SOME DESIGN ASPECTS ON ANCHORING OF TIMBER FRAME SHEAR WALLS BY TRANSVERSE WALLS

Bo Källsner[1], Ulf Arne Girhammar[2], Johan Vessby[3]

ABSTRACT: The authors of this paper have developed a method for plastic design of timber frame shear walls. Recently, a Swedish handbook has been presented based on the principles of this plastic design method. In this paper a simplified design method for transverse walls, i.e. walls used for anchoring of shear walls against uplift, is presented. Calculated load-carrying capacities of transverse walls using the simplified method seem to be in fairly good agreement with calculated capacities using the finite element method and agree fairly well with measured capacities obtained from full scale testing of transverse walls.

KEYWORDS: Shear wall, Transverse wall, Anchorage, Analytical model, Finite element analysis, Test results

1 INTRODUCTION

1.1 BACKGROUND

The authors of this paper have developed a method for plastic design of timber frame shear walls. The method is flexible with respect to load configurations and boundary conditions and can, for example, be applied on walls where the leading stud on the windward side is fully or partially anchored with respect to vertical uplift. The method can be used for design of walls with various geometric layouts including the influence of openings. Recently, a Swedish handbook has been presented based on the principles of this plastic design method [1].

1.2 AIM AND SCOPE

The aim of this paper is to present a simplified design method for transverse walls, i.e. walls used for anchoring of shear walls against uplift, by applying the principles of the plastic design method.

Calculated load-carrying capacities of transverse walls using the simplified method will be compared with calculated capacities using a more conservative version of the plastic method mentioned above. Finally, comparisons will be made with results obtained from finite element (FE) analyses and some recently conducted full scale tests of transverse walls.

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2 TRANSVERSE WALLS

Transverse walls, i.e. walls that usually are oriented with their length direction perpendicular to the length direction of the horizontally loaded shear wall under study, can often be used for anchoring purposes at the ends of and, in certain cases, also at openings in shear walls. Such a case is illustrated in Figure 1, where only the applied horizontal force and the distributed anchorage force are shown. In order to fulfil the conditions of force and moment equilibrium, more force components need to be added. A simple force distribution in the walls, fulfilling the conditions of equilibrium, is shown in Figure 2.

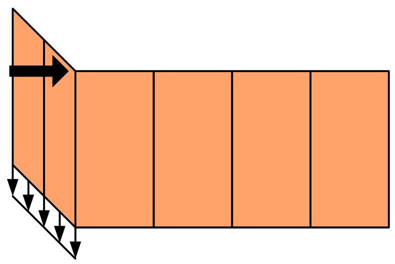

Figure 1: *Anchoring of a shear wall by means of a transverse wall.*

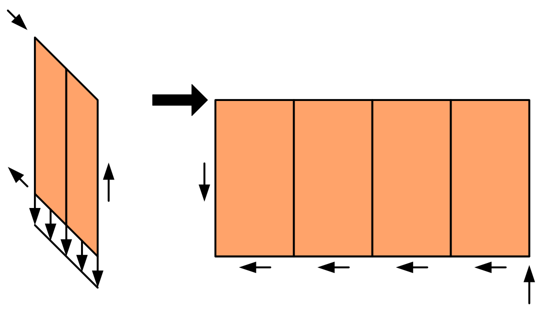

Figure 2: *A simple distribution of forces, in a shear wall and a transverse wall, fulfilling the conditions of equilibrium.*

From the left part of Figure 2 it is clear that the vertical forces must be balanced by two horizontal forces of equal size. Two extreme cases with respect to the boundary conditions of the transverse wall are illustrated in Figure 3. In the left part of the figure the top rail is assumed to be fixed with respect to horizontal displacement and in the right part of the figure to be free.

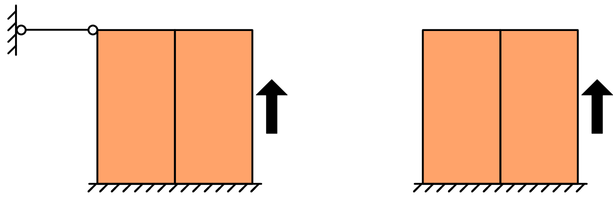

Figure 3: *Two extreme cases of boundary conditions for the transverse wall: horizontally fixed and free top rail with respect to horizontal displacement.*

3 TESTING PROGRAM

The transverse wall configurations tested are shown in Figure 4. The two extreme cases of boundary conditions shown in Figure 3 were used. The bottom rail was continuously anchored to the foundation in all tests. The different types of tests are summarized in Table 1.

