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ANALYSIS OF STRUCTURAL RESPONSE AND DESIGN METHODS FOR SHEAR WALLS IN LIGHT TIMBER FRAME STRUCTURES

ANALÝZA CHOVÁNÍ A METOD NAVRHOVÁNÍ SMYKOVÝCH STĚN LEHKÝCH DŘEVĚNÝCH KONSTRUKCÍ

DOCTORAL THESIS (SUMMARY)
DISERTAČNÍ PRÁCE (TEZE)

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1 INTRODUCTION

Modern timber frame structures are exceptionally light frames, providing predominantly vertical support to the building. When it comes to horizontal forces mainly generated by wind or as a result of an earthquake in seismically active zones, the slender vertical timbers studs on their own would not be capable of resisting these loads. Historically, the resistance was provided by lining the walls with large format sheathing panels.

***Figure 1.1** – Open panel timber frame [1]*

The methods developed to quantify the minimum extent of the sheathing required for stability range from simplified empiric to somewhat complicated FE models.

Simplified methods, suitable for hand calculations, are either purely empiric, matching particular test programme, or have a sound theoretical background with calibration to tests. Currently, there is a transition from the former to the latter, supported by ongoing research addressing both serviceability and ultimate limit states.

A procedure suitable for a hand calculation, i.e. the calculation is not overly arduous, is currently in high demand by the industry. This is particularly the case where the method would enable the use of partially anchored walls without the need for tie-down devices.

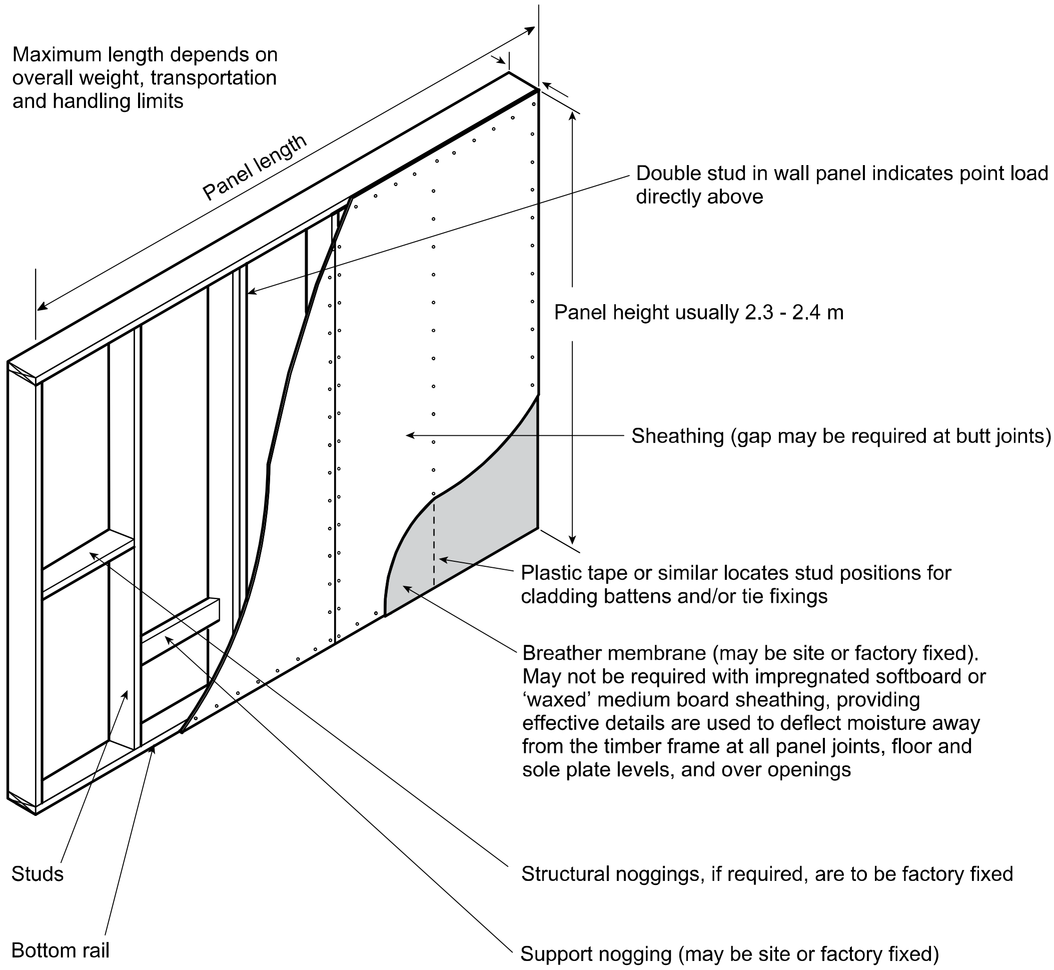


Figure 1.2 *– Typical sheathed wall panel [2]*

From a structural point of view, the timber frame wall diaphragms, also referred to as shear walls or racking walls, constitute complex, nonlinear, structural systems with a high level of uncertainty. In the design of the shear walls, engineers must address many aspects of the wall arrangement and consider material parameters of heterogeneous nature, noticeable manufacturing tolerances, and variations in product assembly. These attributes are the primary source of discrepancy between the analytical models and test results. Differences as substantial as 20-30% are considered to be a good agreement. Noting the complexity of the task, the simplified methods, proposed mainly in Europe, are seen as more practicable than detailed FE modelling.

Research papers presented simplified methods based on sound elastic and plastic models, both amenable to consistent prediction of the structural behaviour of partially anchored shear walls. Some of these methods, however, still demand detailed knowledge or assumptions of the exact size and placement of the sheathing panels. The methods available today do not easily allow for door and window openings in the wall.

Another important aspect is the arrangement of the shear walls within the structure. Noticeably, these methods are formulated for the horizontal load being applied to the top corner of the leading edge of the wall. The actual arrangement differs from this premise and, especially for partially anchored walls, leads to iterative calculations when designing multi-storey buildings.

The presented work comprises a derivation of a new calculation method for shear walls with offset force lever arm, and an investigation into the influence of framing connections and gaps in sheathing panels in the overall shear resistance of the walls.

Summary:

- Section 4 – *Offset force method* outlines the original method for predicting shear wall resistance at the ultimate limit state level. The Offset force method is benchmarked using a parametric study and numerical analysis against selected simplified methods.
- The experimental programme is outlined in Section 5 – *Influence of framing joints & gaps between sheathing panels* and was designed to allow the optimisation of the shear wall panel design concerning material savings and enables full utilisation of production automation and machining.

2 RELATED WORK

2.1 OVERVIEW AND SUMMARY OF THE METHODS

The discussion below draws on the most critical features of the methods and, where appropriate, the review is based on the force distributions shown in Figure 2.1 and Figure 2.2.

