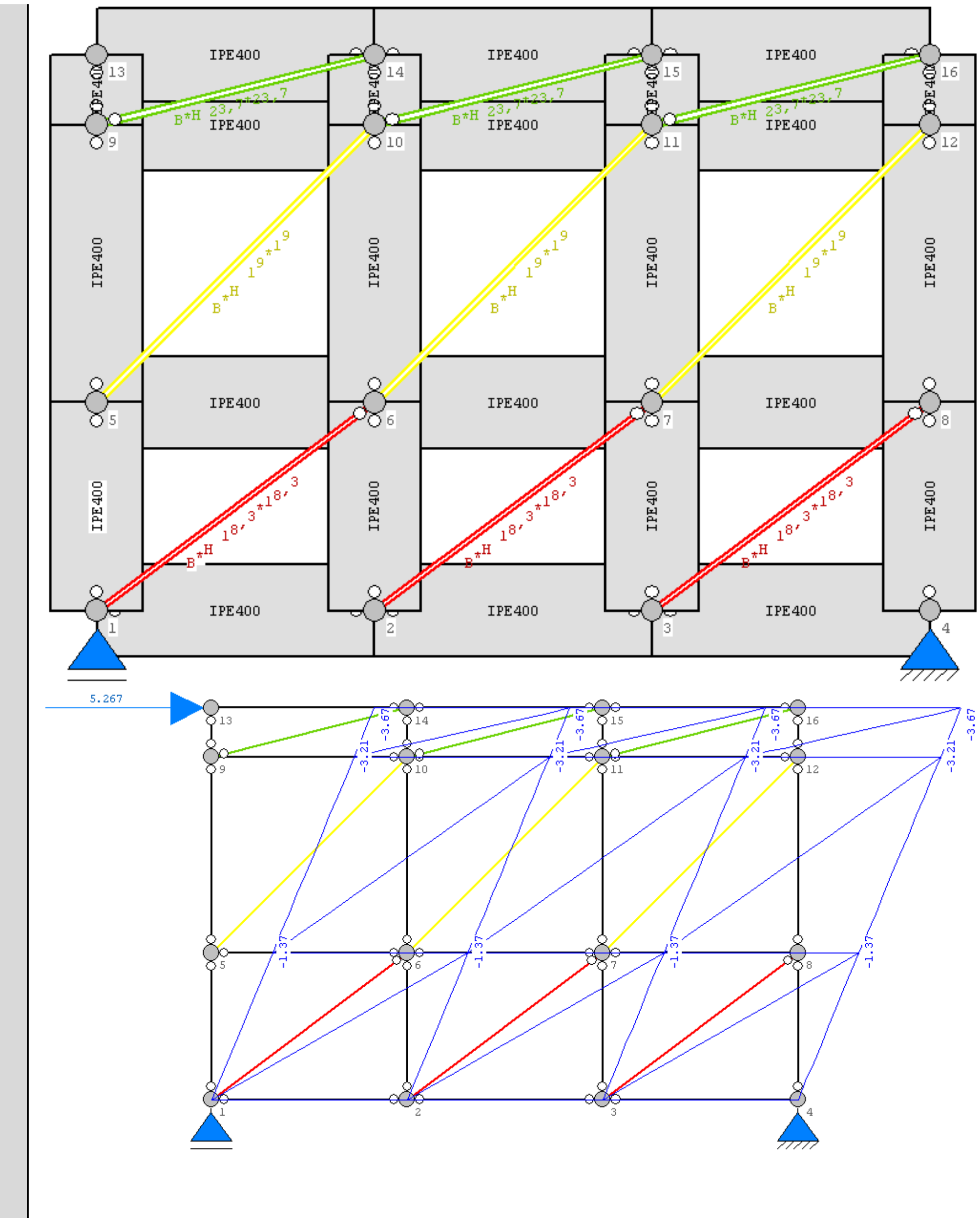


figure 114: Geometry and results of equivalent-brace modelling test (14.3) in framework program.

$$\mathrm{R_{equivalent-brace}} = \frac{5267}{3{,}67} = 1435{,}15\ \mathrm{N/mm}$$

Results

Test (14.3)	Multi-panel model	Equivalent-brace model	
R_{model} [N/mm]	$\frac{5267}{3{,}16} = 1666{,}77\ \mathrm{N/mm}$ (123%)	$\frac{5267}{3{,}67} = 1435{,}15\ \mathrm{N/mm}$ (106%)	R_{test} = 1355,70 (100%) $R_{analytical}$ = 1440,66 (106%)

table 29: Results of modelling approaches test (14.3)

From table 29 can be concluded that both the multi-panel and equivalent-brace model give reasonable results. The stiffness of the multi-panel model differs 23% with the racking-stiffness obtained in the test. The stiffness of the equivalent-brace method differs 6% with the racking-stiffness obtained in the test. With use of test (14.8) will be analysed if the methods can be useful for modelling of perforated shear-walls. The shear-walls in test (14.3) and (14.8) have exactly the same properties except for the perforation.

Shear-wall element in test (14.8)

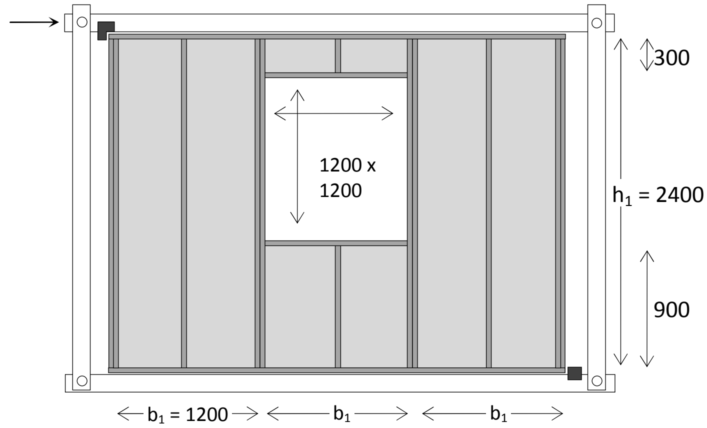


figure 115: Test set-up (14.8)

Test:	(14.8)
Sheathing material:	Gypsum paper board t = 12,5 mm (Gyproc)
Fastener:	Screw Ø3,0 x 37,6 s = 200 mm
Diaphragm dimensions:	3600 x 2400 mm
Perforation dimensions:	window: 1200 x 1200 mm
R_{test}:	823,51 N/mm
F_{04}:	3973,26 N

table 30: Properties of shear-wall in test (14.8)

Multi-panel model (14.8)

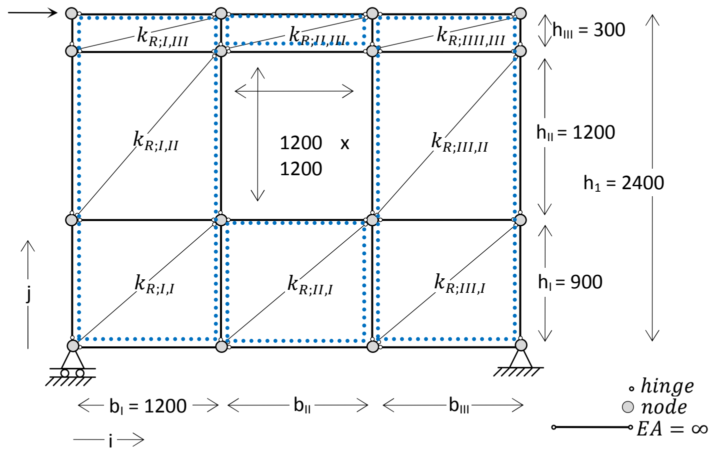


figure 116: Model test (14.8)

In figure 116 a model is shown which represent the shear-wall in test (14.8). The blue dotted line represents the fasteners present in the panel. As can be seen from and , the element consists of four panels and a window, and is translated into a model with 9 - 1 = 8 braces.

Because the perforated shear-wall in test (14.8) has the same properties as the non-perforated shear-wall in test (14.3), the parameters $k_{R;I,I}$, $k_{R;III,I}$, $k_{R;I,II}$, $k_{R;III,II}$, $k_{R;I,III}$, and $k_{R;III,III}$ remain the same. Only $k_{R;II,I}$ and $k_{R;II,III}$ will change in magnitude because of a different fastener pattern. $k_{R;II,I}$ and $k_{R;II,III}$ will be calculated below.

First of all, the fastener contribution will be analysed for both parts:

$$R_{f,Rd;II,I} == \frac{1200}{2(1+\frac{900}{1200})} \cdot \frac{667,62}{200} = 1144,49\ N/mm$$

$$R_{f,Rd;II,III} == \frac{1200}{2(1+\frac{300}{1200})} \cdot \frac{667,62}{200} = 1602,29\ N/mm$$

Shear deformation and strain in the studs remain the same. This will result in:

$$R_{Rd;II,I} = \frac{1}{\frac{1}{1144,49} + \frac{1}{11666,67} + \frac{1}{92888,89}} = 1030,68\ N/mm$$

$$\mathrm{k_{R_{Rd};II,I}} = 1030,68 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 1610,44\ \mathrm{N/mm}$$

$$\mathrm{A_{II,I}} = \frac{1610,44 \cdot 1500,00}{9000} = 268,41\ \mathrm{mm^2} = 16,38\ \mathrm{x}\ 16,38\ \mathrm{mm}$$

$$R_{Rd;II,III} = \frac{1}{\frac{1}{1602,29} + \frac{1}{35000,00} + \frac{1}{2508000,00}} = 1531,21\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd};II,III}} = 1531,21 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 1626,91\ \mathrm{N/mm}$$

$$\mathrm{A_{II,III}} = \frac{1626,91 \cdot 1236,93}{9000} = 223,60\ \mathrm{mm^2} = 14,95\ \mathrm{x}\ 14,95\ \mathrm{mm}$$