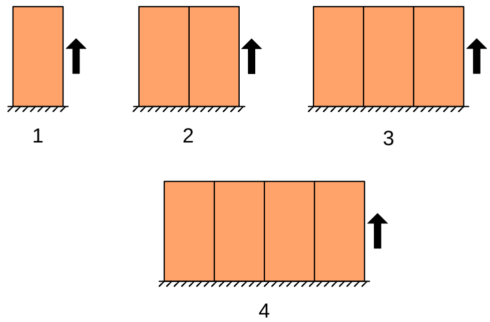

Figure 4: *Transverse wall configurations tested.*

Table 1*: Specification of transverse walls tested.*

Wall configuration	Top rail	Number of tests
1	fixed	17
2	fixed	18
1	free	8
2	free	13
3	free	8
4	free	8

All walls were sheathed with hardboard of dimension 1200×2400×8 mm^3. Only one side of each wall was sheathed. The timber members were of dimension 45×120 mm^2 with a stud spacing of 600 mm. For the sheathing-to-framing joints 50×2.1 mm^2 annular ringed shank nails were used with a centre distance of 100 mm along the perimeter and 200 mm along the centre line of the sheets. In each framing joint two annular ringed shank nails were applied in the grain direction of the vertical studs.

The tests reported in this paper have been conducted at five different occasions. The tests reported in [2]-[4] include only a part of these tests. In some of the tests there were certain problems with the boundary conditions. Thus, in a few tests the horizontal support for the top rail was not completely fixed. These test results have been excluded in the presentation of the results in chapter 4.

The vertical uplift force was applied by fastening a flexible steel bar on the outer side of the timber frame. In the first test series of transverse wall configuration 1 a minor influence of the bar was observed. The results from this test series have not been included in the results presented in chapter 4.

Density and moisture content of the wood and sheet materials close to failure were measured in connection to the testing.

More details about the testing procedure are available in [3].

4 TEST RESULTS

The results obtained from the testing of the transverse walls are summarized in Table 2. The density of the wood material close to failure has also been included in the table. Most of the failures took place in the sheathing-to-framing joints along the bottom rail. For wall configuration 2, with fixed top rail, two failures took place in the sheathing-to-framing joints along the vertical stud where the load was applied. Also for wall configuration 4, with free top rail, two failures occurred along the same stud. Most failures in the sheathing-to-framing joints were of ductile type starting by yielding in bending and after that by withdrawal of the nails. The final failures of the walls were fairly ductile.

Table 2*: Test results for transverse walls with fixed and free top rail including density of bottom rail. (COV = coefficient of variation, * = some test results have been excluded)*

Wall configu-ration	Top rail	Number of tests	Density	Uplift force	
			[kg/m^3]	[kN]	COV
1	fixed	17	409	16.5	0.18
2	fixed	14*	415	21.0	0.10
1	free	4*	436	6.97	0.12
2	free	13	411	11.7	0.15
3	free	8	438	17.7	0.08
4	free	8	417	21.0	0.08

For some of the wall configurations there is an evident influence of the density in the bottom rail on the load-carrying capacity.

5 ANALYTICAL MODELS

5.1 GENERAL INFORMATION

Two analytical models for determining the load-carrying capacity of transverse shear walls will be evaluated: a basic model and a simplified model. Both models have previously been presented and discussed; see e.g. [2]-

[4]. The two models are based on the assumption that the sheathing-to-framing joints have plastic load-displacement characteristics and that the force and moment equilibrium of each timber member and sheet must be fulfilled.

5.2 BASIC MODEL

For the basic model it is assumed that the sheathing-to-framing joints along the top rail and the vertical studs only can transfer forces parallel to the timber members, while the sheathing-to-framing joints along the bottom rail can transfer forces in arbitrary directions. For walls that are more than one storey high it is assumed that the sheathing-to-framing joints along the top rails between the different storeys also can transfer distributed vertical forces.