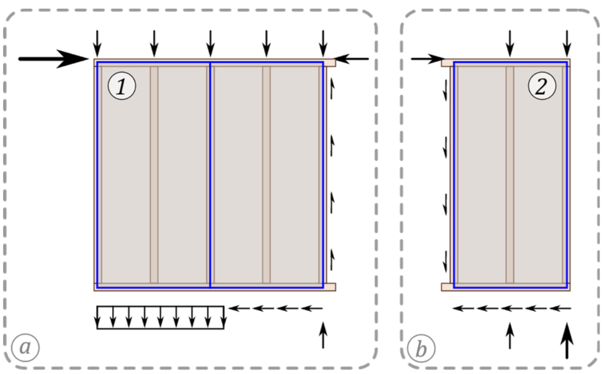


***Figure 2.1** – Källsner & Girhammar, Model 1 – force distribution*

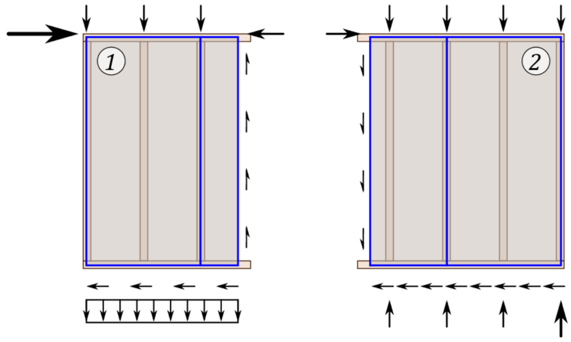


Figure 2.2 *– Källsner & Girhammar, Model 2 – force distribution*

Model 1 of Källsner & Girhammar

Model 1 is a plastic lower-bound method based on the force distribution shown in Figure 2.1, fulfilling force equilibrium in each theoretical wall segment [3, 4]. The split between theoretical segments is always at a stud location. Further work uses this model as a benchmark to compare and contrast the behaviour of other models.

Model 2 of Källsner & Girhammar

The second model [5, 4], depicted in Figure 2.2, attains the full vertical shear capacity of the wall as close to the leading stud as possible, thereby maximising the length on the fully anchored second segment. A minor drawback of the model is that the horizontal equilibrium of the leading segment of the wall is not always satisfied. This is overcome by taking advantage of alternative load paths and direct contact between sheathing panels. The main advantage of this model over the exact lower bound Model 1 is that the length of the leading segment can be calculated directly. The proposed offset force method assimilates this model as a base and a starting point for derivative work.

Method A from Eurocode 5

The force distribution of Method A [6] is identical to the Figure 2.1 part (b) and assumes full plastic shear flow along the perimeter of the sheathing panel. The main drawback of the codified version is the prescribed use of an anchoring device on a leading edge. For the numerical comparison, the strict requirement of a tie-down device was relaxed and replaced with anchorage by a vertical load acting on the wall. Vertical anchorage utilising the fixing capacity of the sheathing-to-framing fasteners along the wall base was not assumed. Further work adopts this model to compare and contrast other models.

Method PD 6693-1 applied in the UK

This method was presented in the United Kingdom [7] and introduces a reduction factor for openings. Figure 2.1 part (a) shows the assumed force distribution. The mechanical model is identical to the one adopted by Canadian standard CSA 086-01 [8].

Method from CSA 086-01 [8]

This model is deemed to be already represented by the PD 6693-1 method.

Danish method by TRÆ [9]

The model assumes a plastic behaviour of sheathing-to-framing fasteners, that are loaded in a general direction with respect to the panel edge. The method was initially developed for unidirectional support of sheathing panels but can be applied to panels supported on all four edges.

NZS 3603 [10]

The model is primarily based on limiting the maximum deflection, with a strong dependency on adjustment factors to the various components of the horizontal displacement. These factors were derived to reflect local building practice and materials.

3 AIM OF THE WORK

The aim of this work is twofold. Firstly, it became apparent that current simplified methods do not cover the most common loading arrangement with horizontal force applied at an elevated location above the wall-head height. After review and identification of a suitable base method, a new calculation procedure for predicting shear resistance of partially anchored walls is derived and presented. The parametric analysis of the proposed method captures its behaviour and compares it with established procedures and experimental results from the literature.

Secondly, automated manufacturing lines provide a limited facility for robotic joining of the framing members. Their capability does not allow for installing angle brackets and rarely supports the insertion of screws from the inside of the framing. Additionally, the sheathing panel producers stipulate a movement gap between individual panels. The experimental programme makes investigations into the effects of such manufacturing stipulations on the wall panel shear capacity.

4 OFFSET FORCE METHOD

4.1 AIM

There are many simplified methods allowing a hand calculation of capacity of fully and partially anchored light timber frame shear walls. However, these methods are formulated for a particular loading arrangement where the horizontal load is applied directly to the top corner of the wall leading edge, and where additional vertical loading is distributed along the top edge.

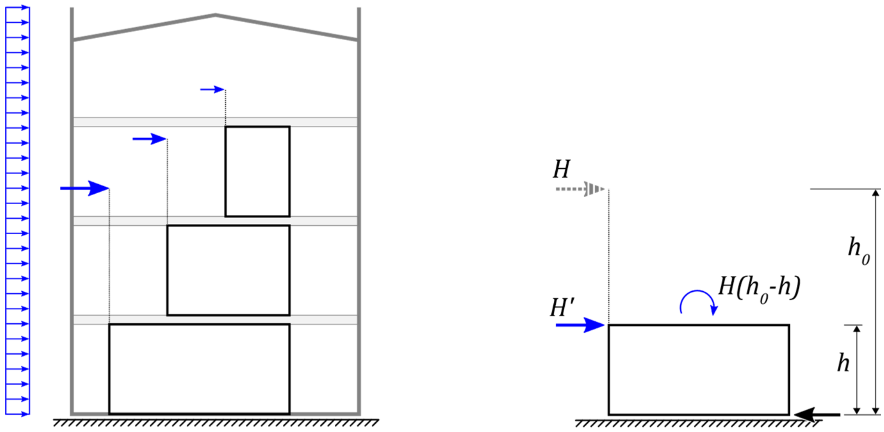


***Figure 4.1** – Shear walls in multi-storey buildings, resultant horizontal loads*

As shown in Figure 4.1, the actual arrangement differs from the premise of those models, and such formulation for partially anchored walls leads to iterative calculations in the design of multi-storey buildings. The resultant of the shear force H is typically offset above the head of the wall and the actual lever arm h_0 needs to be determined individually for each wall. Consequently, an additional overturning moment is generated as a result of the offset loading. In accounting for the effects of this overturning moment and the associated adjustment of the vertical stabilising loads, existing methods demand an iterative approach.

The proposed method allows the bypassing of this iterative process by taking advantage of the fact that the point of application of the resultant shear force, i.e. the force height above the wall base, is known in advance. Naturally, a continuous load path through the structure above such a wall must exist and be capable of transferring the shear force and associated bending moment. The presented model remains consistent with the results of the original underlying method when the offset of the force coincides with the wall height.

Providing the fixings and the vertical stabilising loads for each wall are known or given from the outset, the proposed model enhances the ease of use of the original method [5, 4]. It is achieved by decoupling the wall height from the actual point of application of the horizontal load. As a result, it enables a direct calculation of the maximum shear wall capacity, having already taken into account the actual shear force lever arm. The constraints of the original method, however, still apply in the general case. That is, an iterative process to establish the maximum stabilising load for walls with a series of point loads or tie-downs, is still required. This iterative calculation can be eradicated by conveniently converting the distinct point loads into a single leading point load and a uniformly distributed load.