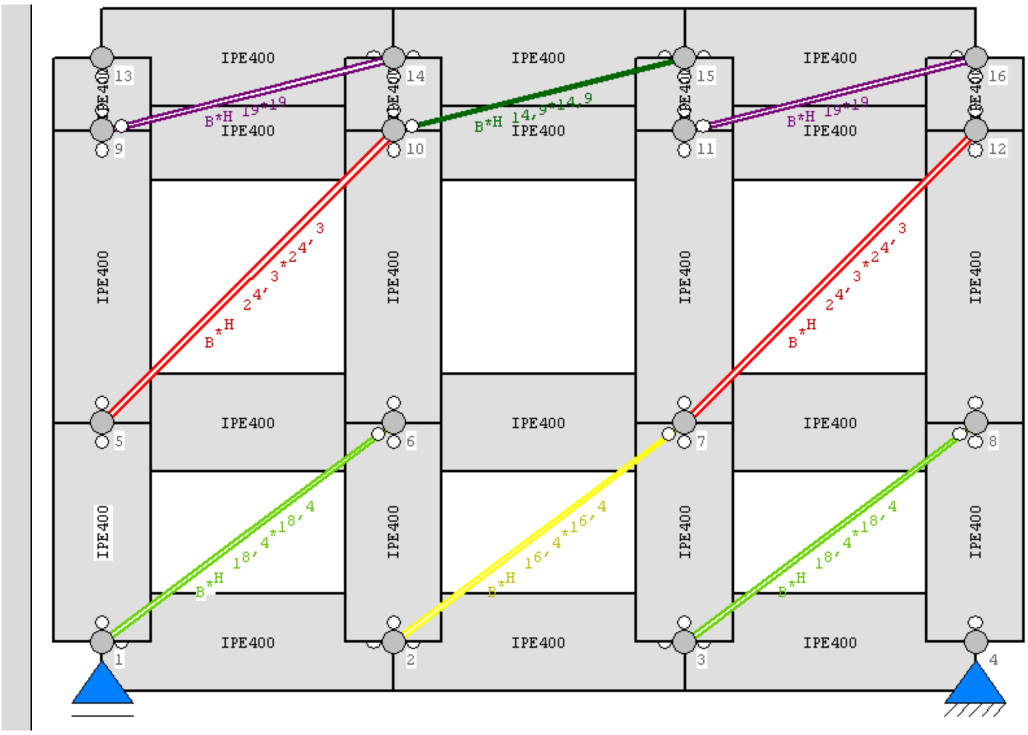
From the multi-panel modelling of test (14.3) is known that:

$$\mathrm{A_{I,I}} = \mathrm{A_{III,I}} = 18,41\ \mathrm{x}\ 18,41\ \mathrm{mm}$$

$$\mathrm{A_{I,II}} = \mathrm{A_{III,II}} = 24,29\ \mathrm{x}\ 24,29\ \mathrm{mm}$$

$$\mathrm{A_{I,III}} = \mathrm{A_{III,III}} = 19,03\ \mathrm{x}\ 19,03\ \mathrm{mm}$$

The multi-panel approach led to the determination of five different brace dimensions (as given above).



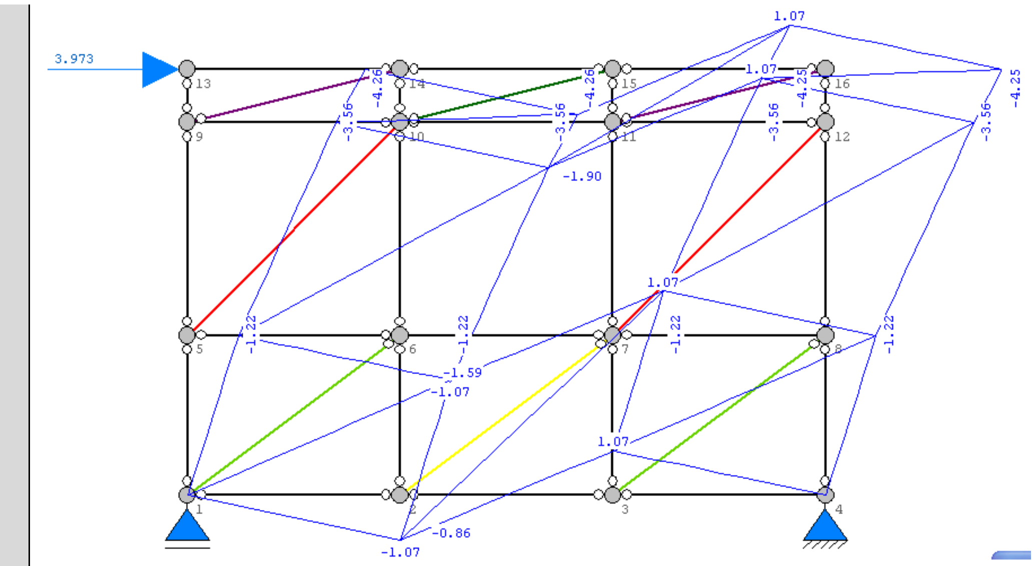


figure 117: Geometry and results of multi-panel modelling test (14.8) in framework program.

$$R_{\text{multi-panel}} = \frac{3973}{4{,}25} = 934{,}82\ \text{N/mm}$$

Equivalent-brace model (14.8)

The equivalent-brace model of test (14.3) was determined such that the model with nine braces represents the analytically determined shear-wall racking stiffness. Because no value for the racking stiffness can be determined for perforated shear-walls with use of the analytical method, the equivalent-brace model of test (14.3) will be used. The perforation will be made by removing the central brace, at the location of the window. This can be seen in the mode below.

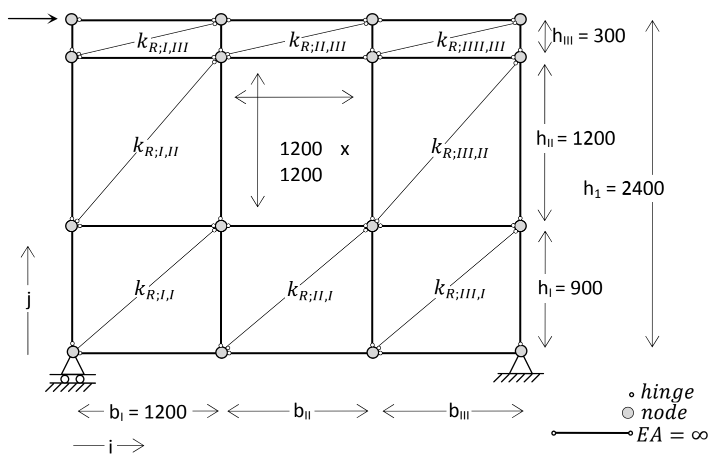


figure 118: Model test (14.8)

The brace remain similar to the equivalent-brace model of test (14.3):

$$A_{i,I} = 18{,}26 \text{ x } 18{,}26 \text{ mm}$$

$$A_{i,II} = 19{,}03 \text{ x } 19{,}03 \text{ mm}$$

$$A_{i,III} = 23{,}69 \text{ x } 23{,}69 \text{ mm}$$

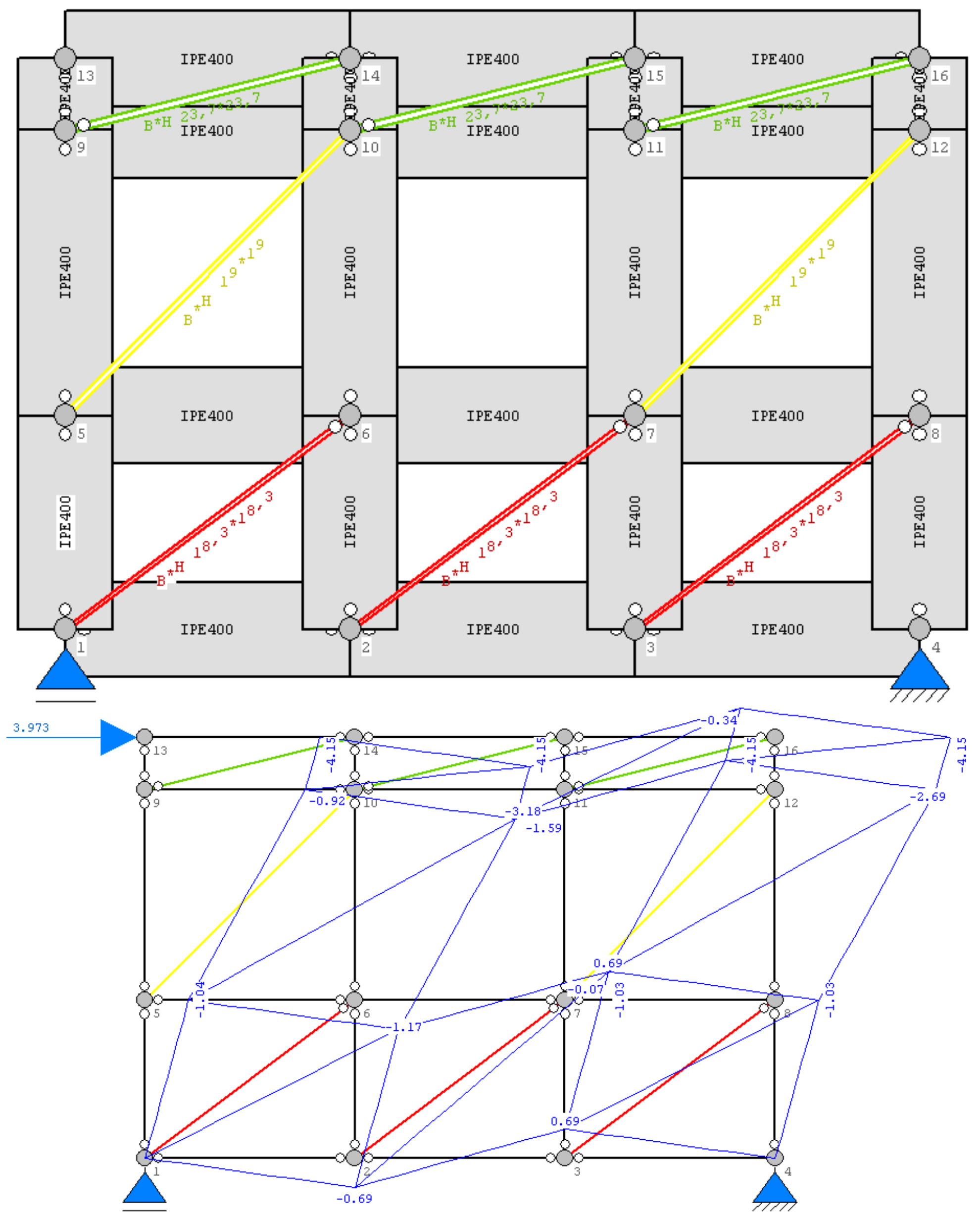


figure 119: Geometry and results of equivalent-brace modelling test (14.8) in framework program.

$$R_{\text{equivalent-brace}} = \frac{3973}{4{,}15} = 957{,}35\ \text{N/mm}$$

Results

Test (14.8)	Multi-panel model	Equivalent-brace model	
R_{model} [N/mm]	$\frac{3973}{4{,}25} = 934{,}82$ (114%)	$\frac{3973}{4{,}15} = 957{,}35$ (116%)	R_{test} = 823,51 (100%)

table 31: Results of modelling approaches test (14.8)

From table 31 can be concluded that both the multi-panel and equivalent-brace model give a reasonable estimation of the shear-wall stiffness. The stiffness of the multi-panel model differs 14% with the racking-stiffness obtained in the test. The stiffness of the equivalent-brace method differs 16% with the racking-stiffness obtained in the test.

