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Influence of parameters affecting the racking strength of partially anchored timber framed walls

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ABSTRACT: A test-based assessment of the racking performance of partially anchored timber framed walls is described in this paper. The tested wall specimens were constructed and tested at the Centre for Timber Engineering (Edinburgh Napier University, UK) considering different configurations of the wall geometry, such as wall length, nail spacing and size and position of studs under different vertical loading arrangements. Based on the experimental results, comparisons are made with the design-based strength values obtained according to relevant European and UK national standards, and are discussed in this paper.

KEYWORDS: Platform frames, Timber racking walls, Raking, Test procedures, PD6693-1, Eurocode 5.

1 INTRODUCTION

Modern (off-site) timber framing is an effective method of construction for lightweight structures of (low-to-midrise) multi-storey dwellings and office buildings. In addition to supporting vertical loads, timber framed walls also provide stability to the building structure against lateral actions, such as wind loads. Oriented Strand Board (OSB), Particleboards or plywood are the materials typically used as sheathing panels, fixed onto the timber frame, providing the required in-plane resistance to the wall.

1.1 PARTIALLY ANCHORED WALLS

Two main categories are usually recognised for timber-framed walls that, in addition to providing bearing capacity, are also capable of resisting in-plane racking forces:

- **Fully anchored**: the wall is prevented from lifting by the use of steel ties, or similar anchoring devices, connected to the underlying support structure.
- **Partially anchored**: resistance against in-plane forces is solely provided by the fixings connecting sheathing panels to the timber frame's bottom rail, and fixings connecting the bottom rail to the support structure.

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1.2 AIMS OF RESEARCH

Partially anchored is the most commonly used system in the UK for timber frame construction, and the present study focuses solely on this method of construction. The main aim of this work was thus providing first-hand experimental-based data on how the variation of some geometric parameters (namely, wall length and panel-to-frame fastener spacings) affect wall racking strength capacities. The study also focused on quantifying any differences arising between the experimental results and the design racking values obtained from Eurocode 5 [1] standard (EC5) and UK National Annex to EC5, which relies on the design procedure described in the PD 6693-1 document [2] for UK design guidelines for timber framed racking walls. A detailed description of the method and underlying principles can be found in [3].

2 Method

A total of 11 specimens were tested, all of them assembled using C16 [4] white spruce timber for the frame, and 9 mm thick OSB for the sheathings, which were fixed using two types of bright smooth nails: 2.8 mm diameter by 49 mm long and 3.0 mm diameter by 52 mm long. Header beam and bottom rail were connected to the suds using 75 mm long screws with a smooth shank diameter of 3.2 mm.

2.1 RACKING TEST SET-UP

The racking tests were set-up following BS EN 594:2011 guidelines [5]. With reference to Figure 1, all wall specimens were fixed to a sole which in turn was lying on the test rig base. The bottom rail was fixed to the test bed by four 12 mm diameter bolts. The racking load was then applied by a load actuator at the top-left corner of the wall, and two linear transducers (LVDT-1 and LVDT-2) were used to take readings of the horizontal deformations. The racking deformation of each wall (Δ_h)

was then taken as the difference between horizontal displacement of the

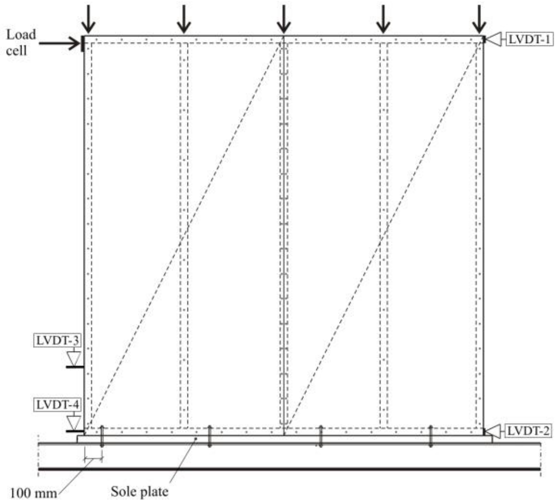

Figure 1: *Racking test set up in accordance with BS EN 594:2011.*

header beam and the rigid-body horizontal translation of the wall. To prevent out-of-plane movements, a system of bracing and rollers was put in place. The presence of vertical loading was taken into account during tests with the aid of a pressurised air-bag device, located between the wall specimen's header beam and the overlying loading rig cross-bar, as shown in Figure 2. The air-bag device sat on steel rollers, which were vertically aligned to the wall studs, thus simulating the path of loads coming from floor joists. The air pressure in the device was calibrated for different increments of total vertical load.