4.2 THEORETICAL MODEL

This new formulation for partially anchored walls separates the location of the horizontal force from the wall height and explicitly allows for a uniformly distributed vertical load. Such provisions closely correspond to real-life design situations where the resultant horizontal load is often effectively elevated above the wall height, and series of point loads are frequently adopted as uniformly distributed load.

4.3 LIMITATIONS

1. The model is applicable to wall diaphragms where the sheathing-to-framing connection is utilised by dowel type fasteners and only if this connection exhibits plastic behaviour.
2. Reverse and dynamic loading, including earthquake design, has not been considered while deriving the model.
3. Owing to the adopted uncontrolled plastic behaviour of the fasteners, the model is not suitable for calculation of displacements.

4.4 LOAD TRANSFER WITHIN THE WALL DIAPHRAGM

The shear wall diaphragm, shown in Figure 4.2, is split into three fictitious segments. Determining the length of each segment is central to the method. The theoretical division into segments depends on the actual loading arrangement and anchorage capacity. The division can lead to a combination of all three, any two, or any single segment representing the shear wall.

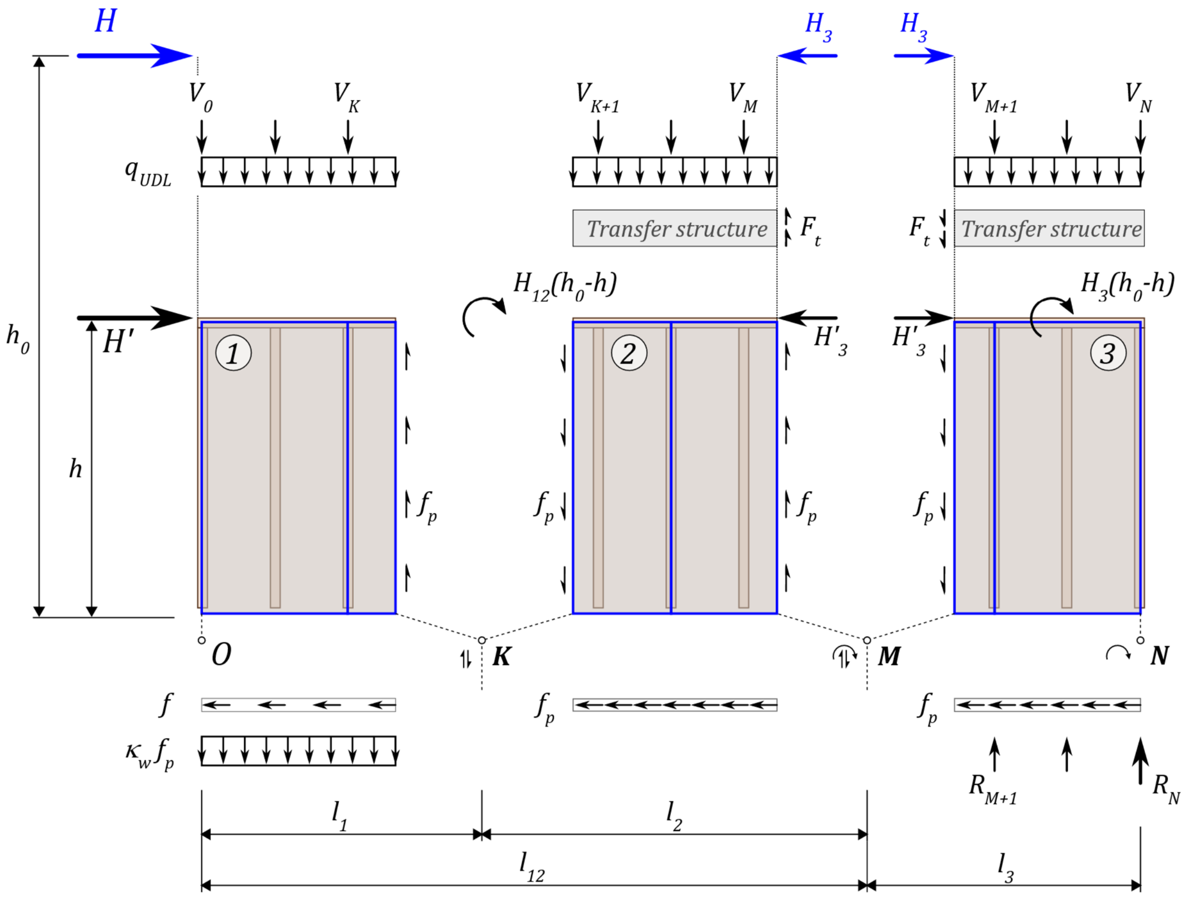


***Figure 4.2** – Force distribution within the wall*

Segment 1

This segment is fully active in anchoring the diaphragm to the base by vertical fixing capacity of the sheathing-to-framing fasteners along the bottom rail and the vertical forces acting on top of the wall. Any vertical load applied to the wall is assumed to be transferred via studs into the sheathing panels.

Segment 2

This part of the wall extends the stabilising lever arm of the segment 1 while providing a full anchorage to segment 3 with respect to the load H_3 on the lever arm h_0 as shown in Figure 4.2. Since the spare vertical shear capacity in the sheathing fasteners is limited, no additional vertical load is transferred by the sheathing panels of segment 2. Therefore, this segment does not contribute to the tie-down anchorage. It is assumed that the structure above the examined wall diaphragm is capable of transferring all vertical loads between points K and M, and further on to the first stud in segment 3. Segments 1 and 2 rotate together as a rigid body about point M.

Segment 3

The remainder of the wall diaphragm is considered fully anchored in respect of the uplift caused by the horizontal force H_3 applied at the offset lever arm h_0.

4.5 SIMPLIFIED LOADING ARRANGEMENT

Further simplification of the equations above is possible for an arrangement shown in Figure 4.3, where the stabilising loads are restricted to a single leading point load and a uniformly distributed load along the full length of the shear wall.

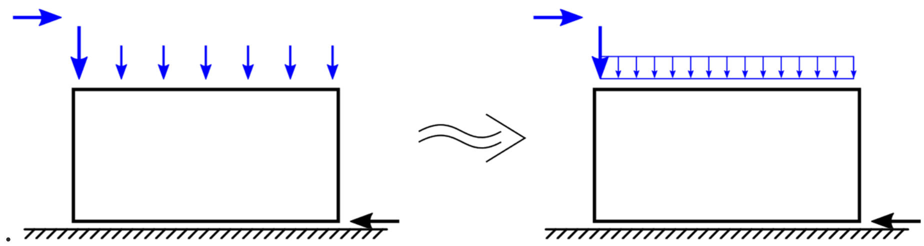


Figure 4.3 *– Approximation of vertical loads*

For this specific arrangement the following explicit formulae apply:

Length l_1 of the first segment is expressed in (4.1):

$$l_1 = \frac{h\, f_p - V_0}{\kappa_w\, f_p + q_{UDL}} \tag{4.1}$$

The rigid body action extends to length l_{12} as (4.2):

$$l_{12} = \frac{(h_0 - h)\kappa_w\, f_p{}^2 + q_{UDL}(h_0 f_p - V_0)}{q_{UDL}(\kappa_w\, f_p + q_{UDL})} \tag{4.2}$$

