Building 1
Grosvenor Court, Hipley Street
OLD WOKING, SURREY
GU22 9LL

Silvatec Design

Tel: +44 (0) 1483 769518
Fax: +44 (0) 1483 770863
E-mail: design@silvatecdesign.com
Internet: www.silvatecdesign.com

Headbinders on Timber Frame Wall Panels

Dr. S. Atek, T. Knight, J. A. Figueira

Rev A 22-01-10

Headbinders on Timber Frame Wall Panels

Dr. S. Atek, T. Knight, J. A. Figueira

Silvatec Design Ltd. Building 1, Grosvenor Court, Hipley Street, Old Woking, Surrey, GU22 9LL

Aim

To investigate limitations of double head rails with regard to stud alignment to joists/trusses and relate these limits to the potentially high loads possible on internal load bearing walls subject to long continuous spans for joists/trusses.

1. Introduction

Timber frame stud walls are the loadbearing element of any timber frame building. Their primary purpose is to transfer dead and imposed loads from the roof and floors down to the foundations. They have obvious secondary roles with regards to racking resistance, fixing for claddings and housing the insulation but the fundamental role is the ability to carry the vertical loads. It is necessary for these walls to be erected plumb to ensure that all studs are axially loaded to avoid induced bending from eccentricity and also erected square to ensure flush plasterboard and final finishes. In order to aid this it is common for timber frame walls to be fitted with a secondary head rail, or head binder, which is fitted to the top of panels and laps over continuous panel connections and at corners with 90 degree abutments. In the early years of timber frame construction the span of domestic joists was limited to the stock lengths of softwood timbers, the engineering support available to the market and an unwillingness to use timber frame for more generously sized housing or commercial developments. This meant that generally floor joist spans were of a modest length and a modest loading. In these scenarios it had been acknowledged by the industry that if you needed to, it was acceptable to misalign joists or trusses with the loadbearing studs below and allow the double head rail to carry the loads. As the industry has expanded it has seen a multitude of engineered joists become readily available offering joist spans of far greater magnitudes and continuous span joists of up to 14m. The concept of timber frame and its obvious benefits has also lent itself to be used for commercial buildings subject to significantly increased imposed loads and also to larger residential buildings which require longer joist spans.

Although the limitations of timber frame have been greatly reduced through the advancement of re-engineered timbers there is still an industry expectation that the misalignment of joists or trusses with the stud below is acceptable as long as a head binder is used. This is something that is often seen on timber frame sites and can be seen in the "2008 NHBC Standards" which notes in chapter 6.2 "*If head binders are not provided joists and roof trusses, including girder trusses and similar loads, should bear directly over studs*". This would suggest that it is acceptable for a joist or truss to be misaligned should a head binder be used. The TRADA publication "*Timber Frame Construction – 4th Edition*" has a different view and in chapter 4.1.2.2 states "*A double head rail, i.e. top rail and head binder, with studs at 600mm centres may allow loads from single floor joists or rafters to occur between studs. Structural calculations will identify when shear and bending force are not adequately transferred through the head binder and top rail to the adjacent studs".* The latter part of this statement is the intention of this report. With increased joist spans and commercial loadings, what are the limitations of a double head rail with respect to misaligned joists?

2. Methodology

The simulations were conducted by applying 1kN joist reactions on a set of wall panels of lengths ranging from 600mm to 5400mm and consisting of single 38x89 mm studs of strength class C16 spaced at 600mm between centres. Two different scenarios for joists spacing have been considered: the case of 400mm centres joists and the most common case of 600mm spacing.

Both bending and shear deflection were used in deriving the static model of the double rail used to calculate the support reactions, which in turn are applied to the bending moment and shear forces functions.

The basis behind using a unity load derives from the linear proportionality of the shear, bending moment and deflection to the applied forces and reactions as shown in equation (1), (2) and (7). The reactions are proven to be proportionate to the applied forces within the provisions of the boundary conditions.

The applied shear force S, and the bending moment M are formulated as follows:

$$S(z) = R_1 + \sum_{i=2}^{n} R_i \delta(z - x_i) - \sum_{j=1}^{m} F_j \delta(z - a_j) \ \ldots \ (1)$$

$$M(z) = R_1 z + \sum_{i=2}^{n} R_i [z - x_i] - \sum_{j=1}^{m} F_j [z - a_j] \ldots (2)$$

Where R_i is the *i*-th reaction of the double rail, F_j is *j*-th joist reaction, x_i is the position of R_i, a_j is the position of F_j and z is an arbitrary position on the double rail. In addition, we have

$$\delta(x) = \begin{cases} 1 \ for \ x \geq 0 \\ 0 \ for \ x < 0 \end{cases} \ where \ x \in \mathbb{R} \ldots (3)$$

$$[x - y] = \begin{cases} x - y \ for \ x \geq y \\ 0 \ for \ x < y \end{cases} \ where \ x, y \in \mathbb{R} \ldots (4)$$

The sign convention that has been used in equations (1) and (2) is as follows: the positive shear force has the opposite direction of gravity and the positive bending moment is clockwise. The same convention is used in the graphical results.