2.2 TEST SERIES

Three series of tests (I, II and III) were carried out on wall specimens with a constant height of 2.4 m. For test series I and II six wall specimens with constant length of 2.4 m and variable nails spacing (ranging from 50 mm to 150 mm) were assembled and tested, whereas five wall specimens were assembled for test series III, with a base

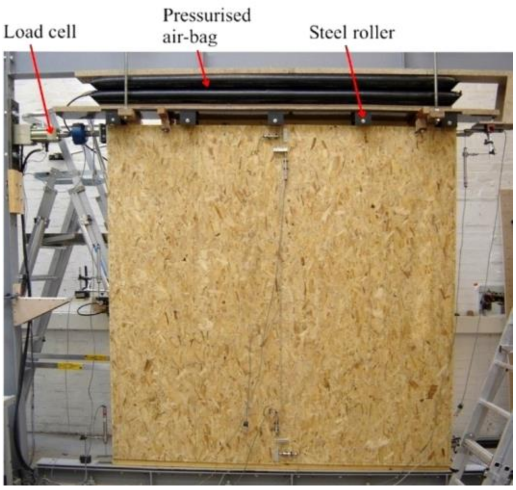

Figure 2: *Application of vertical loading via air-bag device.*

length ranging from 300 mm to 1800 mm (see Figure 3). Details of configurations for all test series and specimens are given in Table 1.

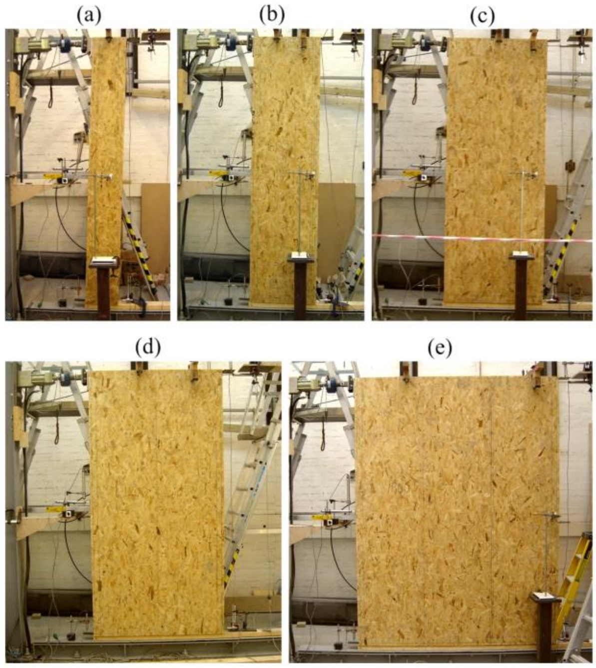

Figure 3: *Racking test series III on wall specimens with different base lengths: (a) L = 300 mm, (b) L = 600 mm, (c) L = 900 mm, (d) L = 1200 mm, (e) L = 1800 mm.*

Table 1: *Wall specimens, summary of test series.*

Test ID	Wall length [mm]	Vertical load [kN]	Nails [mm] size	Nails [mm] Spacing[a]
I-1				50
I-2	2400	0	2.8x49	100
I-3				150
II-1				50
II-2	2400	25	3.0x52	100
II-3				150
III-1	300			
III-2	600			
III-3	900	0	2.8x49	100
III-4	1200			
III-5	1800			

[a] Of the perimeter panel-to-frame connections.

2.3 PD 6693-1

According to the UK National Annex to Eurocode 5, the procedure to adopt for racking strength analyses of timber framed walls is that given in the PD 6693-1 document [2]. Such a semi-empirical procedure is based on the plastic model developed by Källsner and Girhammar [6] for partially anchored framed wall diaphragms. According to the PD method, a lower bond estimate of the wall's racking strength (indicated in here as $F_{\mathrm{h,max}}$) is obtained as a function of the panel-to-frame fastener strength per unit length, f_{pd}, cumulated along a certain length, l_{eff}, at the bottom of the wall:

$$F_{\mathrm{h,max}} = f_{\mathrm{pd}}\, l_{\mathrm{eff}} \qquad (1)$$

The value of f_{pd} is obtained by dividing the mean value of panel-to-frame fasteners strength, $F_{v,mean}$ by the fasteners spacing, s:

$$f_{pd} = \frac{F_{v,mean}}{s} \tag{2}$$

As explained in [7], a mean strength value is considered in Eq. (2) instead of a 5-percentile value because when a significant number of fasteners are located in a line configuration (as in this case) it is unlikely that all these fasteners will only achieve the minimum failure strength. Such a mean strength value is derived from the characteristic (5-percentile) value, $F_{v,Rk}$, increased by a factor between 1.2 and 1.3, depending on the value of s [2]:

$$F_{v,mean} = (1.5 + s)F_{v,Rk} \tag{3}$$

where s in Eq. (3) is expressed in meters. If OSB panels are used for the sheathing material, the value of $F_{v,Rk}$ can be derived according to EC5 procedure for determining the characteristic strength of laterally loaded connections formed with meatal dowel fasteners. In particular, the characteristic load-carrying capacity of the connection will be obtained from EC5 Eq. (8.6), and the critical mode of failure has been found to be *failure mode (d)* for both nail sizes.
Having derived the relevant values of f_{pd}, the remaining parameter to insert into Eq. (1) is the effective anchoring length l_{eff} given as follows [2]:

$$l_{eff} = -\frac{H}{\mu} + \left[\frac{H^2}{\mu^2} + L^2\left(1 + \frac{2M}{\mu f_{pd} L^2}\right)\right]^{0.5} \tag{4}$$

where H and L are the height and base length of the wall respectively, whereas M is the stabilising moment at the leeward side of the wall:

$$M = Q\frac{L}{2} \tag{5}$$

with Q being the overall load (i.e. in kN) acting along the top of the wall. According to the PD method, the term μ is to be taken as follows:

$$\mu = max\left\{1; \frac{f_{ax}}{f_{pd}}\right\} \tag{6}$$

with f_{ax} being the withdrawal strength per unit length of connections fixing the wall to the underlying structure. For values of strength ratio (f_{ax}/f_{pd}) greater than 1 the failure condition will be clearly dictated by the strength of the panel-to-frame connections (f_{pd}), and since the base rails of the tested walls were anchored to the test rig basement by bolts, the value of μ is always taken equal to unity in this study.
A further requirement concerns the value of the effective anchoring length as from Eq. (4) which is subject to the following inequality condition:

$$0 \le l_{eff} \le L \tag{7}$$

3 RESULTS

Strength results obtained from tests, and their comparison with the analytical (PD 6693-1) results, are summarised in the following sections.

3.1 EFFECT OF NAILS SPACING

In Figure 4 the variation in racking strength of the wall as a function of the nail spacing (s) is shown. As it can be clearly seen, the analytically predicted values are consistently lower than those experimentally found regardless of the nail spacing. Nonetheless, although staggered downward the analytical curves effectively follow the variation of racking strength due to the different fastener spacing used. With reference to Figure 4(a): for s = 50 mm the experimentally found racking strength is 23.13 kN whereas the design rule function gives a racking strength of 13.25 kN, which is 43% less than the corresponding experimental value. For s = 150 mm, the experimental racking strength value is 13.10 kN while the analytical value is 4.78 kN, i.e. 63% smaller. A less pronounced difference is found instead for the walls tested under 25 kN applied vertical load: As shown in Figure 4(b), for s = 50 mm the experimentally found racking strength is 40.72 kN whereas the design rule function provides a value of 22.68 kN, i.e. -44%. For s = 150 mm the analytical value is -41% of the corresponding experimental value.

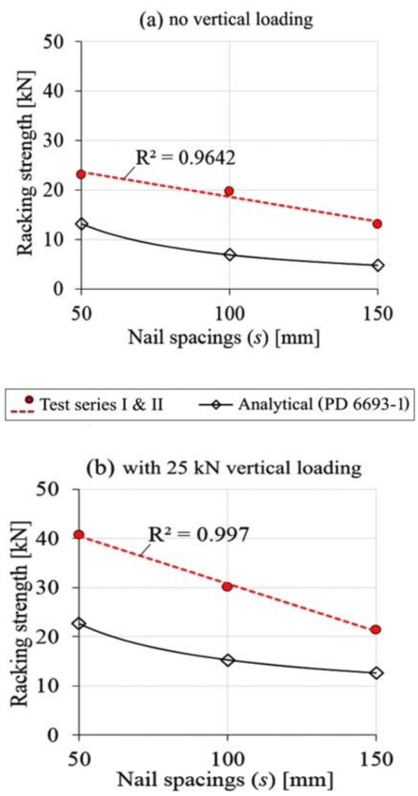

Figure 4: *Racking strength as a function of the panel-to-frame nail spacing (s). The experimental values are referred to: (a) test series I, i.e. walls assembled with 2.8 mm x 49 mm nails and without applied vertical load. (b) test series II, i.e. walls assembled with 3.0 mm x 52 mm nails and with 25 kN vertical load (see Table 1).*