Shear-wall element in test (14.15)

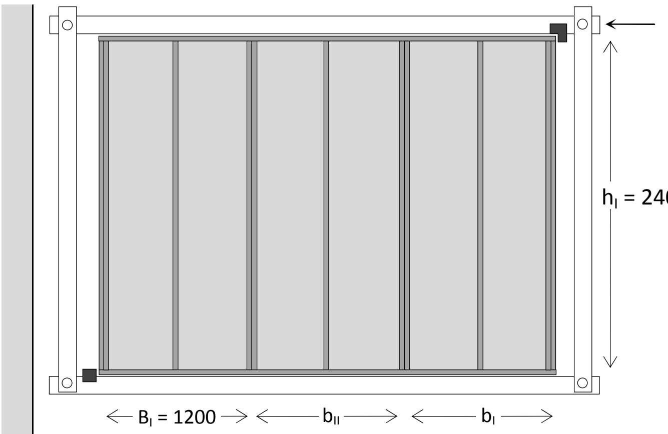


figure 120: Test set-up (14.15)

Test:	(14.15)
Sheathing material:	Particleboard t = 12,3 mm
Fastener:	Nail Ø2,13 x 50,1 s = 100 mm
Diaphragm dimensions:	3600 x 2400 mm
Perforation dimensions:	none
R_{test}:	1988,36
F_{04}:	8338,12

table 32: Properties of shear-wall in test (14.15)

Multi-panel model (14.15)

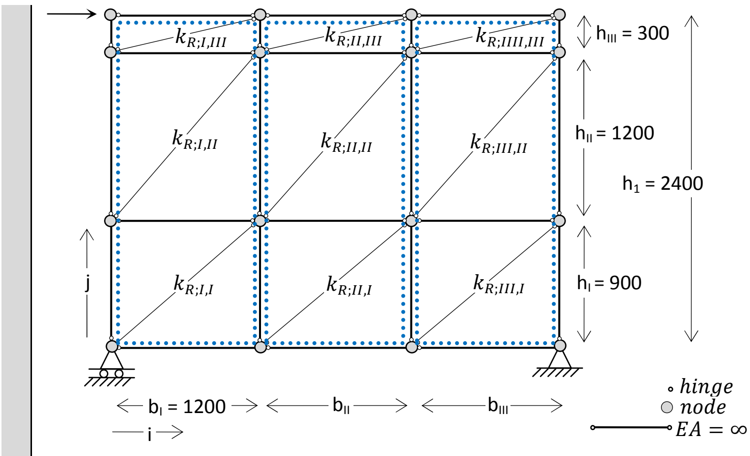


figure 121: Model test (14.15)

In figure 121, a model is shown which represent the shear-wall in test (14.15). The blue dotted line represents the fasteners present in the panel. The parameter $k_{R;i,j}$ will be determined below for each element part, using the analytical method.

First of all, the fastener contribution will be analysed:

In general:

$$R_{f,Rd;i,j} = \frac{b_{i,j}}{2(1 + \frac{h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s}$$

In panel parts i,I one horizontal and two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{(1 + \frac{2 \cdot h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(1 + \frac{2 \cdot 900}{1200})} \cdot \frac{728{,}96}{100} = 3499{,}01\ N/mm$$

In panel parts i,II only two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{(\frac{2 \cdot h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{(\frac{2 \cdot 1200}{1200})} \cdot \frac{728{,}96}{100} = 4373{,}76\ N/mm$$

In panel parts i,III one horizontal and two vertical rows of fasteners are present:

$$R_{f,Rd;i,I} = \frac{1200}{\left(1 + \frac{2 \cdot 300}{1200}\right)} \cdot \frac{728{,}96}{100} = 5831{,}68\ N/mm$$

The shear contribution can be calculated as following:

$$R_{G,Rd;i,I} = \frac{b_{i,I} \cdot t}{h_{i,I}} \cdot G_{mean} = \frac{1200 \cdot 12{,}3}{900} \cdot 960 = 15744{,}00\ \mathrm{N/mm}$$

$$R_{G,Rd;i,II} = \frac{b_{i,II} \cdot t}{h_{i,II}} \cdot G_{mean} = \frac{1200 \cdot 12{,}3}{1200} \cdot 960 = 11808{,}00\ \mathrm{N/mm}$$

$$R_{G,Rd;i,III} = \frac{b_{i,III} \cdot t}{h_{i,III}} \cdot G_{mean} = \frac{1200 \cdot 12{,}3}{300} \cdot 960 = 47232{,}00\ \mathrm{N/mm}$$

In the tests, the uplift was prevented by the test-rig without hold-down anchor, but by means of compression on the top-rail. Consequently, the effect of hold-down slip is equal to the effect of compression perpendicular to grain below the compressed stud. This effect will be calculated for the element as a whole, and will be incorporated in the diagonals of element parts I,I and III,I:

$$R_{hd,Rd;I,I} = R_{c,Rd;III,I} = \frac{{b_1}^2}{h_1^2} \cdot K_{c,90} = \frac{3600^2}{2400^2} \cdot 9262{,}50 = 20840{,}63\ N/mm$$

The effect of strain in the vertical studs due to the compressive and tensile force in the vertical studs will contribute to the stiffness of the shear-wall element as following:

$$R_{str,Rd;i,I} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{I}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{900^3} \cdot 11000 = 92888{,}89\ N/mm$$

$$R_{str,Rd;i,II} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{II}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{1200^3} \cdot 11000 = 39187{,}50\ N/mm$$

$$R_{str,Rd;i,III} = \frac{b_2 \cdot h_2 \cdot {b_\mathrm{i}}^2}{\mathrm{h}_\mathrm{III}^3} \cdot E_2 = \frac{45 \cdot 95 \cdot 1200^2}{300^3} \cdot 11000 = 2508000{,}00\ N/mm$$

Combining all contributions calculated above, the stiffness of the panel parts $R_{Rd;i,j}$ can be calculated. This racking stiffness can be translated to a brace representing the panel part. The dimensions of the braces can be calculated as following:

<u>Parts I,I and III,I</u>

$$R_{Rd;I,I} = R_{Rd;III,I} = \frac{1}{\frac{1}{3499{,}01} + \frac{1}{20840{,}63} + \frac{1}{15744{,}00} + \frac{1}{92888{,}89}} = 2450{,}62\ N/mm$$

$$\mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{III,I}} = \mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{I,I}} = \mathrm{R}_{\mathrm{Rd;I,I}} \cdot \left(1 + \frac{{\mathrm{h}_\mathrm{I}}^2}{{\mathrm{b}_\mathrm{I}}^2}\right) = 2450{,}62 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 3829{,}09$$

E (C18) = 9000 N/mm^2

length of the brace is $l_{I,I} = l_{I,III} = \sqrt{1200^2 + 900^2} = 1500\ \mathrm{mm}$

$$\mathrm{A}_{\mathrm{I,I}} = \mathrm{A}_{\mathrm{III,I}} = \frac{\mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{I,I}} \cdot \mathrm{l}_{\mathrm{I,I}}}{\mathrm{E}} = \frac{3829{,}09 \cdot 1500{,}00}{9000} = 638{,}18\ \mathrm{mm}^2 = 25{,}26\ \mathrm{x}\ 25{,}26\ \mathrm{mm}$$

<u>Part II,I</u>

$$R_{Rd;II,I} = \frac{1}{\frac{1}{3499{,}01} + \frac{1}{15744{,}00} + \frac{1}{92888{,}89}} = 2777{,}18\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd;II,I}}} = 2777{,}18 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 4339{,}34\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{II,I}} = \frac{4339{,}34 \cdot 1500{,}00}{9000} = 723{,}22\ \mathrm{mm}^2 = 26{,}89\ \mathrm{x}\ 26{,}89\ \mathrm{mm}$$

<u>Part i,II</u>

$$R_{Rd;i,II} = \frac{1}{\frac{1}{4373{,}76} + \frac{1}{11808{,}00} + \frac{1}{39187{,}50}} = 2951{,}22\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd;i,II}}} = 2951{,}22 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 5902{,}44\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{i,II}} = \frac{5902{,}44 \cdot 1697{,}06}{9000} = 1112{,}98\ \mathrm{mm}^2 = 33{,}36\ \mathrm{x}\ 33{,}36\ \mathrm{mm}$$

<u>Part i,III</u>

$$R_{Rd;i,III} = \frac{1}{\frac{1}{5831{,}68} + \frac{1}{47232{,}0} + \frac{1}{2508000{,}00}} = 5180{,}06\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd;i,III}}} = 5180{,}06 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 5503{,}81\ \mathrm{N/mm}$$

$$\mathrm{A}_{\mathrm{i,III}} = \frac{5503{,}81 \cdot 1236{,}93}{9000} = 756{,}43\ \mathrm{mm}^2 = 27{,}50\ \mathrm{x}\ 27{,}50\ \mathrm{mm}$$

The multi-panel approach led to the determination of four different brace dimensions (as calculated above). The model is loaded with the 40% F_{max} load from the test: F_{04} = 8338,12 N