Considering the transverse wall in Figure 3 with a horizontally fixed top rail it is possible to separate the wall in two parts as shown in Figure 5. The vertical section is placed through the centre of the joining stud between the two sheets and the horizontal section is placed through the sheets just above the bottom rail. The applied vertical uplift force is denoted by V and the horizontal reaction force is denoted by R. The distributed force in the sheathing-to-framing joints is denoted by f_p and is assumed to have attained its plastic capacity along the bottom rail. No transfer of tensile or shear forces in the framing joints is assumed. Since forces only can be transferred in the length direction of the top rail and the studs it is evident that the resultant of the distributed force f_p with an angle of ϕ_1 in the left sheet must pass through the upper right corner of the sheet in order to fulfil the conditions of equilibrium. To determine the vertical anchorage capacity V of the transverse wall, the equilibrium of the right part of the wall (where the distributed force f_p now is directed with an angle of ϕ_2) also needs to be studied. This vertical anchorage capacity V for different cases was determined in [2] and [3] and will not be repeated in this paper. One disadvantage of this basic model is that the equations for determining the anchorage capacity are not suited for hand calculation and should be tabulated or included in a simple spreadsheet program.

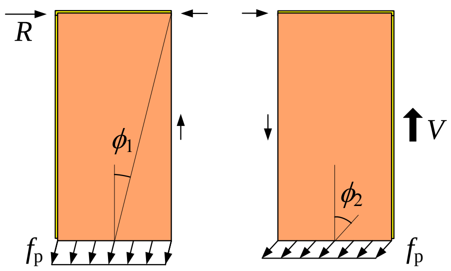

Figure 5: *Assumed force distribution within a transverse wall according to the basic model.*

5.3 SIMPLIFIED MODEL

In the basic model it is assumed that the sheathing-to-framing joints, along the vertical stud joining the two sheets, only are capable of transferring forces in the length direction of the stud. The reason for this assumption is that the edge distances of the sheathing-to-framing fasteners are small and may result in splitting failures in the timber studs or tear-out failures in the sheets.

In the simplified model it is assumed that also certain tensile forces can be transferred between the two sheets via the sheathing-to-framing joints along the vertical stud. In fact it is assumed that these tensile forces are large enough to treat the sheets as a single sheet. This simplification is partly justified by the test results presented in chapter 4, where no local failures were observed in the sheathing-to-framing joints along the vertical studs joining the sheets. One advantage of this assumption is that it leads to less complicated design equations. This simplified model for anchoring shear walls by means of transverse walls is used in the new Swedish handbook [1].

5.3.1 Horizontally fixed top rail

A transverse wall, with a horizontally fixed top rail according to the left part of Figure 3, is analysed assuming that the two sheets can be replaced by one single sheet. Instead of assuming different directions of the distributed force f_p along the bottom rail of the two sheets as indicated in Figure 5, it is here assumed that the resultant force $f_p l$ passes through the upper right corner of the whole single sheet of the transverse wall as in Figure 6.

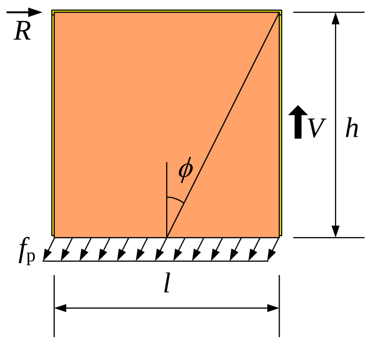

Figure 6: *Assumed force distribution within a transverse wall according to the simplified model for the case of fixed top rail.*

For a given height h and length l the angle ϕ can be calculated from the expression

$$\phi = \arctan(\frac{l}{2h}) \tag{1}$$

Vertical force equilibrium gives

$$V = f_p l \cos\phi \tag{2}$$

Noting that V must not exceed the full plastic capacity of the sheathing-to-framing joints along the stud, $f_p h$, gives the condition

$$V = \min \begin{cases} f_\text{p} l \cos\phi \\ f_\text{p} h \end{cases} \tag{3}$$

Considering the horizontal force equilibrium the reaction force R is obtained as

$$R = \min \begin{cases} f_\text{p} l \sin\phi \\ f_\text{p} h \dfrac{1}{\sqrt{3}} \end{cases} \tag{4}$$

5.3.2 Horizontally free top rail

In the case of horizontally free top rail the transverse wall is assumed to be divided into two parts of equal length, where the two parts are in force equilibrium according to Figure 7. Denoting the length of each part by l, Equations (1) – (4) are valid also for this boundary condition.

