Stability of Domestic Dwellings

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Summary

A comparative study of typical UK timber frame domestic dwellings is carried out in relation to system stability. The concepts of stiffness proportionality, redundancy, continuity and robustness are explored in relation to current UK timber frame design detailing. In particular the application of BS EN 1995 for the design of the sole plate to foundation connection is considered with guidance given to allow safe but economical design from information which is normally available from suppliers.

1. Introduction

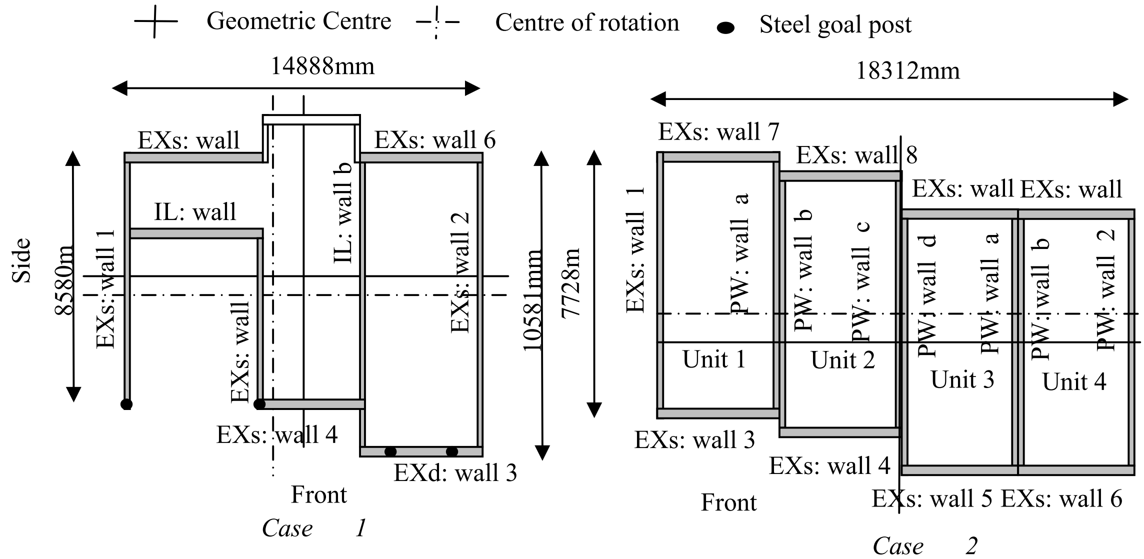
In platform timber frame design it is normal to consider system stability in two parts:

1. *Overall system resistance to sliding and overturning as a result of the applied wind action:* In the majority of circumstance the self weight of the system results in a holding down moment and, as a result of friction, a resistance to sliding, both of which are greater than the applied overturning and sliding forces.
2. *The transmission of applied shear to the foundation*: The wall diaphragms of the system transfer the applied wind action to the foundation via shear connections and holding down straps.

The focus of this paper is the transmission of applied shear to the foundations.

2. Comparative study

A comparative study of 3 different 2 storey platform timber frame design cases (Figure 1), in relation to shear transmission, has been carried out. Each design case is reflective of normal UK timber frame construction and is carried out in accordance with BS 5268: Section 6.1: 1996 as a result of it being current design practice. Externally the systems are masonry clad with the masonry tied to the timber frame with standard wall ties. The roof system consists of fink roof trusses braced in accordance with BS 5268: Section 6.1:1996. The floor diaphragms for the design cases considered are constructed from I joists decked on top with 22mm chip board flooring and on the underside with a 13mm plasterboard ceiling.



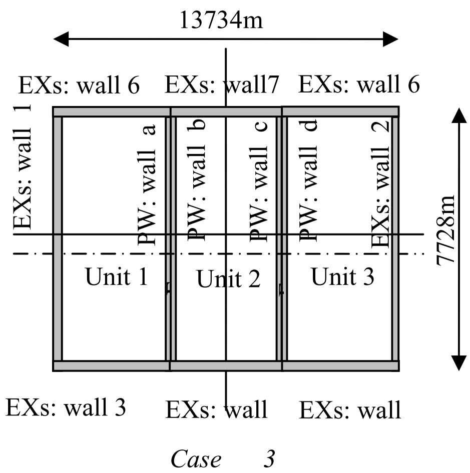


Figure 1 Design cases

The applied loads to the system have been calculated in accordance with British Standard Codes of Practice; BS 6399-1:1997 “Loadings for buildings”, Parts 1, 2 & 3. The site location and building orientation is the same for all three design cases and as a result the applied wind action is consistent. However, it is to be noted that the height to ridge of Design Case 1 is 8.9m and that the pitch of the roof is 40 degrees spanning front to back. Design Cases 2 and 3 have an overall height to ridge of 7.4m and the pitch of the roof is 35 degrees spanning each individual unit, wall 1 to wall a, wall a to wall b and so on.