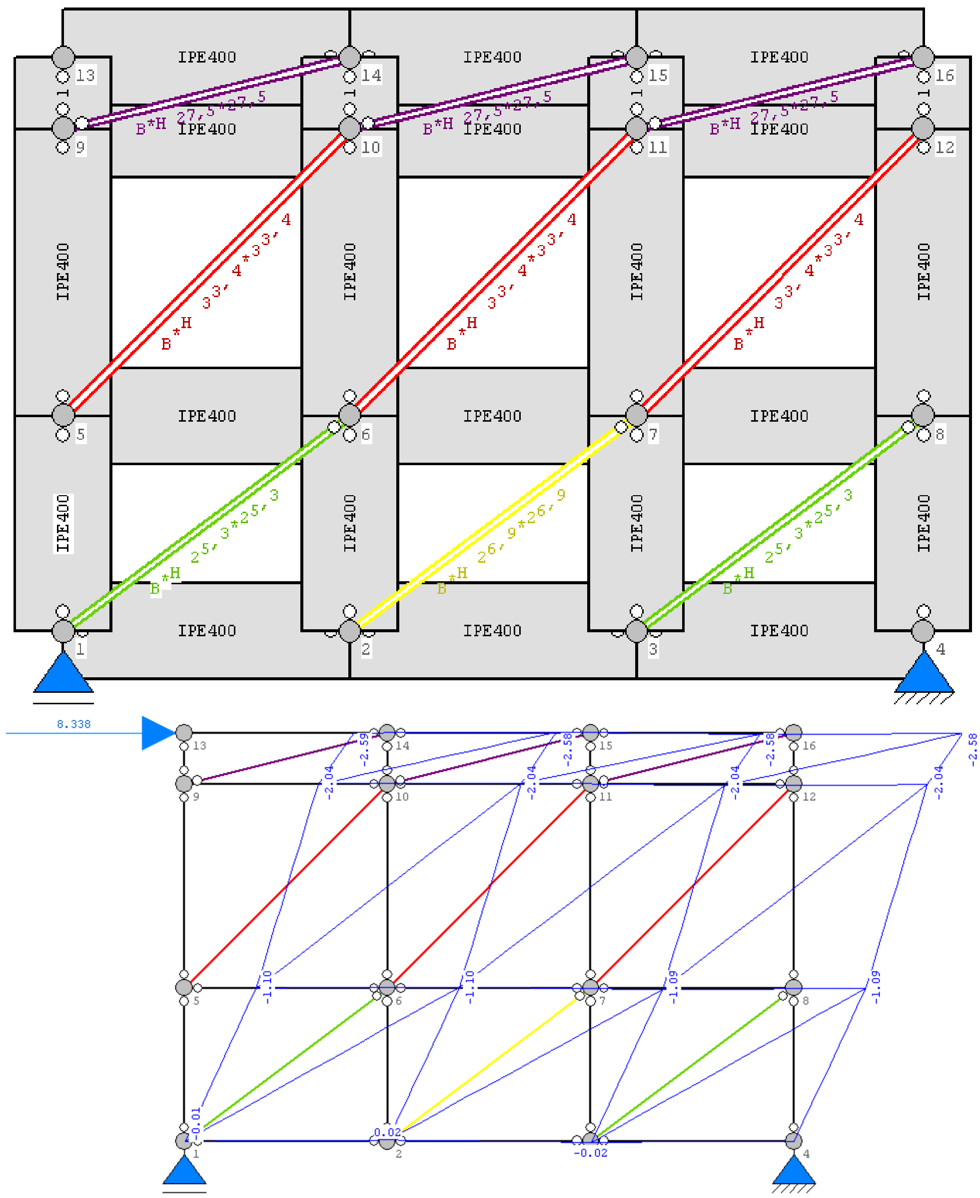


figure 122: Geometry and results of multi-panel modelling test (14.15) in framework program.

$$R_{\text{multi-panel}} = \frac{8338}{2{,}58} = 3231{,}78\ \mathrm{N/mm}$$

Equivalent-brace model (14.15)

The procedure to determine the brace stiffness for the elements i,j with use of the equivalent-brace model was illustrated with test (14.3) earlier in this chapter.

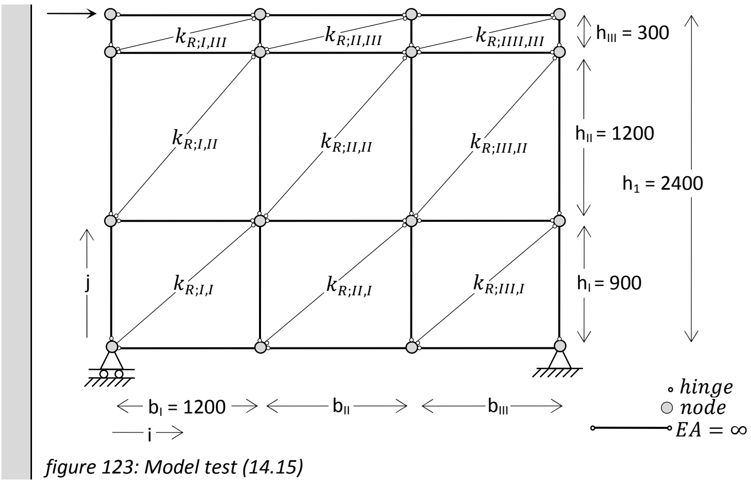


figure 123: Model test (14.15)

From Appendix VII (page 149) is known that: $R_{analytical}$ for test (14.15) = 2476,82 N/mm, this is the racking stiffness determined with the analytical method as shown in paragraph 4.8 (page 78).

<u>In general:</u>

$$k_{i,j} = \frac{h_1}{h_i} \cdot \frac{b_i}{b_1} \cdot R \cdot (1 + \frac{h_i^2}{b_i^2})$$

<u>Brace I,I, II,I and III,I</u>

$$k_{i,I} = \frac{2400}{900} \cdot \frac{1200}{3600} \cdot 2476{,}82 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 3440{,}03\ N/mm$$

$$\mathrm{A_{i,I}} = \frac{3440{,}03 \cdot 1500{,}00}{9000} = 573{,}34\ \mathrm{mm}^2 = 23{,}94 \text{ x } 23{,}94\ \mathrm{mm}$$

<u>Brace I,II, II,II and III,II</u>

$$k_{i,II} = \frac{2400}{1200} \cdot \frac{1200}{3600} \cdot 2476{,}82 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 3302{,}43\ N/mm$$

$$\mathrm{A_{i,II}} = \frac{3302{,}43 \cdot 1697{,}06}{9000} = 622{,}72\ \mathrm{mm}^2 = 24{,}95 \text{ x } 24{,}95\ \mathrm{mm}$$

<u>Brace I,III, II,III and III,III</u>

$$k_{i,III} = \frac{2400}{300} \cdot \frac{1200}{3600} \cdot 2476{,}82 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 7017{,}66\ N/mm$$

$$\mathrm{A_{i,III}} = \frac{7017{,}66 \cdot 1236{,}93}{9000} = 964{,}48\ \mathrm{mm}^2 = 31{,}06 \text{ x } 31{,}06\ \mathrm{mm}$$

The equivalent-brace approach led to the determination of three different brace dimensions (as calculated above). The model is loaded with the 40% F_{max} load from the test: F_{04} = 8338,12 N

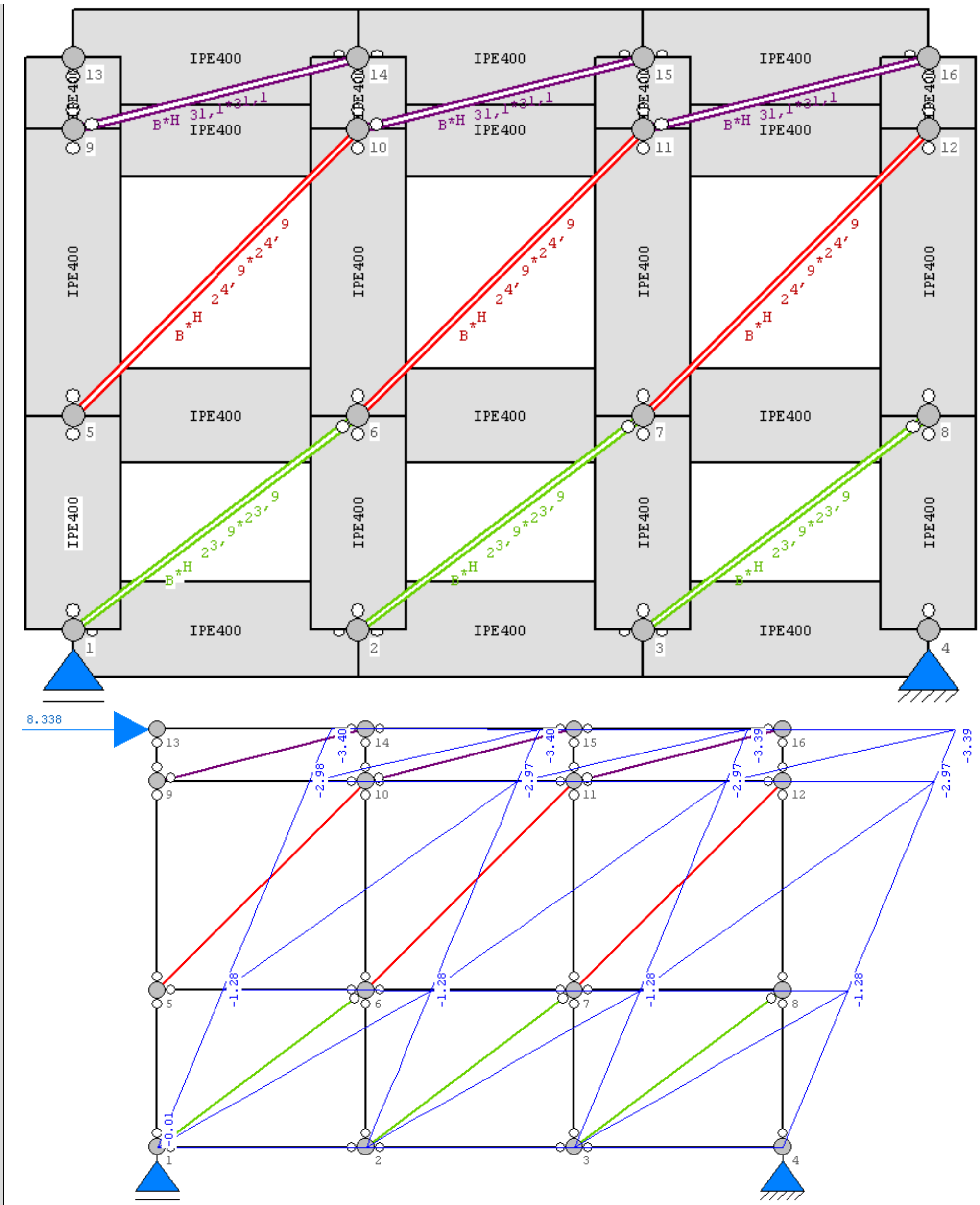


figure 124: Geometry and results of equivalent-brace modelling test (14.15) in framework program.

$$\mathrm{R}_{\text{equivalent-brace}} = \frac{8338}{3{,}39} = 2459{,}59\ \mathrm{N/mm}$$

Results

Test (14.15)	Multi-panel model	Equivalent-brace model	
R_{model} [N/mm]	$\frac{8338}{2{,}58} = 3231{,}78$ (163%)	$\frac{8338}{3{,}39} = 2459{,}59$ (124%)	R_{test} = 1988,36 (100%) $R_{analytical}$ = 2476,82 (125%)

table 33: Results of modelling approaches test (14.15)

From table 33 can be concluded that both the multi-panel and equivalent-brace model give reasonable results. The stiffness of the multi-panel model differs 63% with the racking-stiffness obtained in the test. The stiffness of the equivalent-brace method differs 24% with the racking-stiffness obtained in the test. As can be seen from comparison with tests (14.3) and (14.8), the differences between test and analytical calculation are somewhat larger. This can have to do with the uncertainty in K_{ser} for particleboard sheathing-to-timber fasteners. The shear-walls in test (14.15) and (14.16) have exactly the same properties except for the perforation. The perforated shear-wall element in test (14.16) will be analysed on the next pages.