The total capacity H_{max} is (4.3):

$$H_{max} = \frac{2V_0 + q_{UDL}\, l_{12}}{2h_0} l_{12} + \frac{\kappa_w\, f_p}{2h_0}(2l_{12} - l_1)l_1 + f_p\, l_3 \tag{4.3}$$

4.6 PARAMETRIC ANALYSIS AND NUMERICAL VALIDATION

4.6.1 Comparison with a reference dataset

The methods are referenced in the charts as follows:

EC5_mA	is Method A from BS EN 1995-1-1. However, vertical loads were permitted as a replacement for a tie-down at the leading edge;
Kall_Gir_m1	is the Model 1 by Källsner & Girhammar;
Kall_Gir_m2	is the Model 2 by Källsner & Girhammar;
PD6693_mod	is the PD 6693-1 disregarding fastener improvement factor;
Offset_force	is the derived method presented in Section 4 Offset force method;
Flow_sht	refers to the fictitious situation when the sheathing-to-framing fasteners act in plastic shear along the whole length of the wall diaphragm. The capacity corresponds to fully anchored diaphragm at the leading edge and effectively defines the maximum achievable shear resistance;
Experimental	is the dataset published in [4].

The influence of variable lead and distributed discrete loads

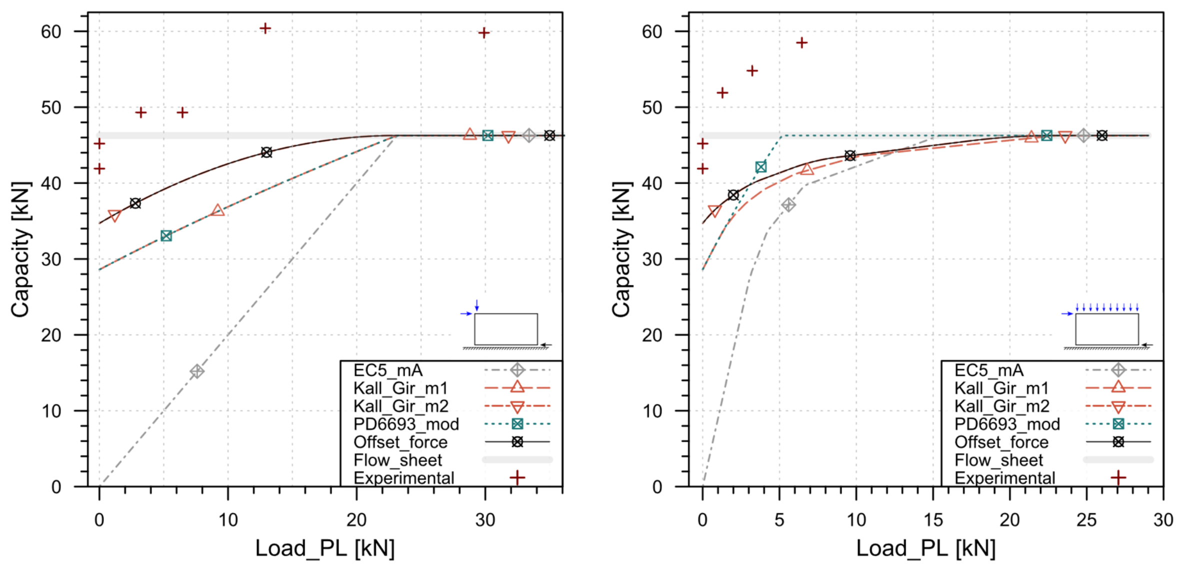


***Figure 4.4** – Variable vertical loading, wall length 4.8 m*

4.6.2 Comparison with reference ad-hoc testing

References for the methods remain as for previous studies, except for the source of experimental data, which comes from [11].

Wall without stabilising load and with horizontal load at an offset position

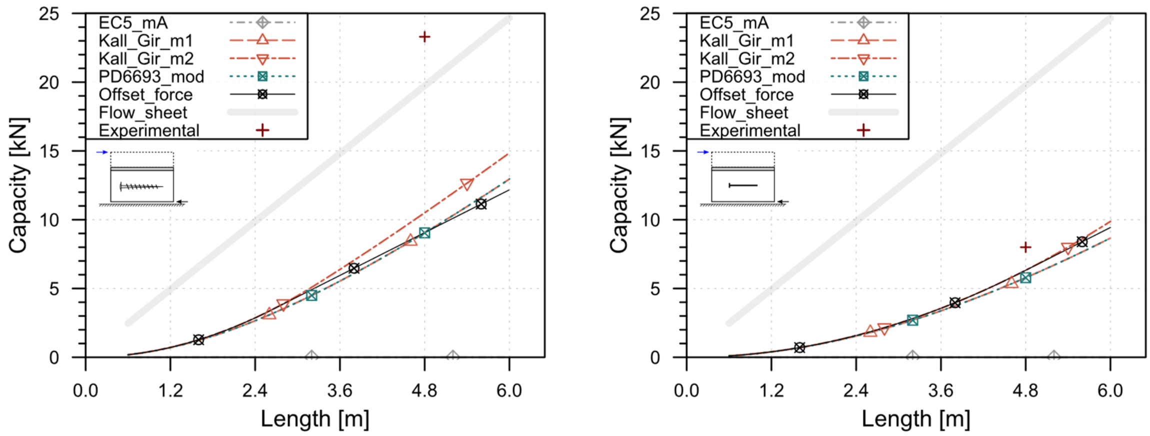


Figure 4.5 – *Offset horizontal load without stabilising vertical loading*

Wall with stabilising discrete point loads and horizontal load at an offset position