The relationship between the bending deflection and the bending moment is shown below [1]

$$M = EI \frac{d^2 v}{dz^2} \ldots (5)$$

The shear deflection is expressed as follow [1]

$$\zeta = \int \frac{\beta}{GA} S \, dz \ldots (6)$$

[1] Strength of Materials for Civil Engineers. T.H.G Megson. Nelson Edition. 1980 ISBN 0-17-761081-6

Where E is the young modulus, I the second moment of area, β the form factor, G the shear modulus and A is the cross-sectional area. For rectangular cross-sections $\beta = \frac{6}{5}$

The total deflection ξ is presented as follow

$$\xi(z) = \nu(z) + \zeta(z)$$
$$= \frac{1}{EI}\iint R_1 z\, dz^2 + \frac{1}{EI}\iint \sum_{i=2}^{n} R_i[z - x_i]\, dz^2 - \frac{1}{EI}\iint \sum_{j=1}^{m} F_j[z - a_j]\, dz^2 + \int \frac{\beta}{GA} R_1\, dz$$
$$+ \int \frac{\beta}{GA}\sum_{i=2}^{n} R_i \delta(z - x_i)\, dz - \int \frac{\beta}{GA}\sum_{j=1}^{m} F_j \delta(z - a_j)\, dz \ldots (7)$$

The reactions R_i and arbitrary integration constants are the solutions of the deflection equation at the boundary condition which in turn are reformulated in equations (1) and (2).

The resulting bending moments, shear forces, support reactions and combined bending and shear deflections are used to calculate the permissible joist reactions according to BS5268 Part 2 where grade stresses and modulus of elasticity are modified using the following factors: K_3=1.00, $K_7=(300/76)^{0.11}$=1.163. The headbinder is assumed to be working in low moisture environment as defined by service classes 1 and 2.

The analysis excludes the load sharing factors K_8 and K_9 which modifies the minimum value of the modulus of elasticity as these conditions are not guaranteed to be met within the provision of the British standards.

Adequate nailing is assumed to be provided as to allow all the stresses to be equally distributed throughout the combined section of the head rail and the headbinder. The calculations of the section modulus, the shear area and the second moment of inertia use the augmented section as a basis for calculations.

All joists reactions are assumed to be equal resulting from a floor with no span change in length and direction and no effect from the decking or wall sheathing was taken into account.

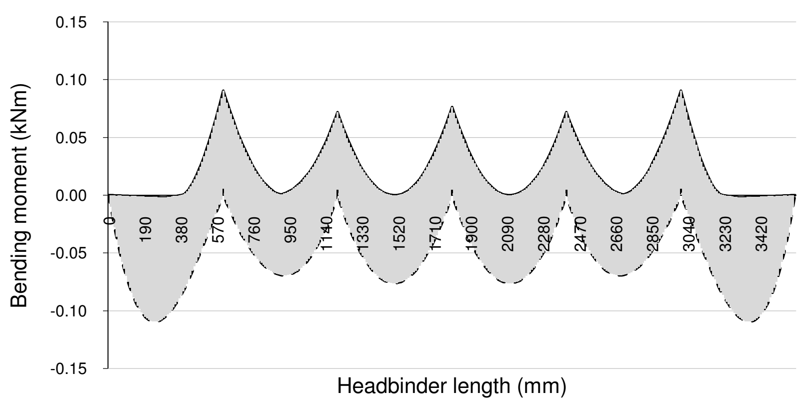

Figure 1: Bending moment envelope: case of 600mm C/C joists over a 3.6m Long wall.

Figure 1 shows the bending moment envelope diagram of a double rail in the case of a floor with 600mm C/C joists spacing supported by a 3.6m long wall.

The envelopes correspond to the maximum magnitudes of the bending moments, shear forces and deflections (represented in continuous lines) and the minimum magnitudes (represented in dotted lines) in algebraic value along each position of the double head rail for all possible module shift scenarios. All the possible bending moments, shear forces and deflections diagrams as a function of the module shift are enclosed within these two curves.

We notice that the maximum bending moments in absolute values are located within the outer bays of the wall. The same observation is drawn for the different wall panels used in the simulations. For 600mm spaced joists, the position of the worst bending moment for all possible module shift scenarios occurs within 300mm from the edges for a single span panel and varies between 230mm and 240mm from the panel edges for walls ranging between 1200mm long and 5400mm long.

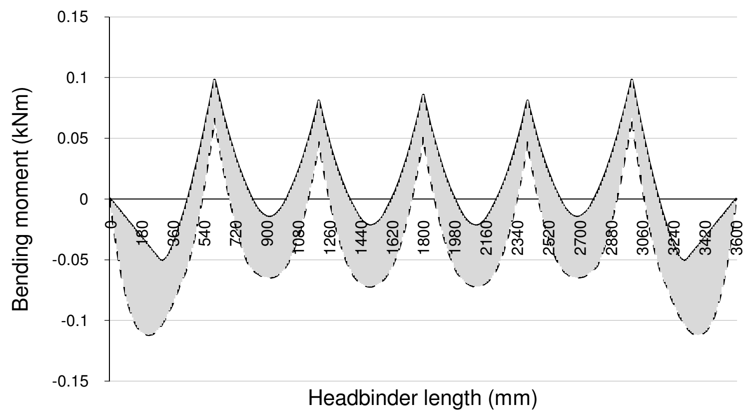

Figure 2: Bending moment envelope: case of 400mm C/C joists over a 3.6m wall.