Shear-wall element in test (14.16)

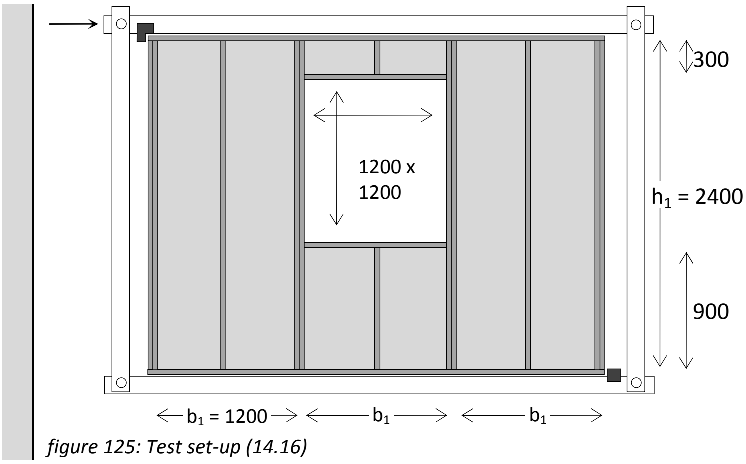


figure 125: Test set-up (14.16)

Test:	(14.16)
Sheathing material:	Particleboard t = 12,3 mm
Fastener:	Nail Ø2,13 x 50,1 s = 100 mm
Diaphragm dimensions:	3600 x 2400 mm
Perforation dimensions:	window: 1200 x 1200 mm
R_{test}:	1131,67 N/mm
F_{04}:	6841,36 N

table 34: Properties of shear-wall in test (14.16)

Multi-panel model (14.16)

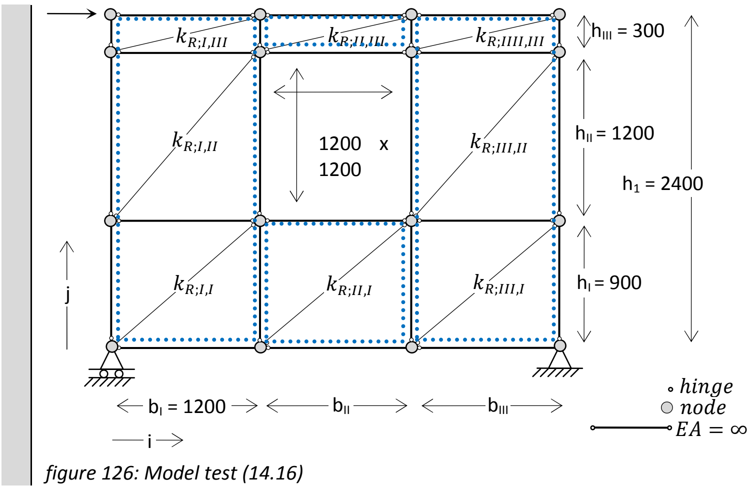


figure 126: Model test (14.16)

In figure 126 a model is shown which represent the shear-wall in test (14.16). The blue dotted line represents the fasteners present in the panel. As can be seen from figure 125 and figure 126, the element consists of four panels and a window, and is translated into a model with 8 braces.

Because the perforated shear-wall in test (14.16) has the same properties as the non-perforated shear-wall in test (14.15), the parameters $k_{R;I,I}$, $k_{R;III,I}$, $k_{R;I,II}$, $k_{R;III,II}$, $k_{R;I,III}$, and $k_{R;III,III}$ remain the same. Only $k_{R;II,I}$ and $k_{R;II,III}$ will change in magnitude because of a different fastener pattern (reference is made to figure 121). $k_{R;II,I}$ and $k_{R;II,III}$ will be calculated below.

First of all, the fastener contribution will be analysed:

$$R_{f,Rd;II,I} = \frac{1200}{2(1+\frac{900}{1200})} \cdot \frac{728{,}96}{100} = 2499{,}29\ N/mm$$

$$R_{f,Rd;II,III} == \frac{1200}{2(1+\frac{300}{1200})} \cdot \frac{728{,}96}{100} = 3499{,}01\ N/mm$$

Shear deformation and strain in the studs remain the same. This will result in:

$$R_{Rd;II,I} = \frac{1}{\frac{1}{2499{,}29} + \frac{1}{15744{,}00} + \frac{1}{92888{,}89}} = 2107{,}95\ N/mm$$

$$\mathrm{k}_{\mathrm{R}_{\mathrm{Rd}};\mathrm{II,I}} = 2107{,}95 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 3293{,}67\ \mathrm{N/mm}$$

$$A_{II,I} = \frac{3293{,}67 \cdot 1500{,}00}{9000} = 548{,}95\ \text{mm}^2 = 23{,}43 \text{ x } 23{,}43 \text{ mm}$$

$$R_{Rd;II,III} = \frac{1}{\frac{1}{3499{,}01} + \frac{1}{47232{,}00} + \frac{1}{2508000{,}00}} = 3253{,}45\ N/mm$$

$$k_{R_{Rd};II,III} = 3253{,}45 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 3456{,}79\ \text{N/mm}$$

$$A_{II,III} = \frac{3456{,}79 \cdot 1236{,}93}{9000} = 475{,}09\ \text{mm}^2 = 21{,}80 \text{ x } 21{,}80 \text{ mm}$$

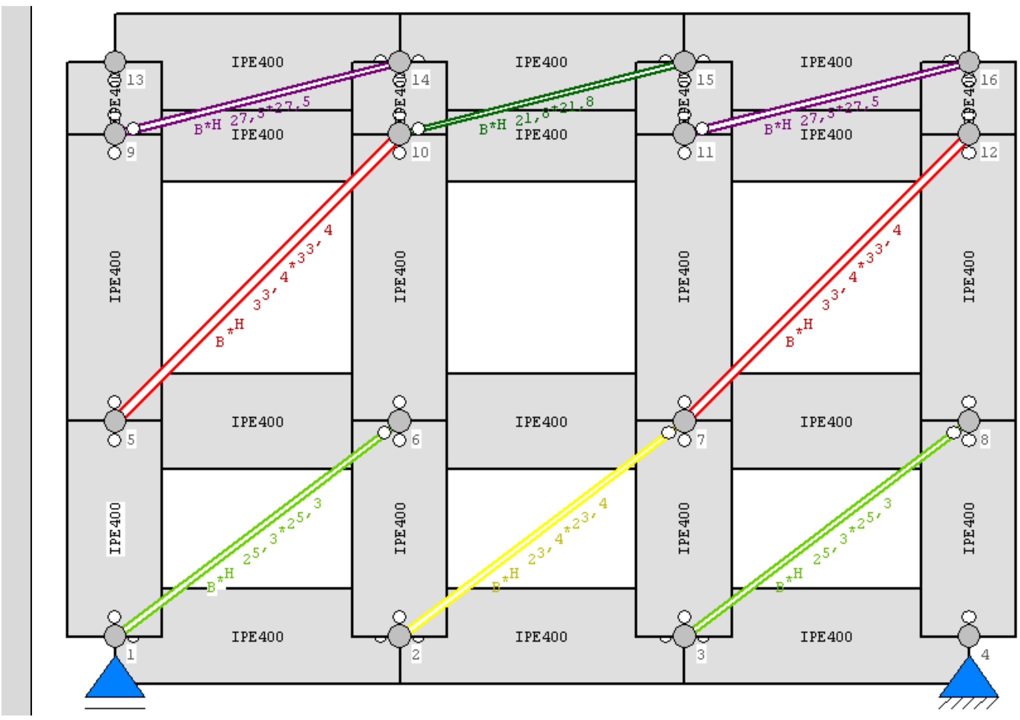
From the multi-panel modelling of test (14.15) is known that:

$$A_{I,I} = A_{III,I} = 25{,}26 \text{ x } 25{,}26 \text{ mm}$$

$$A_{I,II} = A_{III,II} = 33{,}36 \text{ x } 33{,}36 \text{ mm}$$

$$A_{I,III} = A_{III,III} = 27{,}50 \text{ x } 27{,}50 \text{ mm}$$

The multi-panel approach led to the determination of five different brace dimensions (as given above).



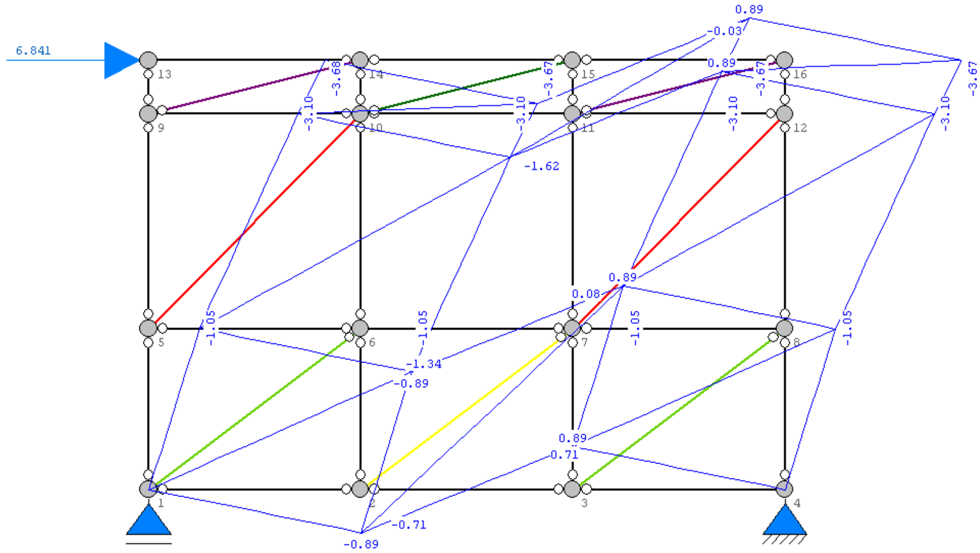


figure 127: Geometry and results of multi-panel modelling test (14.16) in framework program.

$$\mathrm{R}_{\mathrm{multi-panel}} = \frac{6841}{3{,}67} = 1864{,}03\ \mathrm{N/mm}$$

Equivalent-brace model (14.16)

The equivalent-brace model of test (14.15) was determined such that the model with 9 braces represents the analytically determined shear-wall racking stiffness. With use of the analytical method, as originally proposed in paragraph 4.8, no value for the racking stiffness can be determined for perforated shear-walls. Therefore, the equivalent-brace model of test (14.15) will be used to model the perforated shear-wall in test (14.16). The perforation will be made by removing the central brace, at the location of the window. This can be seen in the mode below.