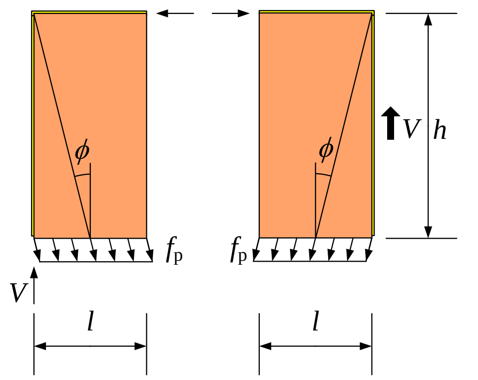

Figure 7: *Assumed force distribution within a transverse wall according to the simplified model for the case of free top rail.*

5.3.3 Influence of vertical distributed load

The influence of a distributed vertical load q_v from self weight, snow or wind can be evaluated by superposition of the two load cases in Figure 6 and Figure 8.

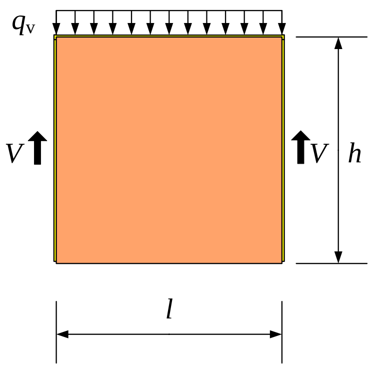

Figure 8: *Influence of a distributed vertical load q_v on anchorage capacity V.*

Thus, the vertical force V is obtained as

$$V = \min \begin{cases} (f_\text{p} \cos\phi + \dfrac{1}{2} q_\text{v}) l \\ f_\text{p} h \end{cases} \tag{5}$$

Since no closed solution for determining the reaction force R is available an approximate expression, somewhat on the safe side, is given here

$$R = \min \begin{cases} f_\text{p} l \sin\phi \\ f_\text{p} h \dfrac{1}{\sqrt{3} + \dfrac{2q_\text{v}}{f_\text{p}} + \dfrac{1}{\sqrt{3}}\left(\dfrac{q_\text{v}}{f_\text{p}}\right)^2} \end{cases} \tag{6}$$

5.3.4 Walls that are more than one storey high

Walls that are more than one storey high can be analysed using the same principles as described above denoting the total height of the wall by h. If horizontal loads are acting at different heights of the building, the principle of superposition can be used for each horizontal load. In this case h denotes the height of the building, up to the level where the horizontal load is acting.

6 FE MODEL OF THE SHEAR WALL

6.1 GEOMETRY AND MATERIAL PROPERTIES

Finite element simulations were performed using the commercial software Abaqus. Geometry and material properties were chosen as in paper [5].
No contact been adjacent sheets was assumed to take place in the analyses.

6.2 MODELS OF THE JOINTS

6.2.1 Sheathing-to-framing joints

The mechanical properties of the sheathing-to-framing joints are of vital importance for the structural behaviour of shear walls. In the present study, an uncoupled spring pair model is used for the modelling of these joints; cf. [5]. The spring properties used are based on experiments performed in the direction parallel with and perpendicular to the timber members; see [6]. The force-displacement curves used in the calculations are shown in Figure 9. In case of unloading, a linear path parallel to the initial stiffness of the considered spring is assumed.

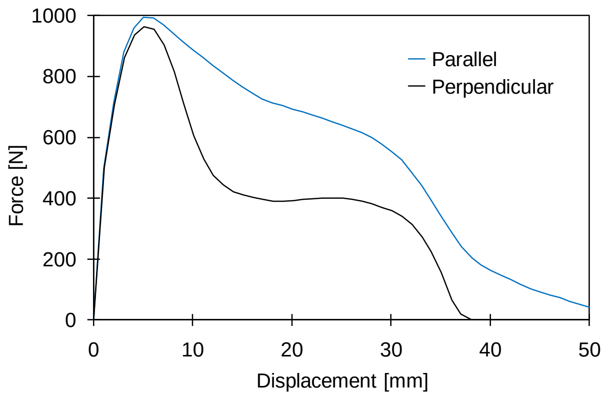

Figure 9: *Assumed force-displacement curves for the sheathing-to-framing joints in the parallel and perpendicular to grain direction of the timber members.*

6.2.2 Framing joints

The mechanical properties of a framing joint are modelled using two uncoupled springs [5]. One spring acting in the length direction of the rail simulates the shear stiffness of the joint and one spring in the length direction of the stud simulates the tension or compression stiffness of the joint. The force-displacement curves used in the calculation are shown in Figure 10. They are based on experimental results given in [7].