When the joists spacing is set to 400mm, it is also observed that the locations of the highest bending moments occur within the outer bays of any wall panel as this is demonstrated in Figure 2. The worst bending moment for all possible module shift scenarios occurs at 200mm from the edges for a single span panel, 210mm for a double span wall panel and varies between 200mm and 220mm from the panel edges in the case of walls ranging between 1800mm long and 5400mm long.

Figure 3 shows the shear forces envelope which for all module shifts scenarios is no more than 1kN for all analysed wall lengths.

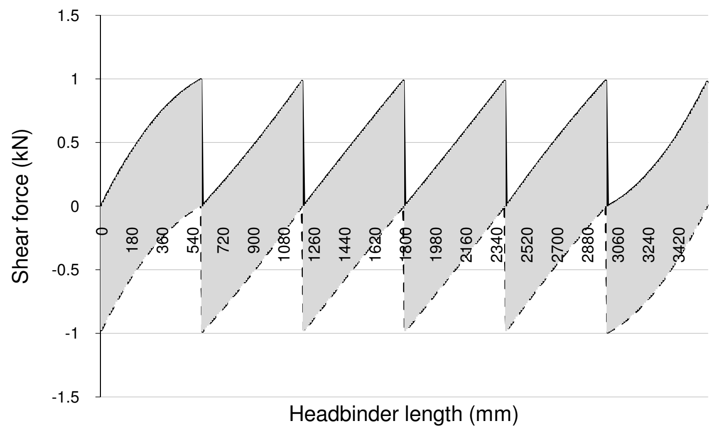

Figure 3: Shear force envelope: case of 600mm C/C joists over a 3.6m wall.

Figure 4 shows that the joist spacing of 400mm induces a noticeable higher shear force at the edges of the panel. The maximum value does not reach 1.5kN which implies a better load redistribution compared to the 600mm spaced joists. In fact, the equivalent uniformly distributed load applied in the 400mm c/c joists configuration is 2.5kN/m. The equivalent joists reactions from a 600mm c/c configuration is 1.5kN resulting in a maximum shear force value of 1.5kN.

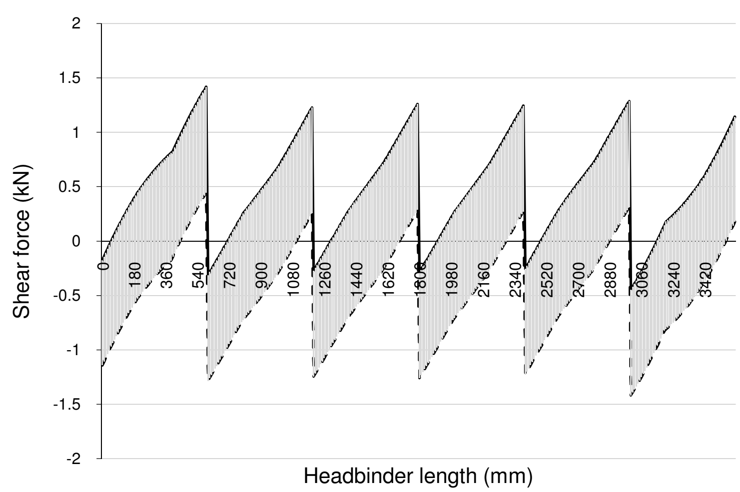

Figure 4: Shear force envelope: case of 400mm C/C joists over a 3.6m wall.

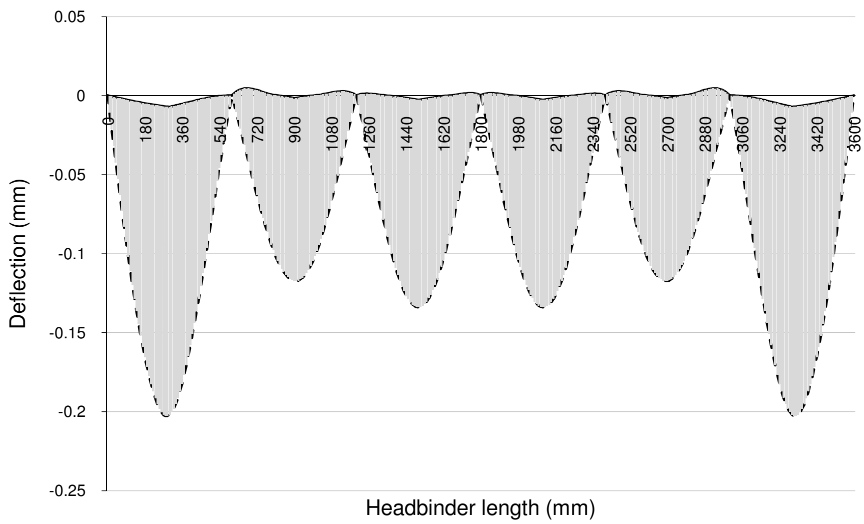

Figure 5: Combined bending and shear deflection envelope: case of 600mm C/C joists over a 3.6m wall.