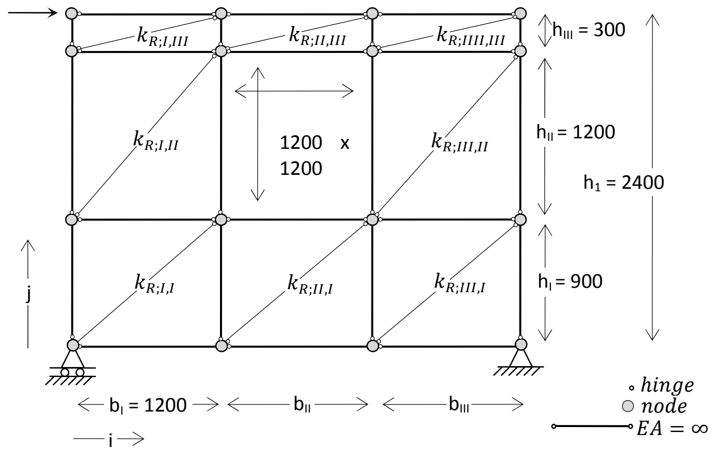


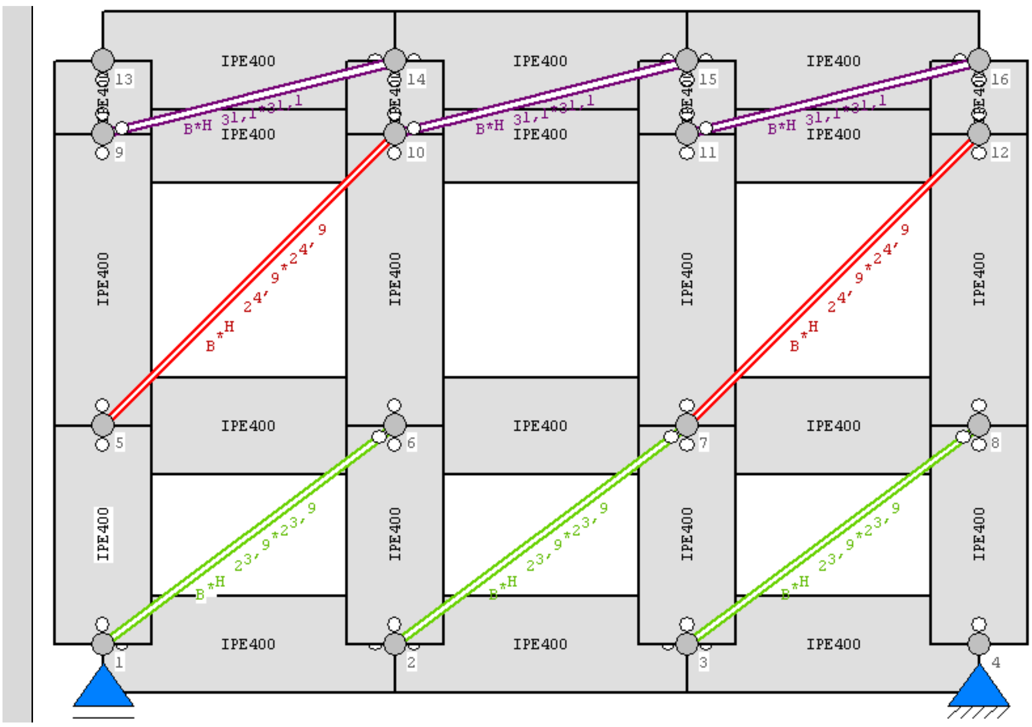
figure 128: Model test (14.16)

The braces remain similar to the equivalent-brace model of test (14.15):

$$A_{i,I} = 23{,}94 \text{ x } 23{,}94 \text{ mm}$$

$$A_{i,II} = 24{,}95 \text{ x } 24{,}95 \text{ mm}$$

$$A_{i,III} = 31{,}06 \text{ x } 31{,}06 \text{ mm}$$



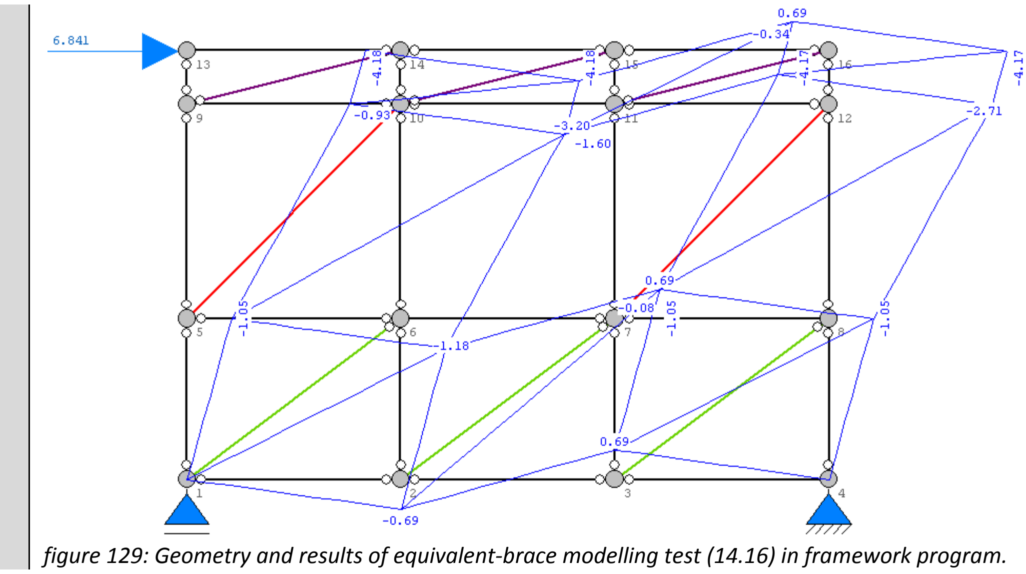


figure 129: Geometry and results of equivalent-brace modelling test (14.16) in framework program.

$$R_{\text{equivalent-brace}} = \frac{6841}{4{,}17} = 1640{,}53\ \text{N/mm}$$

Results

Test (14.16)	Multi-panel model	Equivalent-brace model	
R_{model} [N/mm]	$\frac{6841}{3{,}67} = 1864{,}03$ (165%)	$\frac{6841}{4{,}17} = 1640{,}53$ (145%)	R_{test} = 1131,67 (100%)

table 35: Results of modelling approaches test (14.16)

From table 35 can be concluded that both the multi-panel and equivalent-brace model give reasonable results. The stiffness of the multi-panel model differs 65% with the racking-stiffness obtained in the test. The stiffness of the equivalent-brace method differs 45% with the racking-stiffness obtained in the test. As can be seen from comparison with tests (14.3) and (14.8), the differences between test and analytical calculation are somewhat larger. This can have to do with the uncertainty in K_{ser} for particleboard sheathing-to-timber fasteners. This effect is also visible in the non-perforated counterpart of test (14.16), in the results of test (14.15).

XII.III Additional observations

In addition to the models that were presented before, some additional models were made with use of the perforated timber-frame shear-wall from test (14.8).

Multi-panel model (14.8) without changing fastener equation

In paragraph 6.2.3 was explained why the equation for the fastener component of the shear-wall racking stiffness was changed if used in the multi-panel modelling approach. In this paragraph will be demonstrated what will happen if the equation for the fastener-stiffness is not being changed.

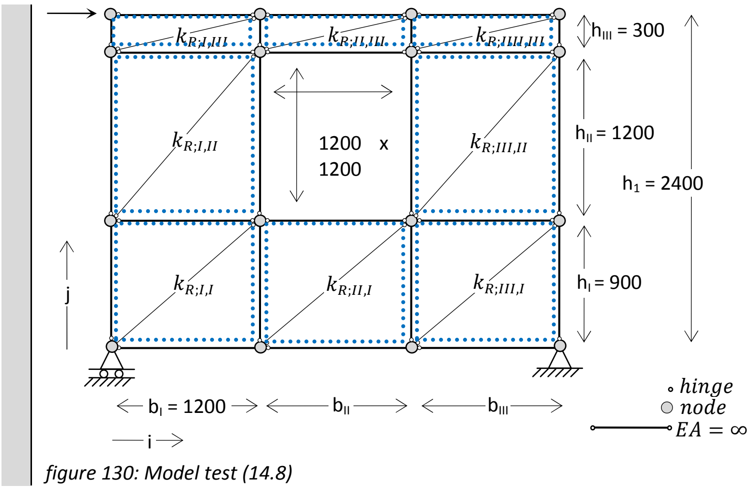


figure 130: Model test (14.8)

In figure 130 a model is shown which represent the shear-wall in test (14.8).

Because the perforated shear-wall in test (14.8) has the same properties as the non-perforated shear-wall in test (14.3), all components in the racking stiffness of the shear-wall remain the same except for the fastener contribution. This will change the stiffness of the diagonals because fastener-slip is taken into account which does not exist, but which is thought to be present along the blue lines. Calculating the brace stiffness for the panels in this way, can be done using the standard equation for the fastener-slip.

Fastener contribution for the panel parts i,I:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{2 \cdot (1 + \frac{h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{2 \cdot (1 + \frac{900}{1200})} \cdot \frac{667,62}{200} = 1144,50\ N/mm$$

Fastener contribution for the panel parts i,II:

$$R_{f,Rd;i,I} = \frac{b_{i,j}}{2 \cdot (1 + \frac{h_{i,j}}{b_{i,j}})} \cdot \frac{K_{ser}}{s} = \frac{1200}{2 \cdot (1 + \frac{1200}{1200})} \cdot \frac{667,62}{200} = 1001,43\ N/mm$$

Fastener contribution for the panel parts i,III:

$$R_{f,Rd;i,I} = \frac{1200}{2 \cdot \left(1 + \frac{300}{1200}\right)} \cdot \frac{667,62}{200} = 1602,29\ N/mm$$

Shear deformation, strain in the studs, compression perpendicular to grain, and strain in the hold-down remain the same. This will result in:

$$R_{Rd;I,I} = R_{Rd;III,I} = \frac{1}{\frac{1}{1144,50} + \frac{1}{20840,63} + \frac{1}{11666,67} + \frac{1}{92888,89}} = 982,12\ N/mm$$

$$\mathrm{k_{R_{Rd};I,I}} = \mathrm{k_{R_{Rd};III,I}} = 982,12 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 1534,56\ \mathrm{N/mm}$$

$$\mathrm{A_{I,I}} = \mathrm{A_{III,I}} = \frac{1534,56 \cdot 1500,00}{9000} = 255,76\ \mathrm{mm^2} = 15,99\ \mathrm{x}\ 15,99\ \mathrm{mm}$$

$$R_{Rd;II,I} = \frac{1}{\frac{1}{1144,50} + \frac{1}{11666,67} + \frac{1}{92888,89}} = 1030,69\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd;II,I}}} = 1030,69 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 1610,45\ \mathrm{N/mm}$$

$$\mathrm{A_{II,I}} = \frac{1610,45 \cdot 1500,00}{9000} = 268,41\ \mathrm{mm}^2 = 16,38 \text{ x } 16,38\ \mathrm{mm}$$

$$R_{Rd;I,II} = R_{Rd;III,II} = \frac{1}{\frac{1}{1001,43} + \frac{1}{8750,00} + \frac{1}{39187,50}} = 878,44\ N/mm$$