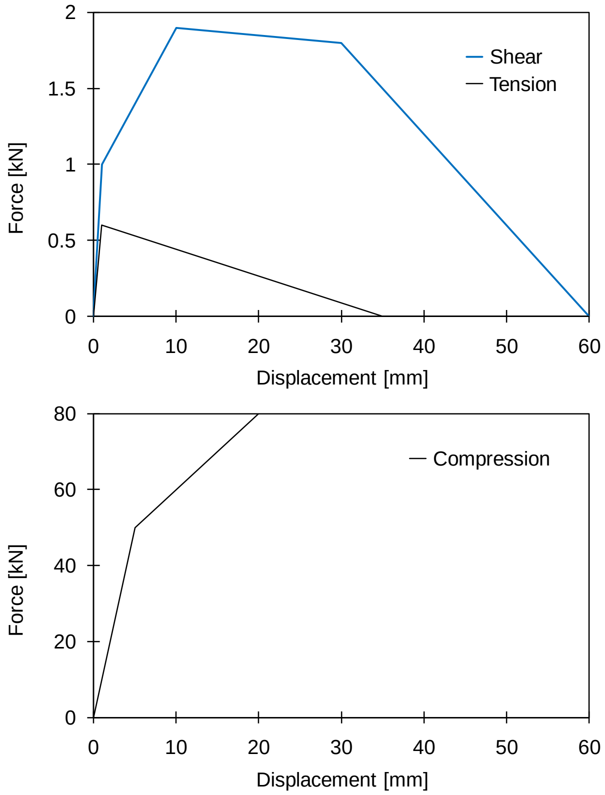

Figure 10: *Assumed force-displacement curves for the framing joints in shear, tension and compression.*

6.3 LOAD APPLICATION AND SUPPORT CONDITIONS

At the testing of the shear walls the vertical uplift force *V* was applied a little outside the outer edge of the stud. In the FE calculations this fact was taken into account by applying the force just on the outer edge of the stud and, thus, giving rise to an additional bending moment. The vertical uplifting force is applied under displacement control implying that the post-peak behaviour of the transverse walls can be studied.

7 COMPARISON OF MODELS

In order to compare the three models previously described in chapter 5 and 6, eight transverse wall configurations designated according to Figure 11 are analysed. The height to width ratio of the sheets is 2:1. For the walls denoted 2S and 2L the letters S and L stand for small and large sheets, respectively. The notation LV is used in case of large vertical sheets. All wall configurations are investigated for the two extreme boundary conditions free and fixed top rail according to Figure 3.

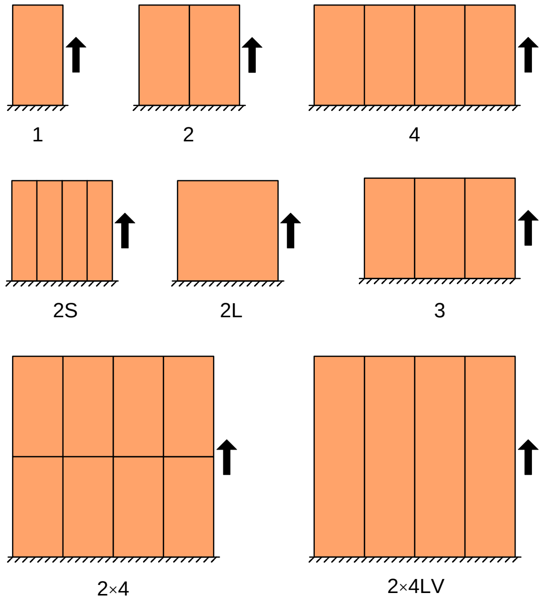

Figure 11: *Different transverse wall configurations studied.*

In Figure 12-14 results from FE calculations, using the model described in chapter 6, are shown in the form of displacement plots. The displacements of the timber members and the sheets are shown at the instance of maximum load of the transverse walls studied and the grey shaded areas indicate the original position and the mesh of the sheets. The displacements are exaggerated by a factor ranging from 25 to 50.