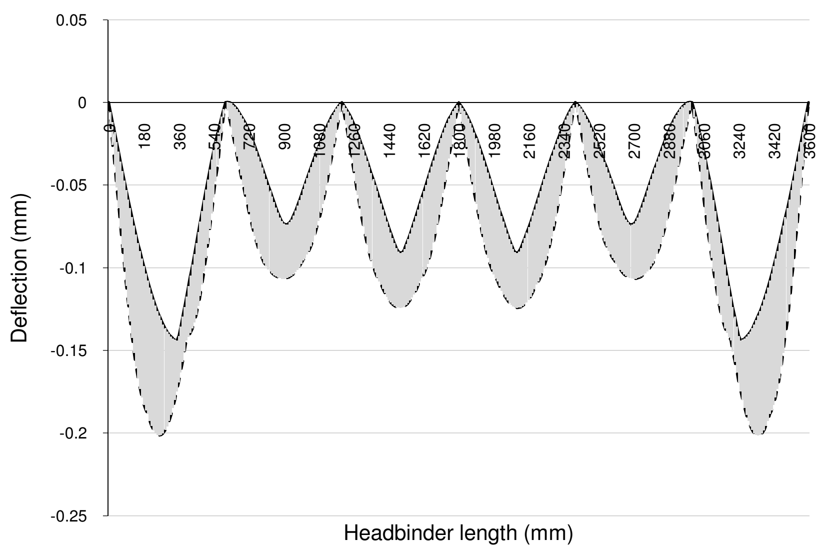

Figure 6: Combined bending and shear deflection envelope: case of 400mm C/C joists over a 3.6m wall.

The calculation of the deflection (for example: Figure 5 and Figure 6 for a 3.6m wall) shows that unlike strength considerations the deflection limits remain far beyond the magnitude of deflection caused by any module shift scenario. The visual effect of the deflection remains unnoticeable even at headbinder joints where the section is halved.

The same observation has been drawn regarding the compression perpendicular to grain capacity thus any risk of inducing failure is solely due to the effect of shear and bending.

3. Results

The results obtained from the simulations have been processed to derive the maximum loads applicable to a double rail in various conditions. Each case is taking into account shear stress, bearing stress, deflection and bending moment of the rail. The maximum value shown in kN is the point at which the rail will fail. These examples will be applicable to both external and internal walls. As the behaviour of the timber in the head rail will change depending on whether it is a single span or continuous span the following results show the limitations of the rail in the various wall panel lengths i.e. a 600mm wall then a 1200mm wall and so on in a modular format.

Note: In the case of BS EN 1995-1-1[2], the contribution of some joist reactions to the total shear force may be disregarded if those joists occur within a distance from the stud, equivalent to the double head rail depth, *h*=76mm (or 38mm if the headbinder joint is made over the stud).

Case 1

This scenario is for joists at 400mm centres over a double rail supported over a stud and with studs at 600mm centres (Figure 7). The module starts at 0mm.

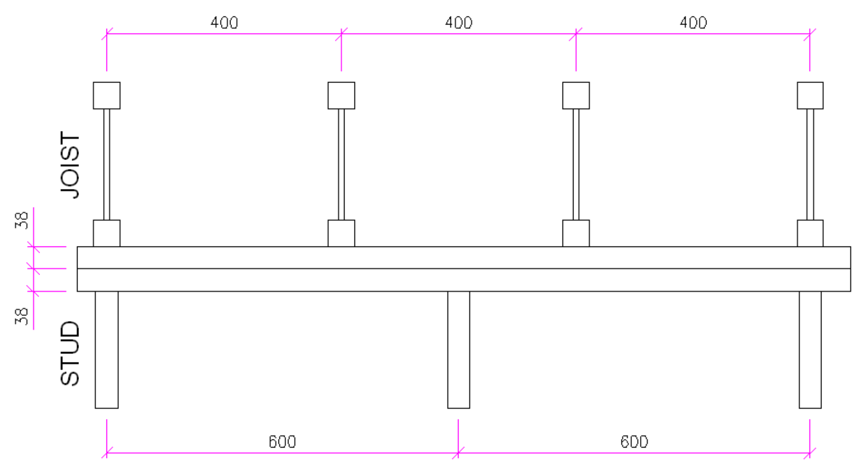

Figure 7: Case of joists at 400mm centres over a double rail supported over a stud and with studs at 600mm centres with no module shift.

[2] BS EN 1995-1-1:2004 Eurocode 5. Design of timber structures. General. Common rules and rules for buildings.

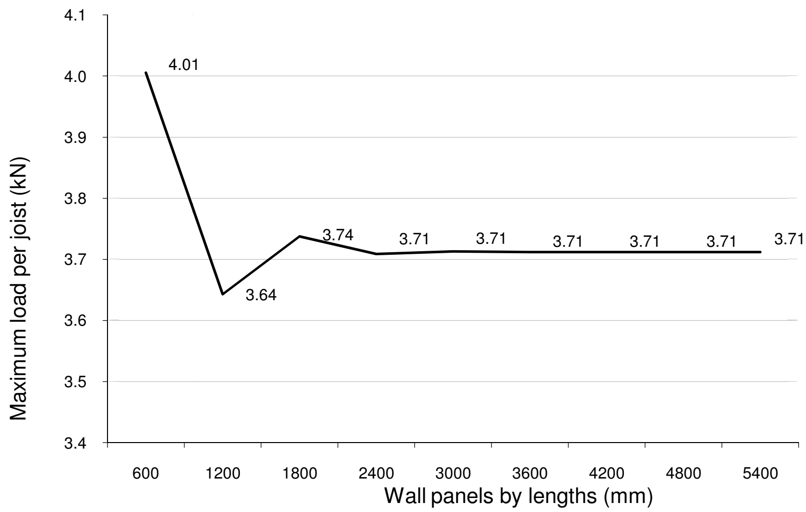

Figure 8: Allowable joists loads spaced at 400mm c/c supported by 600mm c/c studs with no module shift