The timber frame wall diaphragms have an overall height of 2400mm, consist of 38x89mm grade C16 timbers with studs at 600mm centres. The walls are sheathed internally and externally as designated in Table 1. Sheathing is fixed using 3mm diameter by 50mm long galvanised wire nails at 100mm centres to external framing members and 200mm centres to internal framing members.

Rigid diaphragm action has been assumed and as a result applied shear to the system is distributed to the shear walls relative to their stiffness (Prion & Lam, 2003). It can be assumed that stiffness and shear resistance of the walls are directly related; therefore applied wind action in this study is distributed to the walls relative to their shear resistance.

By adopting a rigid analysis system torsion has to be considered. Applied torsion is dealt with by determining the centre of rotation of the system and distributing the resulting torsion forces to the walls relative to the moment resistance they provide to the system. Table 2 & 3 contain a break down in results for Case 1 and Table 4 summaries the results of all three cases. It is noted that if the torsion component is negative, which would serve to reduce the applied level of shear, it is conservatively taken as zero.

Table 1 Wall sheathing arrangement

Type	Description	Sheathing Arrangement	
		External	Internal
Exd	External Double Sheathed	9mm OSB Grade 3	9mm OSB Grade 3
Exs	External Single Sheathed	9mm OSB Grade 3	12.5mm Plasterboard
IL	Internal Load Bearer	12.5mm Plasterboard	12.5mm Plasterboard
PW	Party Wall		12.5mm + 19mm Plasterboard

Table 2 Case 1 wind acting on front

No.	Wall	Applied		
	Resistance	Shear	Torsion	Total
	kN	kN	kN	kN
1	30.05	6.70	0.95	7.65
2	42.10	9.24	0.28	9.52
a	30.01	6.59	0.00	6.59
b	19.84	4.36	0.00	4.36
Σ	**122.00**	26.88	1.23	**28.11**

Table 3 Case 1 wind acting on side

No.	Wall	Applied		
	Resistance	Shear	Torsion	Total
	kN	kN	kN	kN
3	10.40	7.47	0.00	7.47
4	4.82	3.36	0.00	3.36
5	11.48	8.24	0.05	8.29
6	3.74	2.53	0.02	2.54
p	12.55	5.13	0.00	5.13
Σ	**42.99**	26.72	0.07	**26.79**

Table 4 Results of Case 1, 2 & 3 summarised (inclusive of allowable shear transfer)

Case	Wind acting on	Wall Resistance	Allowable shear transfer	Design shear resistance	Applied			Design Outcome
					Shear	Torsion	Total	
		kN	kN	kN	kN	kN	kN	
	Front	122	47.78	47.78	26.88	1.23	28.11	OK
1	Side	42.99	34.54	34.54	26.72	0.07	26.79	OK
	Front	188.44	84.64	84.64	41.81	6.59	48.4	OK
2	Side	31.77	50.14	31.77	23.77	3.48	27.25	OK
	Front	147.48	63.48	63.48	21.78	0	21.78	OK
3	Side	26.22	18.80	18.80	17.8	1.56	19.36	Fail

It is shown in Table 4 that for all three cases the actual wall resistance to shear is greater than the applied wind action and it is noted that for all cases the gable walls provide a high level of resistance as a result of having no openings. No openings assist racking resistance on two major counts:

1. Increased panel area providing racking resistance.
2. Reduction in applied wind force as a result of increased masonry shielding.

For all three cases the centre of rotation is in close proximity to the geometric centre. When the centre of rotation is close to the geometric centre torsion in the system is reduced and as a result the system is capable of carrying increased wind action. This can be critical in cases of large openings; in particular if the systems in Cases 2 & 3 had not been well proportioned in regards to stiffness extra racking resistance would have been required incurring a financial cost. Stiffness proportionality of the system therefore increases the level of shear the system can carry and results in more economical design.