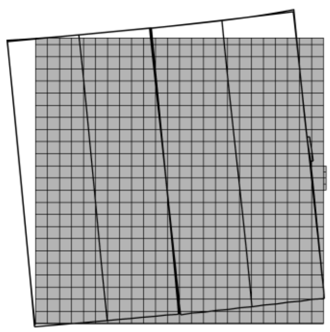

Figure 12: *Displacement plot at the instance of maximum load of transverse wall configuration 2 with free top rail (25 times enlargement).*

The displacement plot in Figure 12, showing a transverse wall consisting of two sheets with a free top rail, indicates that the vertical displacement of the sheets varies more or less linearly along the bottom rail. In this case the vertical displacement is zero about 300 mm from the lower left corner of the wall. In the simplified model, described in section 5.3, it is assumed that the sheathing-to-framing joints have ideal rigid-plastic properties, which means that the vertical displacement in principle is zero in the left corner of the wall; compare Figure 7, where the distributed plastic force f_p is assumed to act in the same direction along the entire bottom edge of the left sheet. From Figure 12 it is also evident that the shear displacements between the sheets are relatively small, something that is in agreement with the assumed force distribution shown in Figure 7.

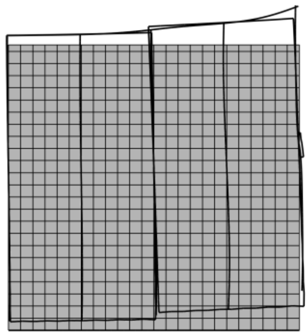

Figure 13: *Displacement plot of transverse wall configuration 2 with fixed top rail (25 times enlargement).*

In Figure 13 a displacement plot is shown for a wall consisting of two sheets where the top rail is assumed to be fixed with respect to horizontal displacements. In this case evident shear displacements are observed between the sheets and the vertical displacement of the sheet to the right is about twice of that for the left sheet.

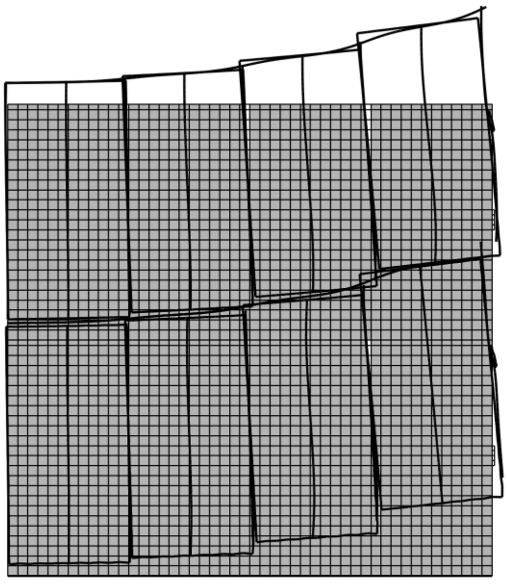

Figure 14: *Displacement plot of transverse wall configuration 2×4 with fixed top rail (50 times enlargement).*

In Figure 14 a corresponding displacement plot for a 2-storey wall is given. The vertical displacement of the very right sheet is of the order six times the one of the very left sheet.

For the wall configurations presented in Figure 11, the load-carrying capacities are calculated using the three models previously described. The results of the calculations are presented in Table 3. For the two analytical models the distributed plastic capacity f_p of the sheathing-to-framing joints has been taken from the curve representing the perpendicular direction in Figure 9. Using the peak value and a centre distance between the fasteners of 100 mm leads to f_p = 0.964 N/mm.

Table 3*: Calculated load-carrying capacities V of transverse walls using the different models.*

Wall configuration	Top rail	Type of model		
		Basic [kN]	Simpl. [kN]	FEM [kN]
1	fixed	11.2	11.2	14.1
2S	fixed	19.5	20.7	23.6
2	fixed	19.8	20.7	25.6
2L	fixed	20.7	20.7	26.2
3	fixed	23.1	23.1	26.1
4	fixed	23.1	23.1	26.1
2×4	fixed	-	41.4	42.5
2×4LV	fixed	39.1	41.4	43.8
1	free	5.78	5.74	6.3
2S	free	11.1	11.2	12.0
2	free	11.2	11.2	11.9
2L	free	11.6	11.2	12.0
3	free	16.0	16.2	16.3
4	free	19.8	20.7	19.5
2×4	free	-	22.4	20.1
2×4LV	free	22.2	22.4	20.8

Comparing the capacity values obtained by the two analytical models it is found that the basic model in general gives lower values than the simplified model. Obviously, the reason for this fact is that in the basic model, the sheathing-to-framing joints along the vertical studs only transfer force components parallel to the timber members, while in the simplified model they also transfer force components perpendicular to the studs. It is only when the transverse wall consists of a single sheet and the top rail is free that the basic model gives higher capacities than the simplified model. The reason here is that when the simplified model is used the wall is always divided into two separate parts.