The double rail is strongest in a single span condition on a short wall 600mm long with a maximum reaction from a joist or truss of 4.01kN. But as the wall and rail increase in length to a more common 2400+ mm then the maximum allowable reaction reduces to 3.71kN. The weakest situation is for a single wall 1200mm long where the maximum reaction is 3.64kN (Figure 8).

Case 2

This scenario is for joists at 400mm centres over a double rail supported over a stud and with studs at 600mm centres. The module is set to the least favourable position in terms of loading the rails (Figure 9).

This example will set the maximum values for non alignment of joist/truss to stud without need for calculation, this is based on the head binder being continuous.

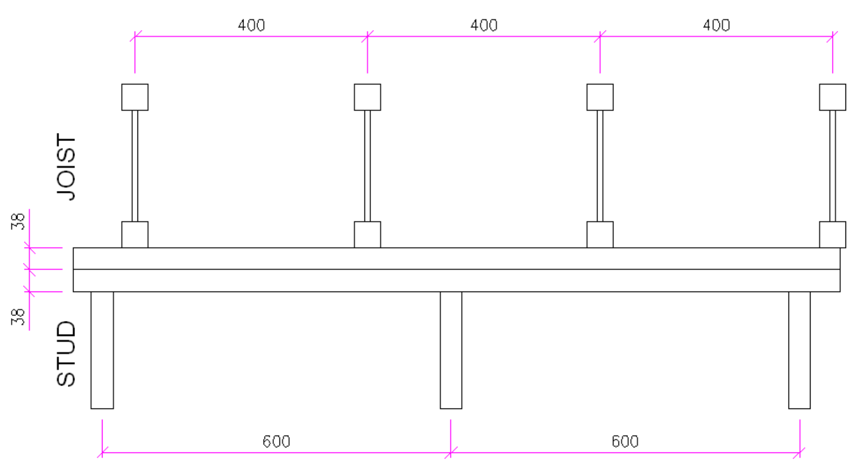

Figure 9: Case of joists at 400mm centres over a double rail supported over a stud and with studs at 600mm centres with module shift.

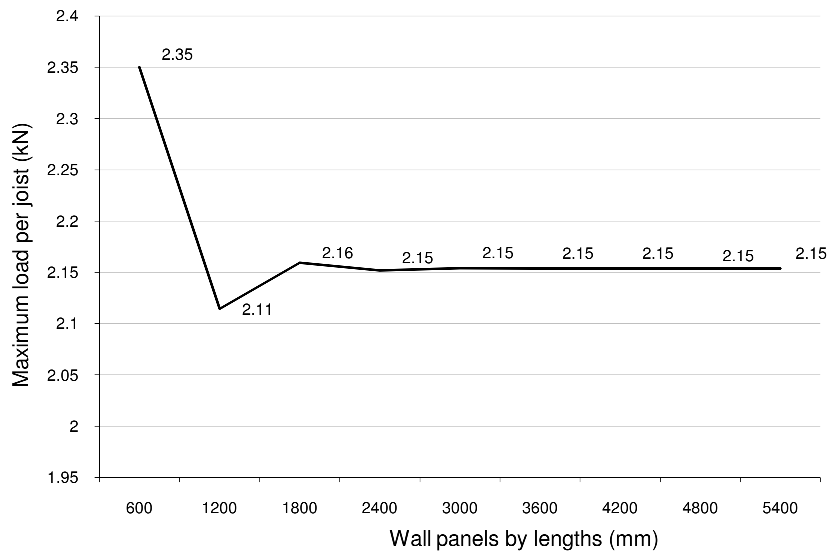

Figure 10: Allowable joists loads spaced at 400mm c/c supported by 600mm c/c studs with the worst module shift

The double rail is strongest in a single span condition on a short wall 600mm long with a maximum reaction from a joist or truss of 2.35kN. But as the wall and rail increase in length to a more common 2400+ mm then the maximum allowable reaction reduces to 2.15kN. The weakest situation is for a single wall 1200mm long where the maximum reaction is 2.11kN (Figure 10).

Case 3

This scenario is for joists at 400mm centres over a discontinuous (up to 10mm gap) double rail or headbinder with studs at 600mm centres in the worst misalignment case (Figure 11). The discontinuity of the headbinder is a distinct possibility as erectors could use stock lengths which in are generally limited to a maximum of 4.8m, and in some instances when large panels are used, the panel rails are jointed with a truss plate of negligible shear capacity in that plane. The double rail will fail with a load over 1kN over all different wall panels ranging between 600mm long and 5400mm long (Figure 12).