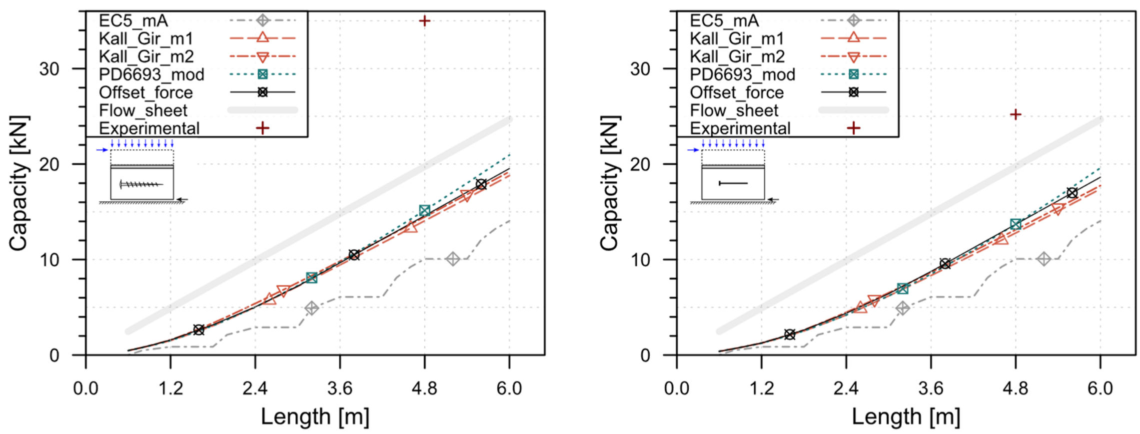


Figure 4.6 – *Offset horizontal load with stabilising vertical loading*

4.6.3 Parametric evaluation for large lever arms

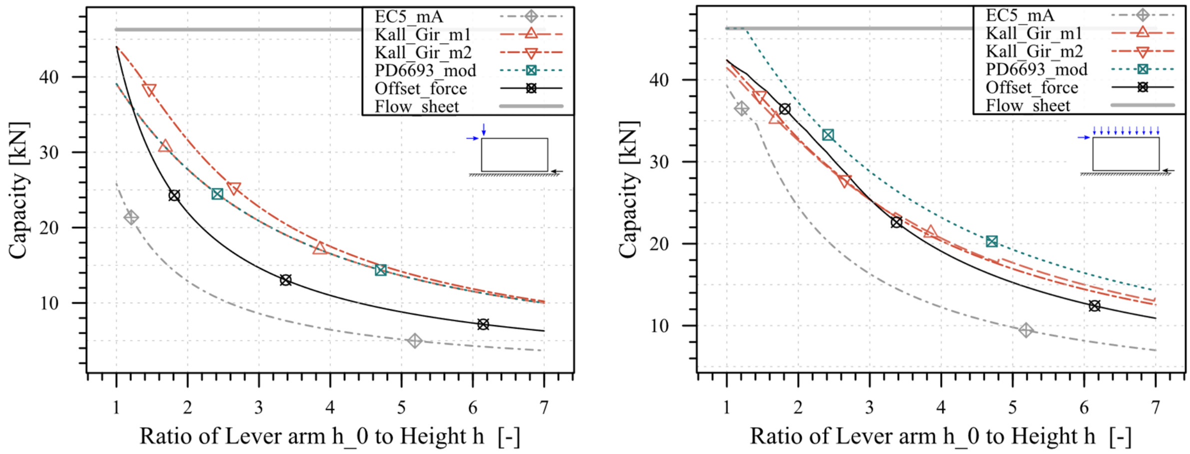


Figure 4.7 *– Variable offset force lever arm, wall length 4.8 m*

The side by side comparison above shows the performance of various methods when applied to a 4.8 m long wall subjected to either a lead point load or a set of distributed point loads.

4.7 DISCUSSION & CONCLUSIONS

The offset force method presented here enables a streamlined calculation of the resistance of shear walls in multi-storey buildings. Its performance is studied and compared with both previously available codified and theoretical methods. By using knowledge of the building’s geometry, the proposed method eliminates the need for the iterative process of redistribution of the overall shear force to the individual walls currently required by other procedures.

All arrangements considered in the study demonstrate that the proposed model not only mirrors the behaviour of the robust lower-bound plastic method [5, 4] but also delivers consistently conservative predictions of the shear wall capacity compared with available test data.

5 INFLUENCE OF FRAMING JOINTS & GAPS BETWEEN SHEATHING PANELS

5.1 AIM

High-capacity production lines from Weinmann, Randek, JJ Smith (H&M), and others offer only a limited capability for automated joining. Nailing or screwing perpendicular to a surface member often being the only option. Installation of bracketry is currently a fully manual operation. It is, therefore, clear that a departure from the automatic capabilities has a knock-on effect on factory control and quality assurance, causes manufacturing reliability issues, and reduces the overall throughput, thus having a significant impact on costs.

5.2 INVESTIGATED ARRANGEMENTS

The total number of proposed tests was 12, comprising four different arrangements.

Specimen make-up

The manufactured dimensions of the wall framing were 2515 mm wide and 2510 mm high, comprising 5 No. 40 mm × 100 mm studs, and top and bottom 40 mm × 100 mm rails. All grade C24. The framing of nine samples was connected with 2 No. ⌀3.1×90 mm annular ring-shanked nails driven perpendicular to the rails into the end grain of the stud. Framing of a further three specimens was utilised using angle brackets, each fixed with 2 × 8 No ⌀4.1×40 mm annular ring-shanked nails. All samples were sheathed with 2 No. 2500 mm × 1250 mm × 8 mm OSB/3. And the sheathing-to-framing connections utilised ⌀2.80×60 mm annular ring-shanked nails spaced at 150 mm centres. The connection to the 100 mm × 160 mm C24 base was effected by a series of threaded rods M16 Grade 8.8 at approximately 625 mm.

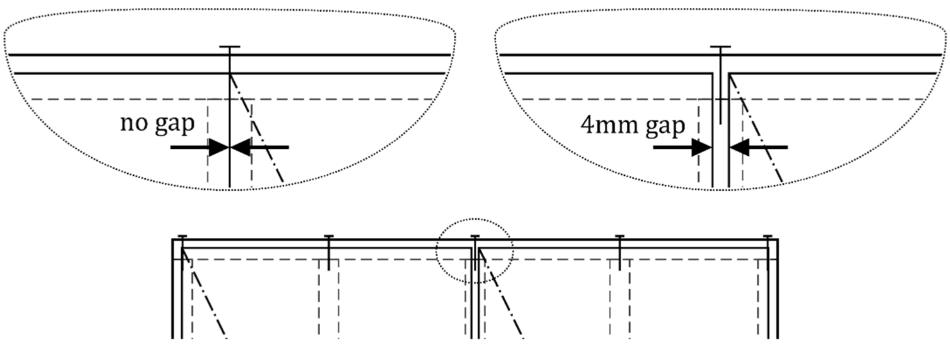


Figure 5.1 *– Gap between the sheathing panels*

Instrumentation and loading

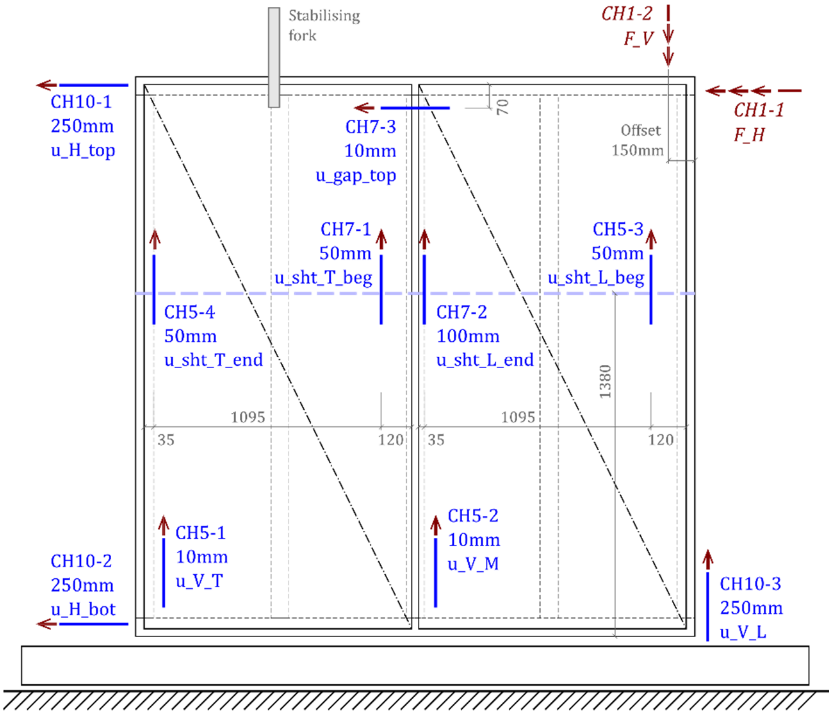


Figure 5.2 *– Apparatus and input channel nomenclature*

The horizontal load was applied into a welded load spreader, the vertical load was applied onto the set of lubricated rollers and spreader plates. The deformations were measured using string potentiometers and differential inductive displacement transducers.