To draw some general conclusions based on the differences in the capacity values obtained by the simplified model and the FE model is somewhat complicated since the differences depend on the mechanical properties of the sheathing-to-framing joints and the framing joints. First, the transverse walls with fixed top rail will be analysed.

For wall configuration 1 (one sheet) the capacities V obtained by the simplified model and the FE model are 11.2 and 14.1 kN, respectively, i.e. a difference of 2.9 kN. From Figure 13 it is clear that the sheet at the very left is mainly displaced vertically implying that the sheathing-to-framing joints along the bottom rail of this sheet will reach their maximum loads more or less at the same time. One difference between the simplified and FE analyses is that the influence of the framing joints is incorporated in the FE analysis. Three framing joints along the bottom rail give a force contribution of about 3 × 0.6 = 1.8 kN (cf. the tension curve in Figure 10). Another difference is that the sheathing-to-framing joints are modelled as continuously distributed in the simplified analysis, while they are modelled as discrete points in the FE analysis. The number of fasteners along the bottom rail is then in the simplified analysis 1200/100 = 12 fasteners and in the FE analysis 13 fasteners. One extra fastener gives a force contribution of about 1.0 kN (cf. the shear curve in Figure 10). The FE model should consequently for wall configuration 1 give about 2.8 kN higher capacity than the simplified model, which is in good agreement with the difference 2.9 kN found above.

A corresponding analysis of transverse wall configuration 2 (two sheets) gives that the force contribution from five framing joints is 5 × 0.6 = 3 kN and from two extra nails is 2 kN, i.e. in total 5 kN. The difference according to Table 3 is 25.6 – 20.7= 4.9 kN.

Further, analysing transverse wall configuration 2S (four narrow sheets) gives that the additional force contribution for the FE method (five framing joints and four extra nails) should be equal to 5 × 0.6 + 4 × 1 = 7 kN. Table 3 states that the FE method gives an additional force of only 23.6 – 20.7 = 2.9 kN. The main reason for this large difference is that the load-displacement curve for the sheathing-to-framing joints in the perpendicular direction has a relatively pointed characteristic with a distinct peak value, cf. Figure 9, and the magnitude of the forces in some of these joints are quite far from this peak value when the maximum capacity of the transverse wall is attained. A more ductile behaviour of the sheathing-to-timber joints would result in a higher capacity for this wall configuration (cf. the results presented in [5]).

For wall configuration 2L (one large sheet) the FE calculation indicates that failure starts along the vertical stud, where the vertical load V is applied. The wall is, however, very close to failure along the bottom rail.

Also for wall configuration 3 and 4 the FE calculations indicate that failure takes place in the sheathing-to-framing joints where the vertical load is applied.

A displacement plot of wall configuration 2×4 has already been shown in Figure 14. It was in this context stated that the sheathing-to-framing joints along the bottom rail were quite differently displaced in the vertical direction. Like for wall configuration 2S, the FE calculation would have led to a higher capacity if the sheathing-to-framing joints had demonstrated a more ductile behaviour. In this case the wall is built up of quite many sheets and it is evident that the FE method only gives a slightly higher capacity than the simplified method.

In wall configuration 2×4LV the number of sheets has been reduced by doubling the height of the sheets. The FE method gives for this wall configuration, as expected, a somewhat higher capacity than for wall configuration 2×4.

In the following, the capacity of transverse walls with free top rail, using the simplified method and the FE method, will be compared. In general it is clear from Table 3 that the capacities calculated by the FE method are closer to the values calculated by the simplified method than the corresponding capacities previously determined assuming fixed top rail. The main reason is that the vertical displacement of the sheathing-to-framing joints tends to become more linearly distributed along the bottom rail when the top rail is free (cf. Figure 12) and that it is difficult to achieve a plastic stress distribution because of the assumed load-displacement curve with a distinct peak characteristic in the perpendicular direction.

8 COMPARISON BETWEEN MEASURED AND CALCULATED CAPACITIES

In Table 4 the calculated and measured load-carrying capacities of all tested transverse wall configurations are given.