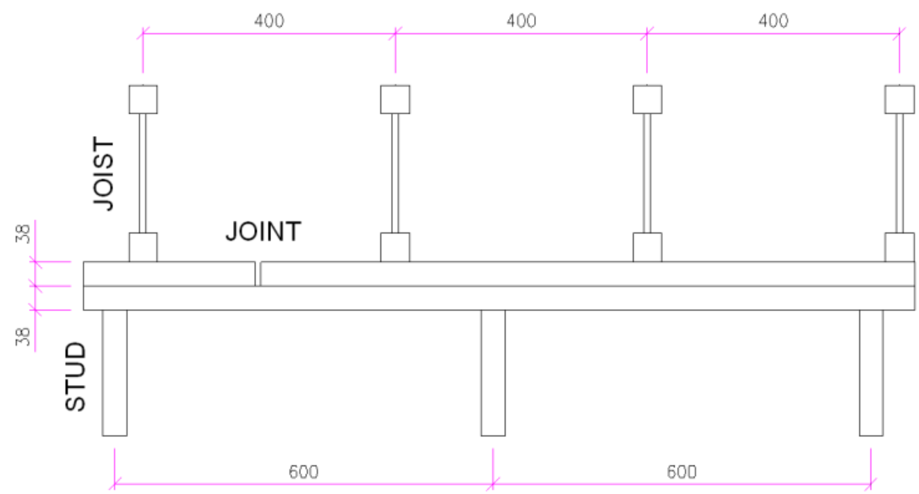

Figure 11: Case of joists at 400mm centres over a double rail with a joint supported over a stud and with studs at 600mm centres with module shift.

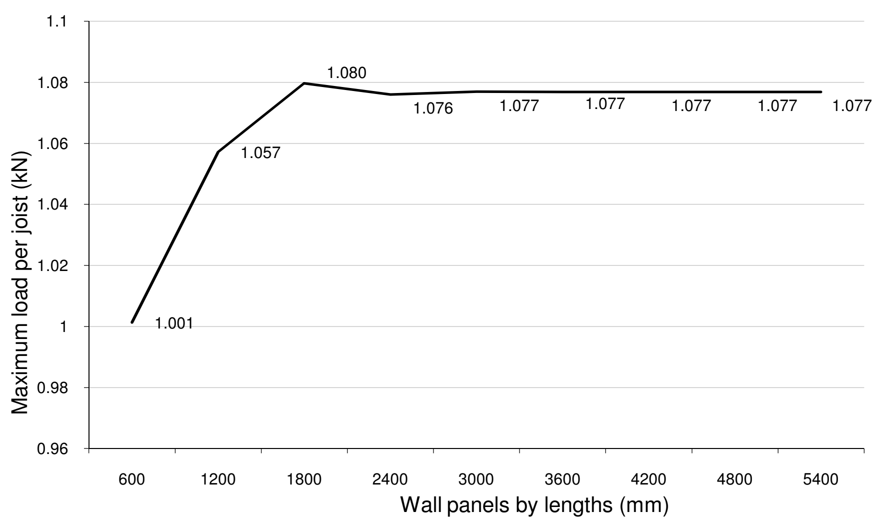

Figure 12: Allowable joists loads spaced at 400mm c/c supported by 600mm c/c studs with a break on the rail (worst module shift)

Case 4

This scenario is for joists at 600mm centres over a double rail supported over a stud and with studs at 600mm centres (Figure 13). The module is set to the least favourable position. The double rail will fail with a load over 3kN in all different wall panels ranging between 600mm long and 5400mm long (Figure 14).

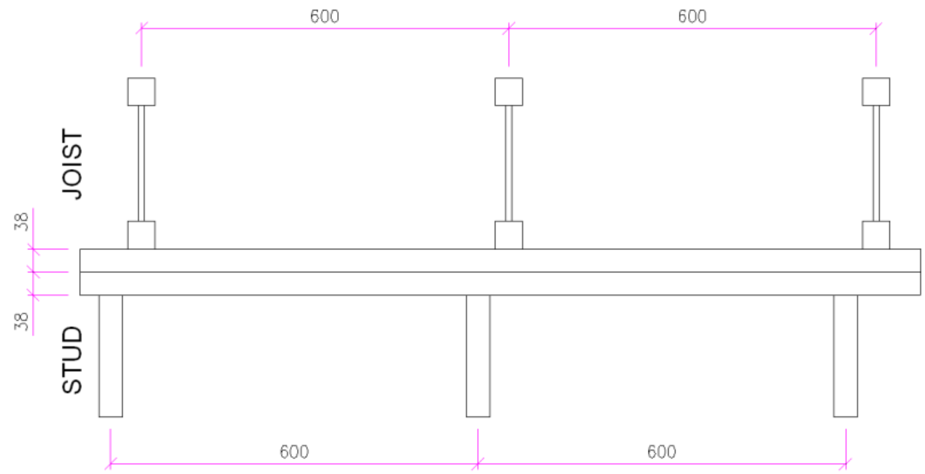

Figure 13: Case of joists at 600mm centres over a double rail supported over a stud and with studs at 600mm centres with module shift.