3. System continuity

System continuity is an important factor when considering the resistance of a system to applied wind action. In particular continuity across party walls is considered. Consider when the wind action is on the side of the building in Cases 2 & 3. The wall diaphragms in the first unit are incapable of carrying the total applied shear; it is the combined shear resistance of the walls of the units which resist the applied action. Therefore, residual shear has to be transferred across the party wall to the subsequent units.

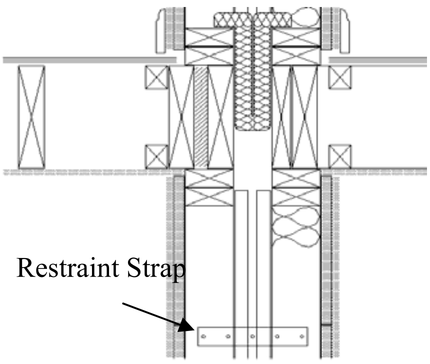


Figure 2 Party wall detail

A typical party wall detail is shown in Figure 2. As a result of thermal and acoustic requirements the two leaves are unconnected for the full height except for 3mm (max) thick, light metal restraint straps tying the two leaves together. These straps are spaced at minimum horizontal centres of 1.2m, one row per storey height at or near ceiling level (TRADA, 2001).

The connection between the metal strap and the wall stud is the critical design criteria and is normally made by 3no 3.35mm diameter 63mm long galvanised wire nails. The permissible strength of this connection is 1.65kN (calculated in accordance with BS EN 1995 and factored in accordance with BS 5268-2:2002). Therefore, the permissible residual shear which can be transferred is 1.4kN/m per storey height.

For cases 1 & 2 the transfer of shear force from unit 1 to 2 is equal (the total applied shear force on case 2 is in excess of this as a result of units 2, 3 & 4 protruding past unit 1). The party wall length in both cases is 7.728m therefore approximately 6 straps per storey can be applied, 12 straps in total. As a result the total shear which can be transferred is 19.83kN which is in excess of the residual force (9.88kN).

It is demonstrated that continuity across the party walls for these cases is achieved through the application of restraint straps. However, it is to be noted that in certain design scenarios transfer of residual shear would be critical.

4 Shear Connections and Holding Down

4.1 Wall plate to sole plate

The level of shear transferred to the sole plate is dependent on the connection between the wall panel footer and sole plate (Figure 3). A typical nailed connection between wall panel footer and sole plate is 90x4.00mm Skewed Galvanised Wire Nails at 300mm centres (between wall studs).

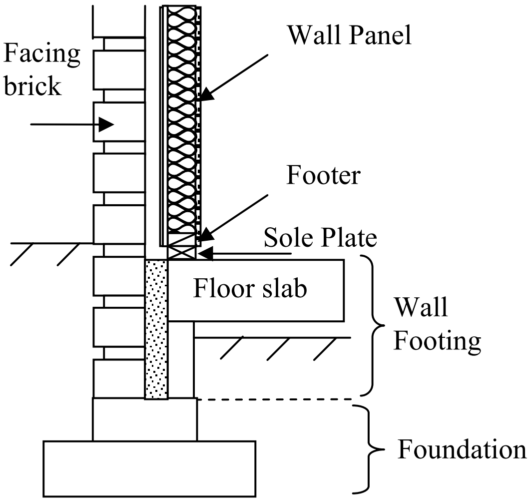


Figure 3 Typical foundation detail

The resistance to shear which can be allowed for in accordance with BS 5268-2:2002 is 410.6N per nail which equates to 1.369kN/m run. Therefore, although in Case 1 the resistance of wall 2 is stated as 42.10kN this is equal to 3.98kN/m run which requires an increase in nailing specification. However, in this case there is a degree of redundancy and the nailing specification is suffice as the wall only requires to transmit 0.9kN/m run. The allowable shear transfer column of Table 3 shows the revised design racking resistance of the systems as a result of the nailing specification.

It is shown that in Case 3 when the wind is acting on the side design failure occurs, therefore increased nailing of the wall panels to the sole plate is required.

4.2 Sole Plate to Foundation

Shear connections come in a manner of forms but the ones most commonly used for domestic dwelling construction in the UK are:

1. Hardened Zinc Plated Nails: shot fired using power actuated systems.
2. Screw Anchors: formed from carbon steel and self tapping.
3. Express Nails: formed from spring steel and hammer fixed into pre-drilled holes.

Shown in Figure 4 are a range of available shear fixings. Table 5 contains information from test conducted in accordance with BS EN 409:1993 to determine the Tensile Strength and Yield Moment capacity of the fasteners shown. Also contained in Table 5 is the yield moment capacity of the fasteners calculated in accordance with BS EN 1995 using the characteristic tensile test results.