***Table 4**: Calculated and measured load-carrying capacities of all tested transverse wall configurations.*

Wall configuration	Top rail	Type of model			Test
		Basic [kN]	Simpl. [kN]	FEM [kN]	[kN]
1	fixed	11.2	11.2	14.1	16.5
2	fixed	19.8	20.7	25.6	21.0
1	free	5.78	5.74	6.3	6.97
2	free	11.2	11.2	11.9	11.7
3	free	16.0	16.2	16.3	17.7
4	free	19.8	20.7	19.5	21.0

A comparison between the measured load-carrying capacities and the capacities calculated by the FE method shows that the agreement is good in the case of free top rail. The FE method seems to underestimate the capacity slightly. The reason is probably that a somewhat conservative load-displacement curve has been chosen for the sheathing-to-timber joints in the perpendicular direction. For the case of fixed top rail, however, the general trend of the results deviates somewhat for transverse wall configuration 2, where the capacity obtained by the FE method is about 20 % larger than the measured capacity. The reason for this deviation has not been fully clarified so far.

The capacity values calculated by the simplified method are lower than the capacities measured in the tests, i.e. the analytical values are on the safe side. However, it should be noted that a lower f_p-value has been used in this study than in the papers [2]-[4].

9 CONCLUSIONS

In this paper a simplified plastic model for design of transverse walls subjected to uplift forces is presented. The simplified model, which is suitable for hand calculations, is compared with a somewhat more complicated plastic model, called the basic model, in which only vertical force components are accepted to be transferred between the sheets and the studs. The comparison of the two models shows that the basic model gives slightly lower load-carrying capacities than the simplified model. However, in case of transverse walls consisting of only a single sheet and with a free top rail, the basic model gives slightly higher load-carrying capacities.

Results are also presented using a FE model in which the mechanical properties of transverse walls made of hardboard are simulated. A comparison between calculated capacities using the FE model and the simplified plastic model shows that for transverse walls with fixed top rail, the FE model in general leads to higher capacity. One reason is that the sheathing-to-framing joints are modelled as discrete points in the FE analysis, while they are modelled as continuously distributed in the simplified model. Another reason is that the influence of the framing joints is not taken into account in the simplified model.

For transverse walls with free top rail the capacities calculated using the FE method are closer to the capacities obtained by the simplified model than in the case of fixed top rail. This result can be explained by the fact that the vertical displacement of the sheathing-to-framing joints tends to become more linearly distributed along the bottom rail when the top rail is free and that it is difficult to achieve a plastic stress distribution because of the assumed load-displacement curve with a distinct peak characteristic in the perpendicular direction.

A comparison between the measured load-carrying capacities and the capacities calculated by the FE method shows that the FE method seems to underestimate the capacity slightly. The reason is probably that a somewhat conservative load-displacement curve has been used in the FE calculations to represent the behaviour of the sheathing-to-timber joints in the perpendicular direction.

The capacity values calculated by the simplified design model are lower than the capacities measured in the tests, i.e. the analytical values are on the safe side.

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REFERENCES

[1] Källsner B., Girhammar U.A. Horizontal stabilisation of timber frame buildings – Plastic design of shear walls (in Swedish). SP Report 2008:47, SP Technical Research Institute of Sweden, 2009.

[2] Girhammar, U.A., Källsner, B. Effect of transverse walls on capacity of wood-framed wall diaphragms. In: Proceedings of CIB/W18, Florence, Italy, 2006.

[3] Girhammar, U.A., Källsner, B. Effect of transverse walls on capacity of wood-framed wall diaphragms – Part 2. In: Proceedings of CIB/W18, Bled, Slovenia, 2007.

[4] Girhammar, U.A., Källsner B. Effect of transverse walls on capacity of wood-framed wall diaphragms without tie-downs. In: Proceedings of WCTE, Miyazaki, Japan, 2008.

[5] Vessby, J., Källsner, B., Girhammar, U.A. Influence of contact stress between sheets on strength and stiffness of timber frame shear walls. In: Proceedings of WCTE, Trentino, Italy, 2010.

[6] Girhammar, U.A., Bovim, N.I., Källsner B. Characteristics of sheathing-to-timber joints in wood shear walls. In: Proceedings of WCTE, Lahti, Finland, 2004.

[7] Palm, S. Tests on framing joints in wood-framed shear walls. Umeå University, Faculty of Science and Technology, Department of TFE – Building Engineering, Report 2002:3, Umeå, Sweden, 2000. (in Swedish).