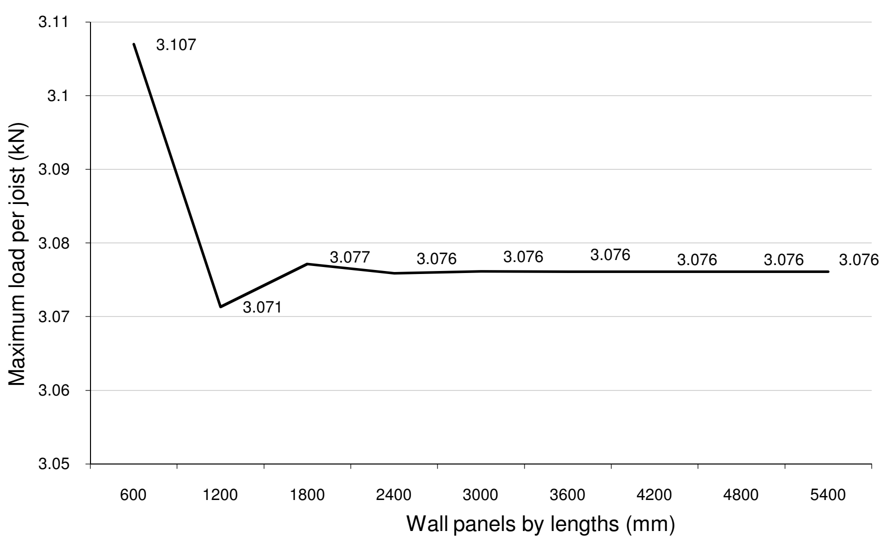

Figure 14: Allowable joists loads spaced at 600mm c/c supported by 600mm c/c studs with the worst module shift

Case 5

This scenario is for joists at 600mm centres over a double rail including a break point (Figure 15), with studs at 600mm centres. This is the worst case misalignment. The double rail will fail with a load over 0.89kN in a single bay wall and 1.22kN in a 4-bay wall or longer (Figure 16).

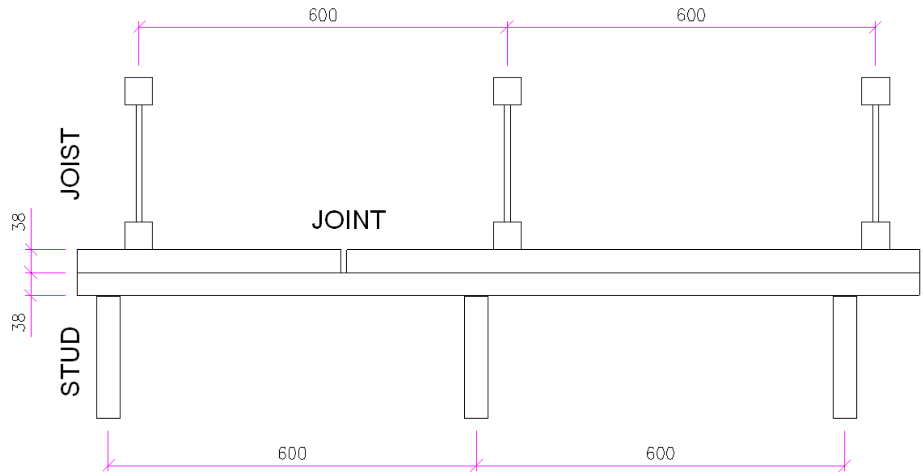

Figure 15: Case of joists at 600mm centres over a double rail with a joint supported over a stud and with studs at 600mm centres with module shift.

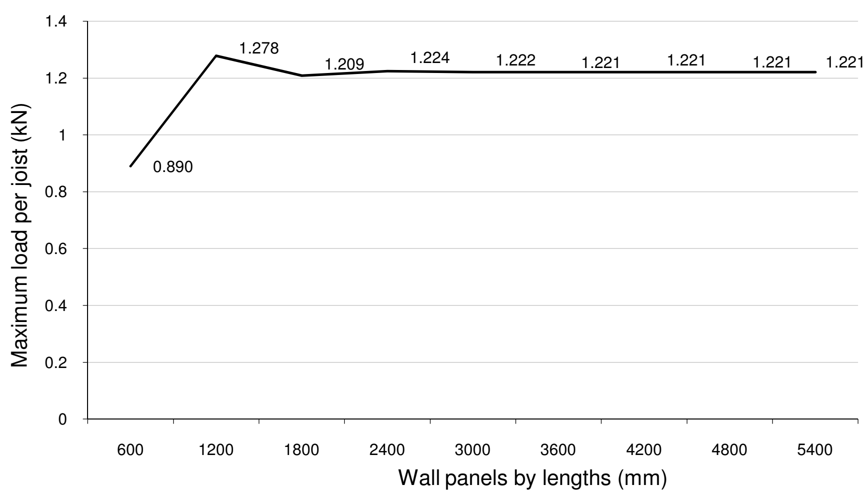

Figure 16: Allowable joists loads spaced at 600mm c/c supported by 600mm c/c studs with a break on the rail (worst module shift)