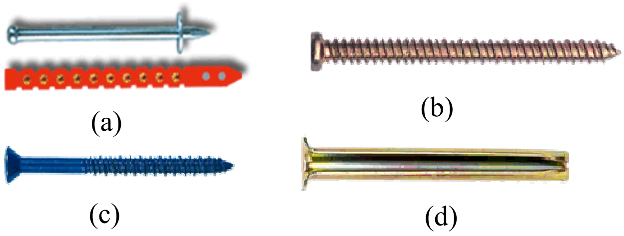


Figure 4 (a) KMN Low velocity shot fired nail;
(b)KF masonry screw anchor;
(c)MSC masonry screw anchor;
(d) EXPN express nail.

It is demonstrated in Table 5 that the percentage difference between calculated and test determined characteristic yield moment is relatively consistent. The reason for the EXPN express nail having a higher level of error will be due to the required interpolation to determine an equivalent root diameter which was back calculated from the measured cross sectional area.

Due to the relative consistency in yield moment determined from tensile strength, BS EN 1995 was used to calculate the characteristic connection strength for a range of fixing diameters and then converted to permissible design values in accordance with BS 5268 (Figure 5).

Table 5 Shear connection information

Type	Fixing Info			Yield Moment			
	Root Diameter	Length	Characteristic Tensile Strength	BS EN 1995 Calculated	Characteristic Test Result		Factored calc' yield moment
	mm	mm	N/mm^2	Nmm	Nmm	% Diff	Nmm
KMN	3.79	72	1523	14594	22487	35	22836
KF	5.27	100	1134	25610	40119	36	40072
MSC	4.29	82		13680	19740	31	21405
	3.47	82	1037	7910	11828	33	12377
EXPN	5.60	90	731	19329	35376	45	30243

For each connection calculation the timber element was considered to be a 38mm deep C16 grade timber, as this is normal sole plate material. The length of all the fixings was set to 80mm to allow for a truer comparison. All the fixings were considered to be fixed without pre-drilled holes with the exception of the EXPN express nail which was considered to be pre-drilled.

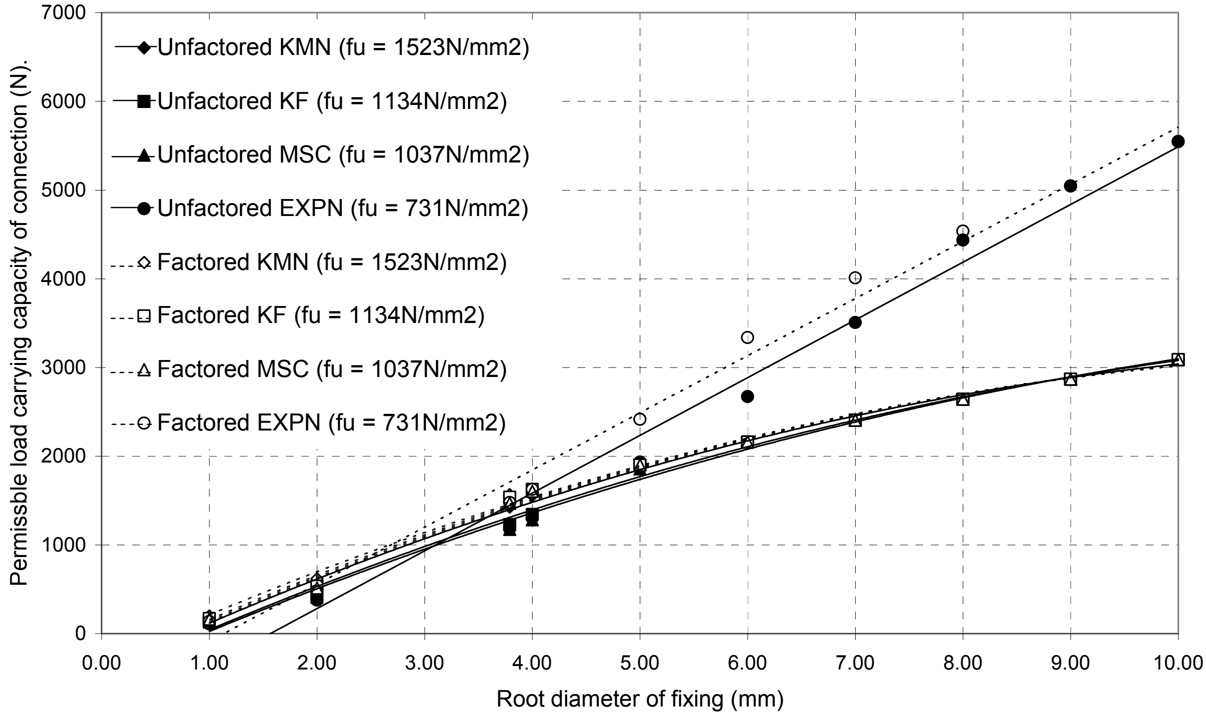


Figure 5 Un-factored calculated connection strength for a range of fixing diameters