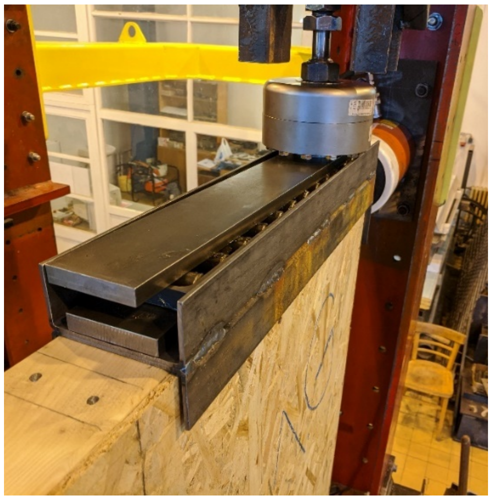
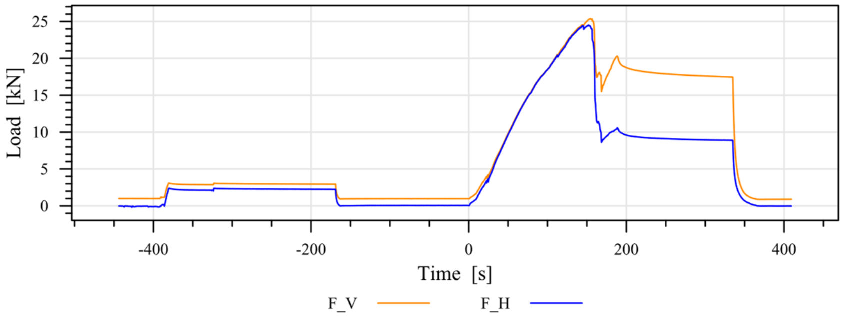


Figure 5.3 *– Application of loading*

Test procedure

The test procedure loosely followed the intent of the superseded EN 594 [12] and current ASTM E564 [13] while utilising a preload cycle.



Key: F_H indicates horizontal force, F_V is vertical restraining load

***Figure 5.4** – Representative load-time diagram*

5.3 OBSERVATIONS

Fully anchored walls

Increase in load cause in the leading bottom and the trailing top corners of each sheathing panel to crack off, conforming to the linear elastic distribution of stresses with peaks in the corner fasteners.

***Figure 5.5** – Failure of the corner fasteners*

When the specimens reached their maximum capacity, all the nails in horizontal chords failed by unzipping from the sheathing panel, see Figure 5.6. and the walls developed a typical shape with a gentle sinusoidal wave-like curvature in the top rail.

__Figure 5.6__ – Groups fastener failure (unzipping) of nails in the top rail

Partially anchored walls

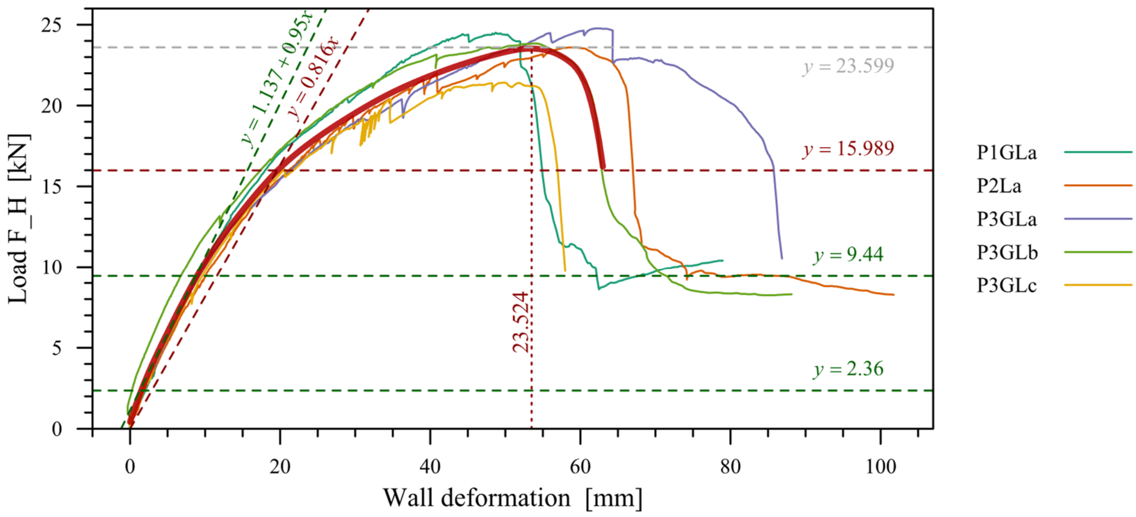
Similarly to fully anchored walls, the sheathing panels started to rotate independently of each other, which was exhibited by a vertical movement between the panels in the panel joint. Once the load started to increase, the bottom corner of the leading sheathing panels cracked off.

As the specimens reached their maximum capacity, all the nails in the bottom rail gradually failed by unzipping from the sheathing panel, see Figure 5.7.

__Figure 5.7__ – Progressive failure of sheathing-to-framing fasteners in bottom rail

Overall, the specimens behaved much more like a rigid body, which was expected as the capacity of the vertical joint between sheathing and framing exceeded the anchorage capacity of the sheathing-to-framing nails along the bottom rail.

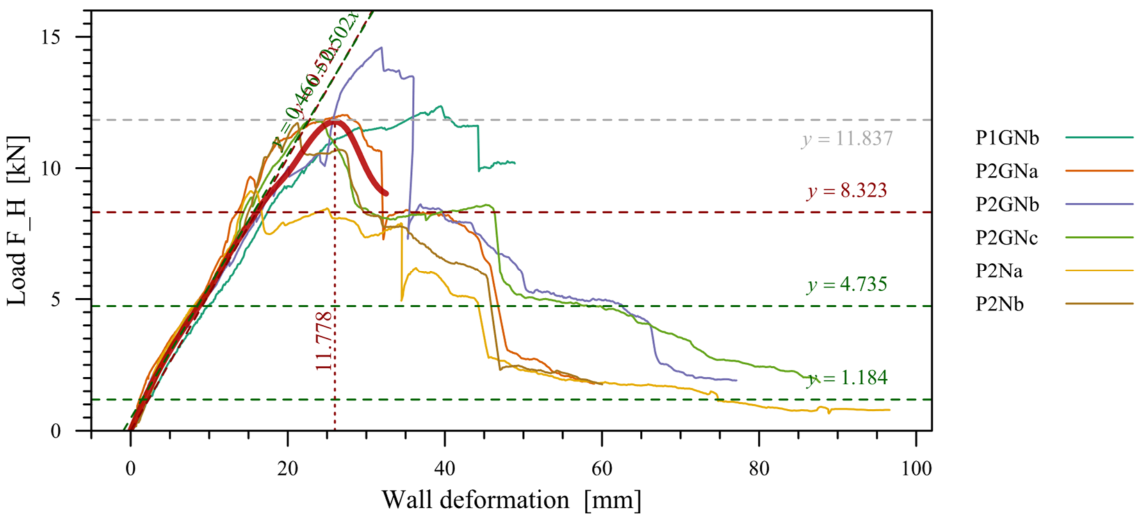
5.4 MAXIMUM CAPACITIES FOR LEADING STUD ANCHORAGE