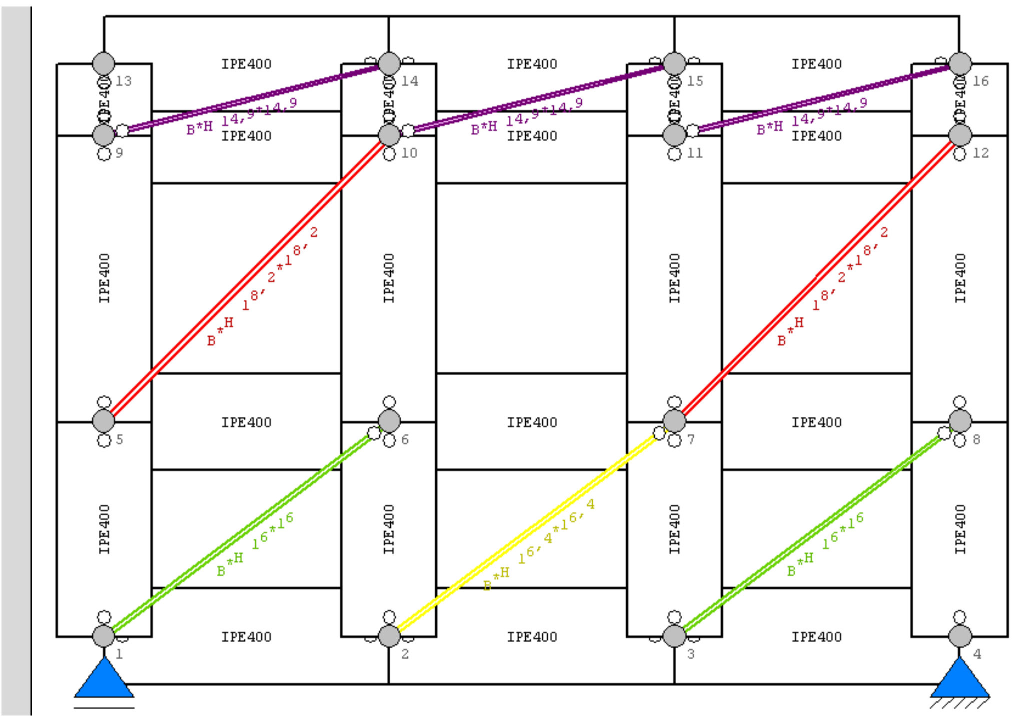
$$\mathrm{k}_{\mathrm{R_{Rd;I,II}}} = \mathrm{k}_{\mathrm{R_{Rd;III,II}}} = 878,44 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 1756,88\ \mathrm{N/mm}$$

$$\mathrm{A_{I,II}} = \mathrm{A_{III,II}} = \frac{1756,88 \cdot 1697,06}{9000} = 331,28\ \mathrm{mm}^2 = 18,20 \text{ x } 18,20\ \mathrm{mm}$$

$$R_{Rd;I,III} = R_{Rd;II,III} = R_{Rd;III,III} = \frac{1}{\frac{1}{1602,29} + \frac{1}{35000,00} + \frac{1}{2508000,00}} = 1531,21\ N/mm$$

$$\mathrm{k}_{\mathrm{R_{Rd;I,III}}} = \mathrm{k}_{\mathrm{R_{Rd;II,III}}} = \mathrm{k}_{\mathrm{R_{Rd;III,III}}} = 1531,21 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 1626,91\ \mathrm{N/mm}$$

$$\mathrm{A_{I,III}} = \mathrm{A_{II,III}} = \mathrm{A_{III,III}} = \frac{1626,91 \cdot 1236,93}{9000} = 223,60\ \mathrm{mm}^2 = 14,95 \text{ x } 14,95\ \mathrm{mm}$$



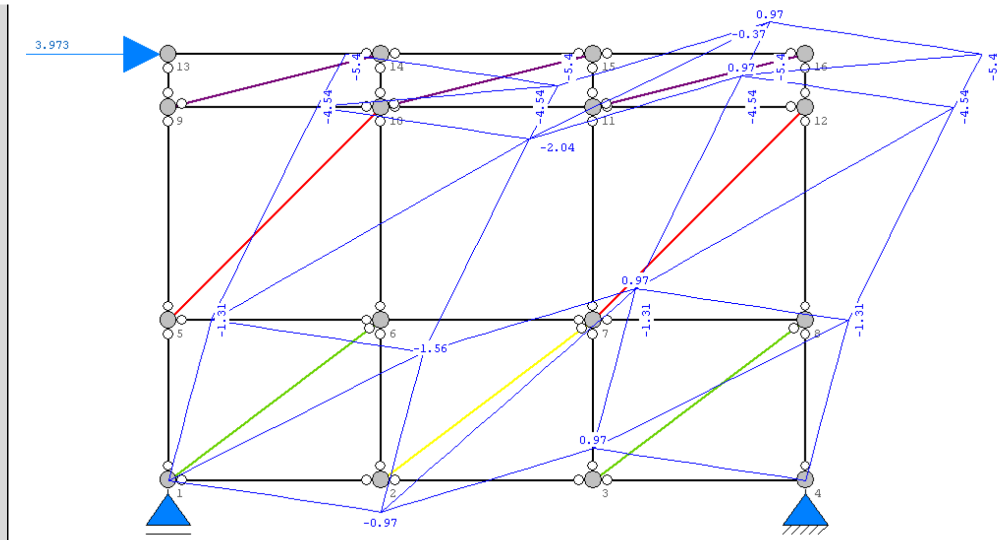


figure 131: Geometry and results of equivalent-brace modelling test (14.8) in framework program.

The multi-panel approach without changing the equation for the fastener component in the timber-frame shear-wall racking stiffness led to the determination of five different brace dimensions (as given above). It can be seen that additional elements in the shear-wall will weaken the shear-wall. The racking stiffness will be lower.

$$R_{\text{multi-panel}} = \frac{3973}{5,4} = 735,74 \text{ N/mm}$$

Test (14.8)	Multi-panel model	Multi- panel model without changing fastener equation	R_{test} = 823,51 (100%)
R_{model} [N/mm]	$\frac{3973}{4,25} = 934,82$ (114%)	$\frac{3973}{4,15} = 735,74$ (89%)	

table 36: Results of modelling approaches test (14.8)

From table 36 can be concluded that using the unmodified equation for the fastener component of the timber-frame shear-wall racking stiffness, as proposed in paragraph 4.8, will lead to a 11% lower racking stiffness in the multi-panel modelling approach. Still a reasonable magnitude is obtained compared with the racking stiffness determined on basis of the load-displacement data found in literature.

Equivalent-brace model (14.3) based on R_{test}

In the equation for the equivalent-brace stiffness the racking stiffness R have to be filled in. In the calculations before, it was chosen to take $R_{analytical}$, as derived with the analytical calculation method presented in chapter 4. It is also possible to use the test-based value for R, R_{test}. In this case also a comparison can be made between the agreement of the panel-area-ratio method and the equivalent-brace model for test (14.8). The shear-wall in test (14.8) is the same as in test (14.3), except for the window space added to the shear-wall in test (14.8).

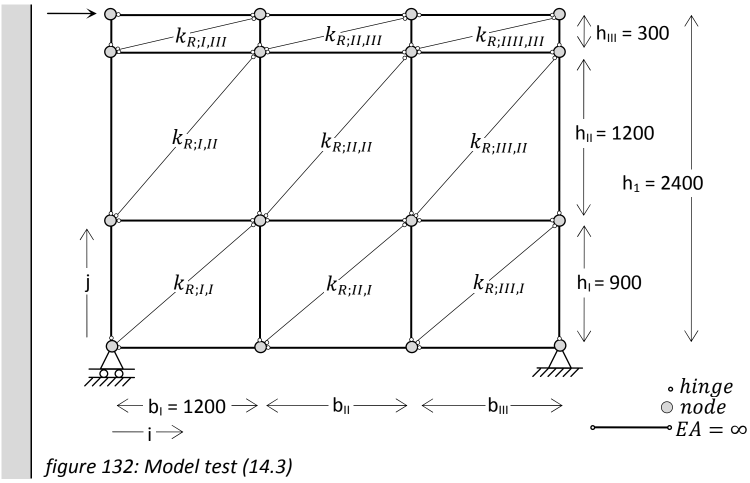


figure 132: Model test (14.3)

$$k_{i,j} = \frac{h_1}{h_i} \cdot \frac{b_i}{b_1} \cdot R \cdot (1 + \frac{h_i^2}{b_i^2})$$

R is the racking stiffness obtained by testing (for reference see Appendix VI) R_{test} (14.3) = 1355,70 N/mm.