As a result of the BS EN 1995 method of determining yield moment capacity from tensile strength tending to provide a consistent underestimation, for the range of fasteners under consideration, a constant factor of 1.56 was applied to the calculated yield moment (Table 5). The factored calculated yield moment was then used to determine the permissible connection strength for the same parameters as before and plotted (Figure 5).

From Figure 5 the following is concluded:

- Applying a correction factor to the calculated yield moment values results in a higher level of consistency in results.
- Although using the characteristic tensile strength of the fastener, the information normally available from suppliers, to determine yield moment tends to result in an underestimation. However, it is not critical in design as characteristic embedment strength tends to govern.
- The reason for the EXPN Express nail having a linear relationship for the given range is as a result of it being pre-drilled. Pre-drilling will increase the connection capacity at higher diameters as a result of reduced splitting of the timber.
- The use of BS EN 1995 design methods for this type of connection will border on the conservative side and therefore be safe. For true design values shear tests would have to be carried out.

Normal spacing of anchors is between 300 and 600mm depending on the nature of the fixing. This will in most cases be an over specification of what is required if the shear transfer between the wall plate and sole plate is 1.37kN/m run. However, shear fixings also add to the overall robustness of the system and provide added resistance to system sliding and, depending on the pull out resistance, overturning.

4.3 Holding Down

The applied shear force on a wall assembly results in an overturning moment which has to be counteracted by holding down anchorage. Shear connections are not designed to transmit vertical forces to the foundation, although some capacity can be achieved (Prion & Lam, 2003). It is normal practice in the UK for holding down straps to be employed Figure 5. Holding down straps connect the vertical end stud to the foundation. They are normally attached to the end stud by means of 6 no 65x3.35 ring shank nails or equivalent, the permissible connection strength (3.29kN) of which is the limiting criteria in design, and have their L-shaped end placed under the masonry cladding to create a holding down resistance.

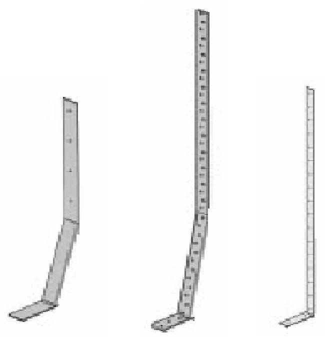


Figure 5 Holding down straps

A level of redundancy can be applied to the required holding down of a wall. According to Andreasson, S (2000) it is reasonable to assume that the dead load applied within the reach of the sheathing panel closest to the end is counteracting the uplift force.

From the design cases considered it is therefore concluded that if redundancy is considered the application of one holding down strap at the end of each wall panel is sufficient, and is indeed in most cases an over-specification. However, for standardisation and safe practice the application of one holding down strap at the end stud of each racking wall panel and at large openings is advised.

5. Conclusions

The following are the main conclusions:

- *Stiffness Proportionality:* is achieved by giving due consideration to the level of stiffness a wall diaphragm brings to the system as a result of its make-up, dimensions and distance from the geometric centre of the system. Where possible, especially in systems where shear wall resistance is close the applied shear force it is important to have stiffness proportionality.
- *System Continuity* The strength of connection can be critical when considering system continuity. In particular connections across party walls are highlighted, this connection can only be considered sufficient if the residual shear from the first block is less than the strength of the connection between the blocks.
- *Shear Connection (wall plate to sole plate):* The connection between the wall plate and the sole plate can be critical and should be checked in design.
- *Shear Connection (sole plate to foundation):* The use of BS EN 1995 calculation methods will tend to be on the conservative side. However, using the characteristic tensile strength of the fastener, which is most often the information available from suppliers, to determine yield moment will not result in overly conservative design.
- *Holding Down:* A level of redundancy can be applied to the required holding down of a panel due to the self weight of the system.

6. Acknowledgements

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7. References

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