Stiffness based on regression curve: y=0.95 x + 1.137; adj. R^2=0.998; k_ser=0.95; k_u=0.816

***Figure 5.8** – Leading stud anchorage, load-displacement diagram*

5.5 ADJUSTED MAXIMUM CAPACITIES FOR BOTTOM RAIL ANCHORAGE



Stiffness based on regression curve: y=0.502 x + 0.466; adj. R^2=0.997; k_ser=0.502; k_u=0.52

***Figure 5.9** – Bottom rail anchorage, adjusted load-displacement diagram*

5.6 ROTATION OF SHEATHING PANELS

The rate of rotation of the sheathing panels is presented as a ratio between the rotation of the leading to that of the trailing panel. This allows a straight-forward comparison between the rotation rates.

Partially anchored walls

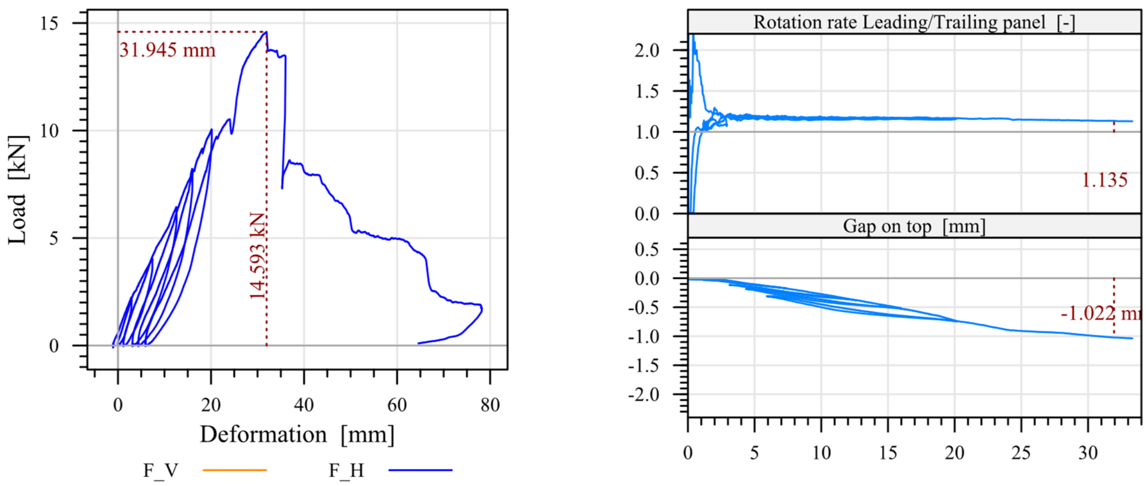


__Figure 5.10__ – Load, rate of rotation and gap changes

Fully anchored walls at a leading stud

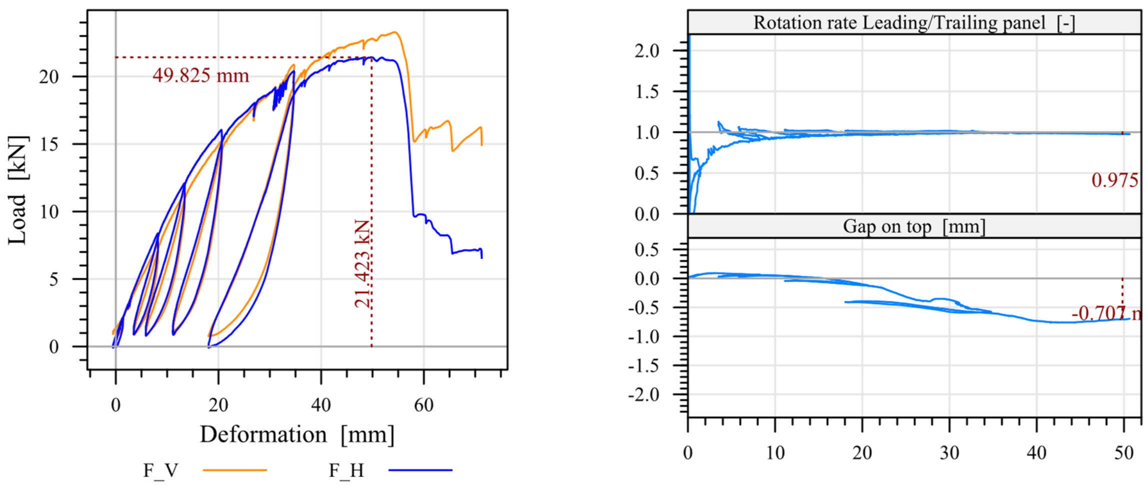


__Figure 5.11__ – Load, rate of rotation and gap changes

5.7 DISCUSSION OF RESULTS

Hypothesis 1

The framing connections do not make a significant difference to the shear capacity of fully anchored walls.

The overall review in Figure 5.12 shows that two results are consistent with the partially and fully anchored mean values, whereas the third result is an outlier of the partially anchored group. The evidence from the test results is insufficient to confirm or deny the hypothesis.