Clearly, the underestimations of the analytical function are much more pronounced for the test case without applied vertical load. An explanation to this is provided as follows: according to the PD method, the racking strength increases with the increase of the stabilising moment $M=QL/2$ (see Eq. (1), (4) and (5)). A further contributor to the stabilising moment is provided by the stud-to-beam connections at the withdrawal bottom end of the wall. Such contribution is ignored in the PD method for a combination of practical and conservative reasons. However, while for the case with Q = 25 kN such contribution only represents a small percentage of the stabilising moment, for Q = 0 kN there will be a stabilising moment entirely due to the withdrawal capacity of these connections, which is ignored in the analysis. This affects the analytical results and contribute to the reason why there is a different behaviour between loaded and unloaded test and corresponding analytical results.

3.2 EFFECT OF WALL LENGTH

Figure 5 shows the variation in racking strength as a function of the wall length, *L*. As it can be observed from the Figure, the analytical racking strength curve remain well below the experimental curve for the entire range. the relative underestimation increases as the wall length is reduced: for *L* = 2400 mm, the analytical racking strength is predicted to be 6.90 kN, i.e. about 65% less than the experimental value of 19.79 kN. As the wall length reduces to 300 mm, the analytically predicted racking strength becomes about 80% lower than the corresponding experimental value of 0.89 kN.

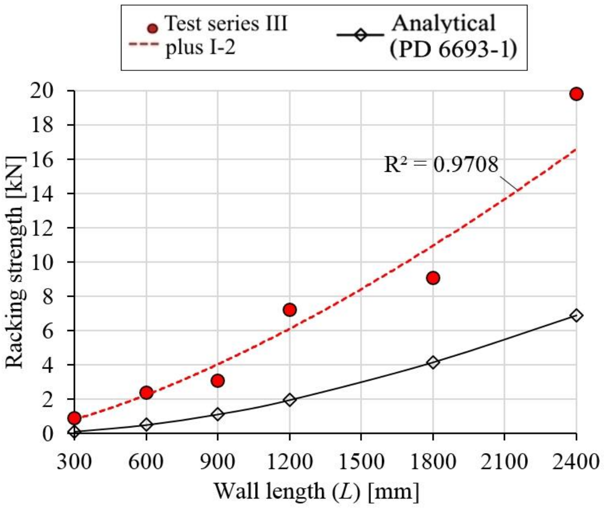

Figure 5: *Racking strength as a function of the wall length (L). The experimental values are referring to test series III plus I-2, i.e. walls made with sheathings fixed at 100 mm spacings.*

4 CONCLUSIONS

Aims of this work were to evaluate how the racking strength of partially anchored timber framed walls is affected by nail spacings and wall length, as well as quantifying any occurring difference between the experimentally-based and design rule (PD 6693-1) results. Substantial discrepancies between the two sets of results are found. In particular, walls without applied vertical loads (e.g. as a result of internal wind pressure acting upwards) or with a small base length are those most penalised by the current UK design guidelines for timber framed walls. In particular:

- The walls' racking strength is more sensitive to variations in the fastener spacings (*s*) when it is subjected to a vertical loading. For example, by looking at the experimental curves, the increase in racking strength, for walls without vertical loading, is about 76% when the fastener spacing is reduced from 150 mm to 50 mm, whereas a strength increase of about 89% is found when a vertical load of 25 kN is also applied to the wall.

- On average, the (analytical) design rule underestimates the racking strength by 25% for walls under vertical loading of Q = 25 kN and by 54% for walls without vertical loading. Noting that the analytical model only provides a lower bound value for the racking strength of the wall, the most likely explanation to why such an underestimation is greater for walls without applied vertical load, is due to the contribution to the stabilising moment, *M*, and given the withdrawal capacity of the stud-to-beam connections.

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