<u>Brace I,I, II,I and III,I</u>

$$k_{i,I} = \frac{2400}{900} \cdot \frac{1200}{3600} \cdot 1355{,}70 \cdot \left(1 + \frac{900^2}{1200^2}\right) = 1882{,}92\ N/mm$$

$$\mathrm{A_{i,I}} = \frac{1882{,}92 \cdot 1500{,}00}{9000} = 313{,}82\ \mathrm{mm^2} = 17{,}71\ \mathrm{x}\ 17{,}71\ \mathrm{mm}$$

<u>Brace I,II, II,II and III,II</u>

$$k_{i,II} = \frac{2400}{1200} \cdot \frac{1200}{3600} \cdot 1355{,}70 \cdot \left(1 + \frac{1200^2}{1200^2}\right) = 1807{,}60\ N/mm$$

$$\mathrm{A_{i,II}} = \frac{1807{,}60 \cdot 1697{,}06}{9000} = 340{,}84\ \mathrm{mm^2} = 18{,}46\ \mathrm{x}\ 18{,}46\ \mathrm{mm}$$

<u>Brace I,III, II,III and III,III</u>

$$k_{i,III} = \frac{2400}{300} \cdot \frac{1200}{3600} \cdot 1355{,}70 \cdot \left(1 + \frac{300^2}{1200^2}\right) = 3841{,}15\ N/mm$$

$$\mathrm{A_{i,III}} = \frac{3841{,}15 \cdot 1236{,}93}{9000} = 527{,}92\ \mathrm{mm^2} = 22{,}98\ \mathrm{x}\ 22{,}98\ \mathrm{mm}$$

The equivalent-brace approach led to the determination of three different brace dimensions (as calculated above). The model is loaded with the 40% F_{max} load from the test: F_{04} = 5266,84 N

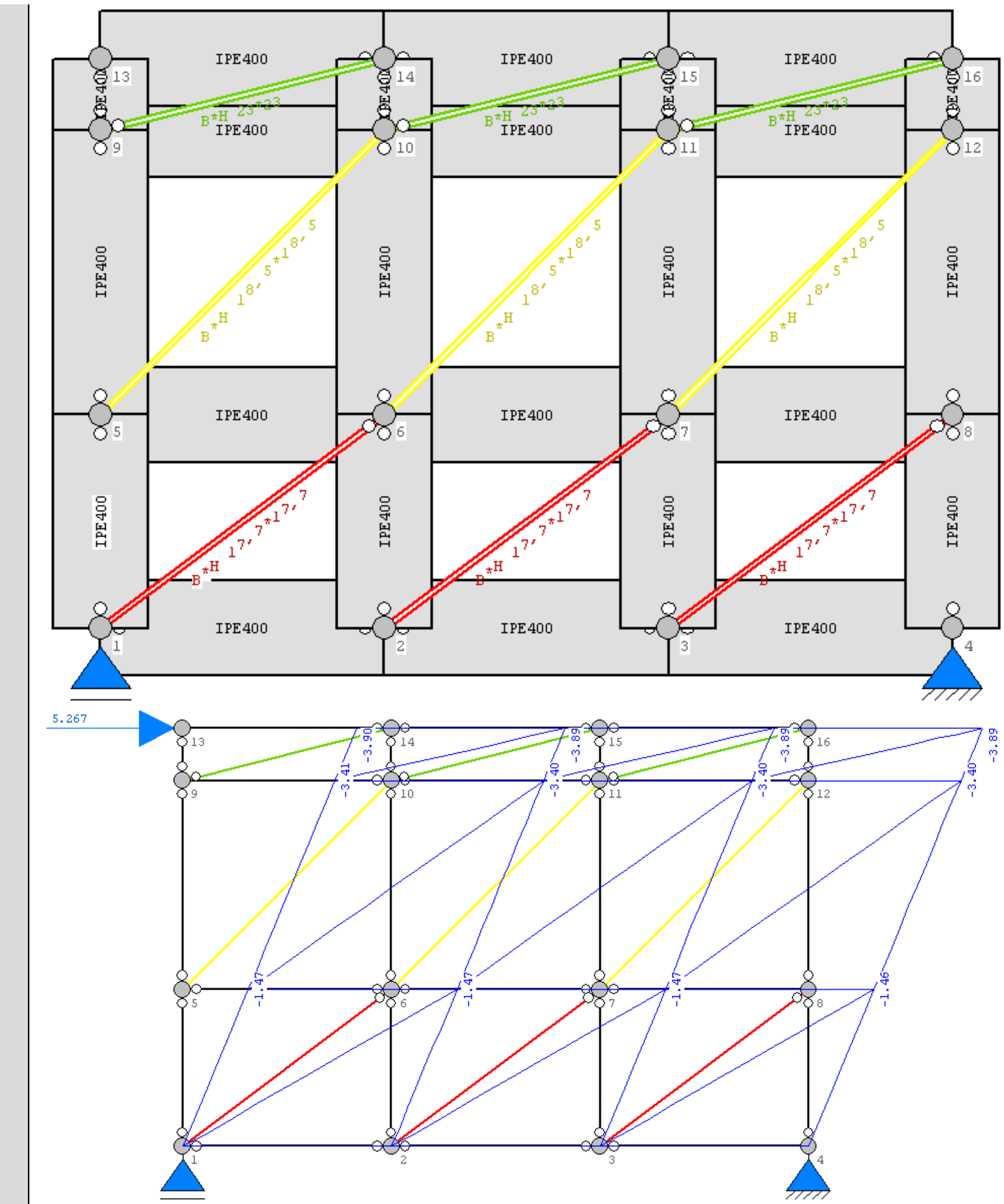


figure 133: Geometry and results of equivalent-brace modelling test (14.3) in framework program.

$$R_{\text{equivalent-brace}} = \frac{5267}{3{,}89} = 1353{,}98\ \text{N/mm}$$

Test (14.3)	Multi-panel model	Equivalent-brace model based on $R_{analytical}$	Equivalent-brace model based on R_{test}	
R_{model} [N/mm]	$\frac{5267}{3{,}16} = 1666{,}77$ (123%)	$\frac{5267}{3{,}67} = 1435{,}15$ (106%)	$\frac{5267}{3{,}89} = 1353{,}98$ (100%)	R_{test} = 1355,70 (100%) $R_{analytical}$ = 1440,66 (106%)

table 37: Results of modelling approaches test (14.3)

The equivalent-brace model of test(14.3) based on R_{test}, as shown above, will be used to create a model of the shear-wall in test (14.8). This will be done in the next paragraph.

Equivalent-brace model (14.8) based on equivalent-brace model (14.3)

The model below is similar as the model shown on the page before. In the model below the brace element on the position of the window space is removed from the model.

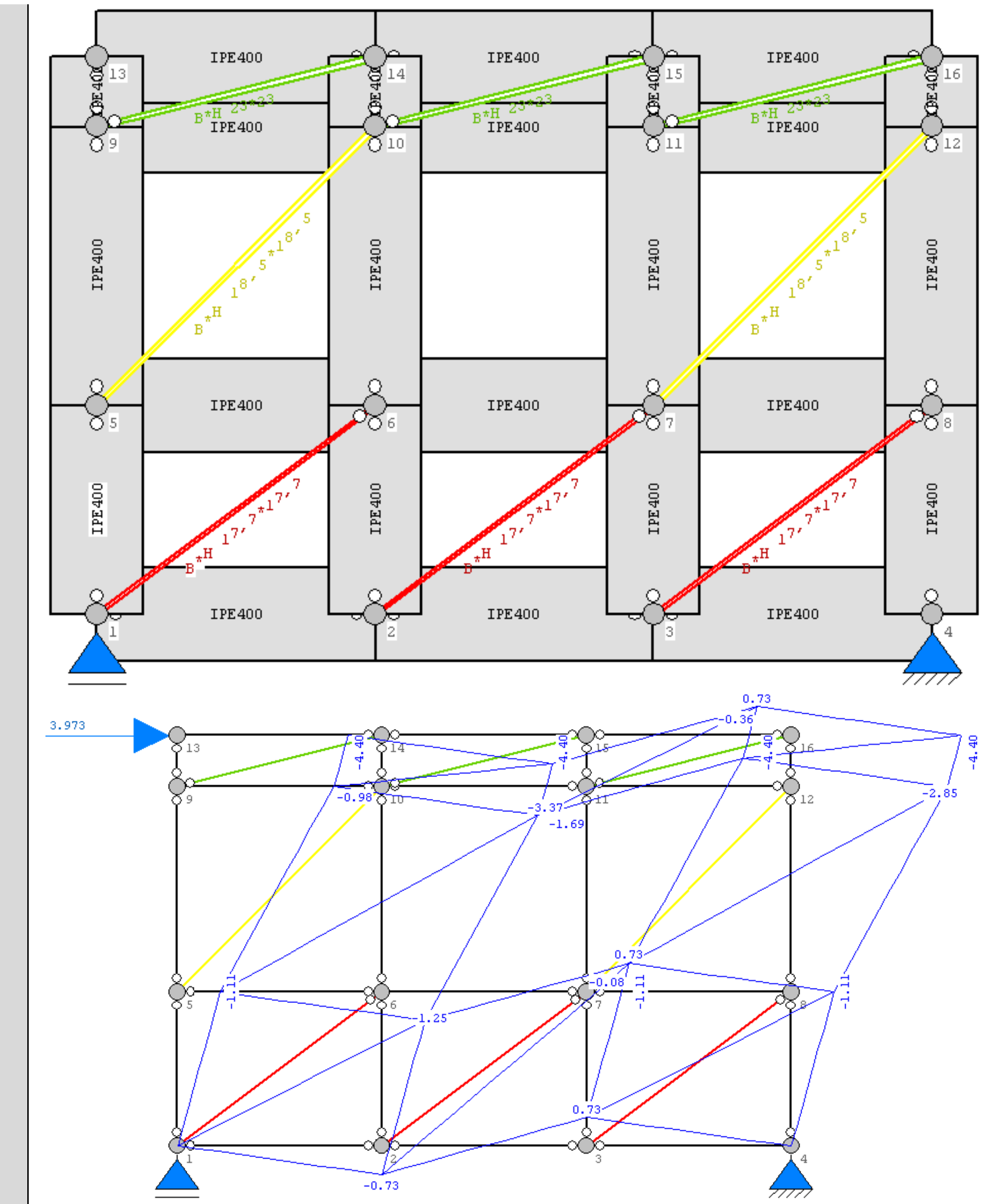


figure 134: Geometry and results of equivalent-brace modelling test (14.8) in framework program.

$$R_{\text{equivalent-brace}} = \frac{3973}{4{,}40} = 902{,}95 \text{ N/mm}$$

Test (14.8)	Multi-panel model	Equivalent-brace model based on $R_{analytical}$ (14.3)	Panel-area-ratio method based on $R_{analytical}$ (14.3)	Equivalent-brace model based on R_{test} (14.3)	Panel-area-ratio method based on R_{test} (14.3)	
R_{model} [N/mm]	$\frac{3973}{4{,}25} = 934{,}82$ (114%)	$\frac{39734{,}15}{4{,}15} = 957{,}35$ (116%)	823 (100%)	$\frac{3973}{4{,}40} = 902{,}95$ (110%)	775 (94%)	R_{test} (14.8) = 823,51 (100%)

table 38: Results of modelling approaches test (14.8)

From table 38 can be seen that the panel-area-ratio method has a deviation of 6% in predicting the racking-stiffness of the perforated timber-frame shear-wall in test (14.8) on basis of the test-based racking-stiffness of non-perforated shear-wall in test (14.3), compared with the test-based racking-stiffness of the perforated shear-wall in tests (14.8). The equivalent-brace method has a deviation of 10%. It can therefore be concluded that, based on the considered tests, both methods are reasonably accurate in predicting the racking stiffness of perforated shear-walls.

MSc Thesis – Modeling and analysis of racking stiffness of timber-frame shear-walls for ultimate and serviceability limit state purposes

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