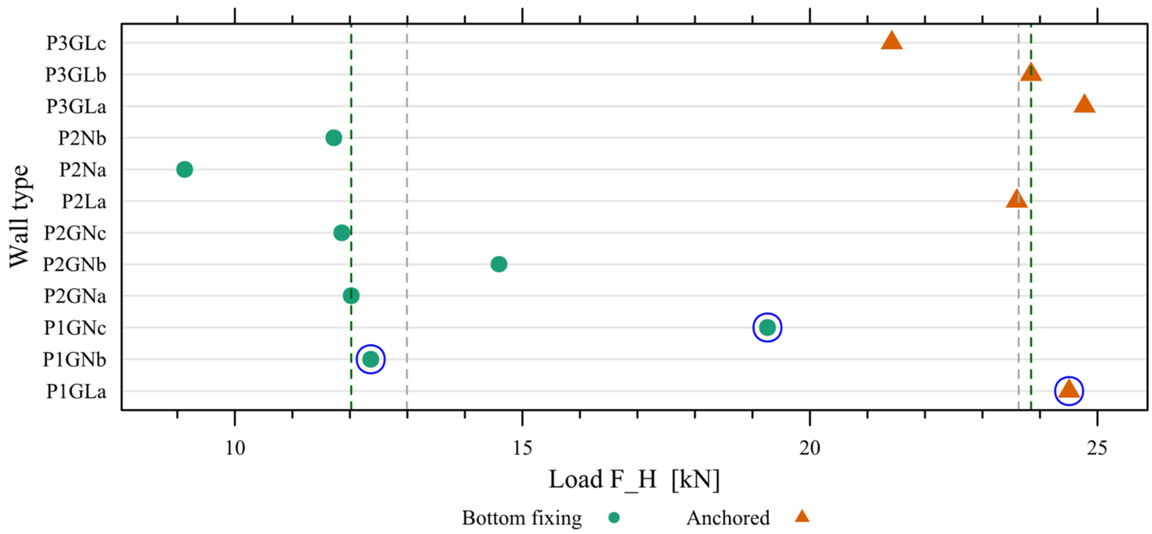


Figure 5.12 *– Horizontal capacity in relation to framing with angle brackets*

Hypothesis 2

The individual sheathing panels would rotate at the same rate for fully anchored walls. Sheathing panels of partially anchored walls would rotate at different rates.

The analysis shows that the gaps between the sheets ultimately tend to close for both setups and that the leading sheathing panels rotate more than the trailing panels for partially anchored walls.

6 CONCLUSIONS, DISCUSSIONS & SUMMARY

Lateral resistance of timber frame structures is typically provided by lining the walls with large-format sheathing panels. In order to quantify the extent of required sheathing, a number of simplified design procedures are available. The simplified solutions range from empirical and semi-empirical methods, treating the geometry of the whole wall, to the shear field theory, requiring detailed knowledge of the framing, and sheathing panel sizes and arrangement.

These simplified models formulate the problem for the horizontal loading applied at the wall height, more specifically, the top corner of the leading edge of the shear wall. The calculation of the shear resistance, when applying these models to a multi-storey building, requires an iterative process of increasing lateral load and stabilising the wall. That is, balancing the overturning moment caused by the horizontal load with the stabilising actions of combined vertical loads and anchorage.

The presented work comprises the derivation of a new calculation method for shear walls with an offset force lever arm. Furthermore, the text documents an experimental investigation into the influence of framing connections and gaps in sheathing panels on the overall shear resistance of the walls.

The Offset force method delivers consistent results without any steps or discontinuities in the performance charts. The prediction accuracy is good. However, in comparison with the iterative approach, it tends to deteriorate with an increasing lever arm of the applied shear force. In comparison with currently available techniques, the marginally lower calculated capacity is traded for a significant reduction in complexity. The method can be used as a stand-alone procedure or to provide an improved first estimate for the iterative calculations.

The experimental programme highlights the trade-offs involved in design for manufacturing, where the machine equipment drives the choice of joinery, and the mass-production sector opts for framing connections utilised by nails driven through one timber element into the end grain of the adjoining member. Although not influencing the structural response on the overall scale, the connections alter the rate of rotation of individual sheathing panels attached to the frame for partially anchored walls.

The experimentally verified performance of walls with gaps between sheathing panels suggests that the difference in the overall capacity and stiffness is insignificant. A wall with gaps between sheathing panels performs the same as a wall arranged without gaps in the sheathing. The gaps between the panels ultimately tend to close for both partially and fully anchored walls.

The sheathing to framing nails in fully anchored walls fail around the whole perimeter of the sheathing panels, whereas the nail failure is localised to the bottom rail in partially anchored walls. Lastly, the partially anchored walls had lower capacity but exhibited an extended linear relationship between applied force and displacement, whereas walls anchored by the leading stud had twice the capacity but strong nonlinear force-displacement diagrams.

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ABSTRAKT

Hlavním cílem disertační práce je stanovení výztužné kapacity smykových stěn dřevěných konstrukcí vícepodlažních budov. V současnosti používané metody jsou odvozené pro přenos vodorovné síly, jejíž působiště je totožné s výškou stěny. Tento předpoklad nicméně zanedbává skutečnost, že kapacita smykových stěn může mimo jiné záviset i na vzdálenosti působící horizontální síly od horní hrany stěny. Pozice výslednice vnějších vodorovných sil je v praxi zpravidla ve svislém směru odsazena nad horní hranu stěny a je tedy třeba uvážit nepříznivý vliv kroutícího momentu, který stěnu destabilizuje. Řešení tohoto problému pro částečně vetknuté smykové stěny proto vede k aplikaci iteračního výpočtu.

Dalším z cílů práce bylo vytvořit postup výpočtu smykových stěn pro vícepodlažní budovy, který by objektivněji odpovídal reálnému působení uvažovaných konstrukcí ve srovnání se stávajícími zjednodušenými metodami. Nový výpočtový model s dolním odhadem smykové kapacity, odvozený na základě plastické metody, byl úspěšně odzkoušen v parametrické studii a porovnáván s dosaženými výsledky dostupných laboratorních experimentů. Následná analýza prokázala, že smyková kapacita výztužné stěny stanovená pomocí nové metody, je srovnatelná s výsledky získanými za použití tradičních postupů s iteračním procesem a díky snadné aplikaci může být tato metoda přínosnou alternativou pro technickou praxi. Součástí práce byla realizace a vyhodnocení experimentálního programu na panelech skutečné velikosti, kde byl analyzován vliv mezer (spar) mezi panely opláštění, jak je doporučováno jejich výrobci. Nastavení experimentálního programu umožnilo ověřit rozdíl ve smykové kapacitě sestav s mezerami a bez mezer mezi panely opláštění.

Závěry vyplývající z vyhodnocení potvrdily vhodnost nového výpočtového modelu výztužné kapacity stěny a také, že mezery prováděné v praxi mezi jednotlivými panely opláštění nemají podstatný vliv na výslednou únosnost a tuhost plně ani částečně vetknutých stěn.

ABSTRACT

The prediction of shear capacity of light timber frame walls in a multi-storey arrangement is the main focus of this dissertation. The available theories neglect to account for the fact that the shear resistance of the walls may depend on the actual vertical position of the applied horizontal force. However, the actual arrangement of the structures in practice introduces a vertical offset between the wall head-height and the position of the resultant of the external horizontal forces. Thus, the horizontal shear force is accompanied by dependent overturning moment. Solving such a problem for partially anchored walls inevitably leads to an iterative calculation.

The aim is to provide a comprehensible and less calculation-intensive procedure for multi-storey buildings that would be competitive with existing simplified methods. A model derived from lower bound plastic method was successfully put to the test in a parametric study and compared with limited test results. The results show that the capacities predicted using the novel method compare favourably with the results obtained from traditional theories using a more complicated iterative process. Therefore, the presented single-step approach may be appealing to the industry.

A test program was formulated to understand better the implications of the recommended best practice of introducing gaps between sheathing panels. It was set to experimentally verify the difference in the shear capacity for setups with and without gaps between the sheathing panels. The significance of this study is that it informs the industry that the manufacturers' recommendation to incorporate a gap between sheathing panels would not compromise the structural integrity. Considering the model uncertainty and the safety margins, the introduction of gaps does not alter the strength or stiffness of the wall.