Wall panel length	Maximum allowable joist reaction (kN): Worst misalignment of joists at 600mm c/c	Maximum allowable joist reaction (kN): Worst misalignment of joists at 600mm c/c in case of discontinuous headbinder/rail	Maximum allowable joist reaction (kN): Worst misalignment of joists at 400mm c/c	Maximum allowable joist reaction (kN): Worst misalignment of joists at 400mm c/c in case of discontinuous headbinder/rail
600 mm	3.107	0.890	2.350	1.001
1200 mm	3.071	1.278	2.114	1.057
1800 mm	3.077	1.209	2.159	1.080
2400 mm	3.076	1.224	2.152	1.076
3000 mm	3.076	1.222	2.154	1.077
3600 mm	3.076	1.221	2.154	1.077
4200 mm	3.076	1.221	2.154	1.077
4800 mm	3.076	1.221	2.154	1.077
5400 mm	3.076	1.221	2.154	1.077

Table 1: Summary of headbinder failure loads for worst case scenarios with studs at 600mm c/c

The following graphs in Figure 17 and Figure 18 show the permissible uniformly distributed linear loads (UDLs) for: (a) the case of a continuous headbinder and head rail, (b) the case of a discontinuous headbinder or head rail. The supporting wall is assumed to comprise single studs 38x89mm spaced at 600mm for all scenarios. The references 600 NS,400 NS,600 S, 400 S stand for 600mm c/c joists spacing with no module shift, 400mm c/c joists spacing with no module shift, 600mm c/c joists spacing with worst module shift, 400mm c/c joists spacing with worst module shift respectively. In the case of 600 NS the maximum load has been computed based on the bearing capacity of the rail on a single 38x89mm stud.

The resulting UDLs have been obtained from the minimum values of the allowable joists reactions on all wall types ranging from 600mm to 5400mm.

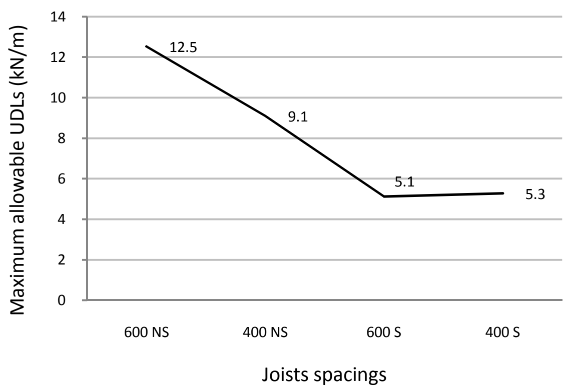

Figure 17: Case of a *continuous* headbinder and rail

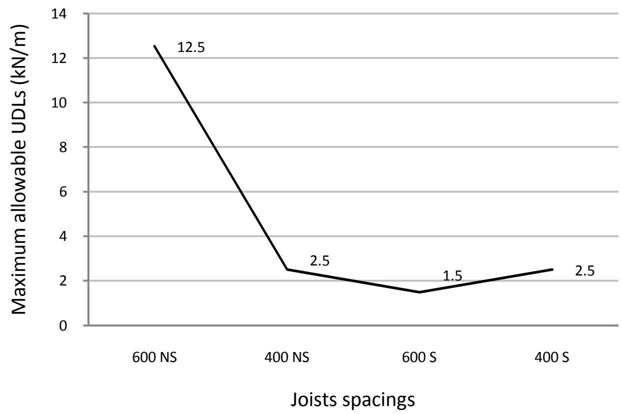

Figure 18: Case of a *discontinuous* headbinder or rail

In cases of continuous joist spans internal 90mm walls will carry more loads and will be more vulnerable. The following joist lengths have been estimated in the case of <u>*mid-span*</u> support in different loading scenarios taken from BS 6399 Part 1 (Figure 19). The latter include domestic loading, nursing homes common areas, hotels rooms and hotels common areas. Partition loading has been included in the calculations. The results were drawn for two cases: (a) the case of a continuous headbinder and head rail (Figure 20) and (b) the case of a discontinuous headbinder or head rail (Figure 21).

The graphs for the discontinuous headbinder or head rail have been derived using the worst possible module shift scenario with the discontinuity being at the position that yields the worst effect. This is also applicable to Figure 18.

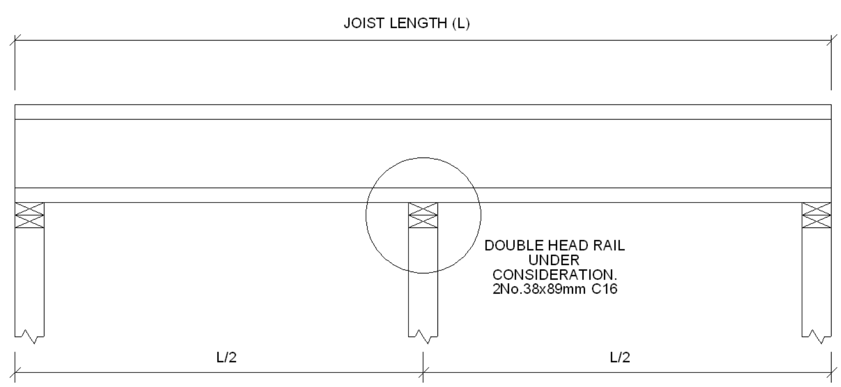

Figure 19: Double rail under joist with mid-span support

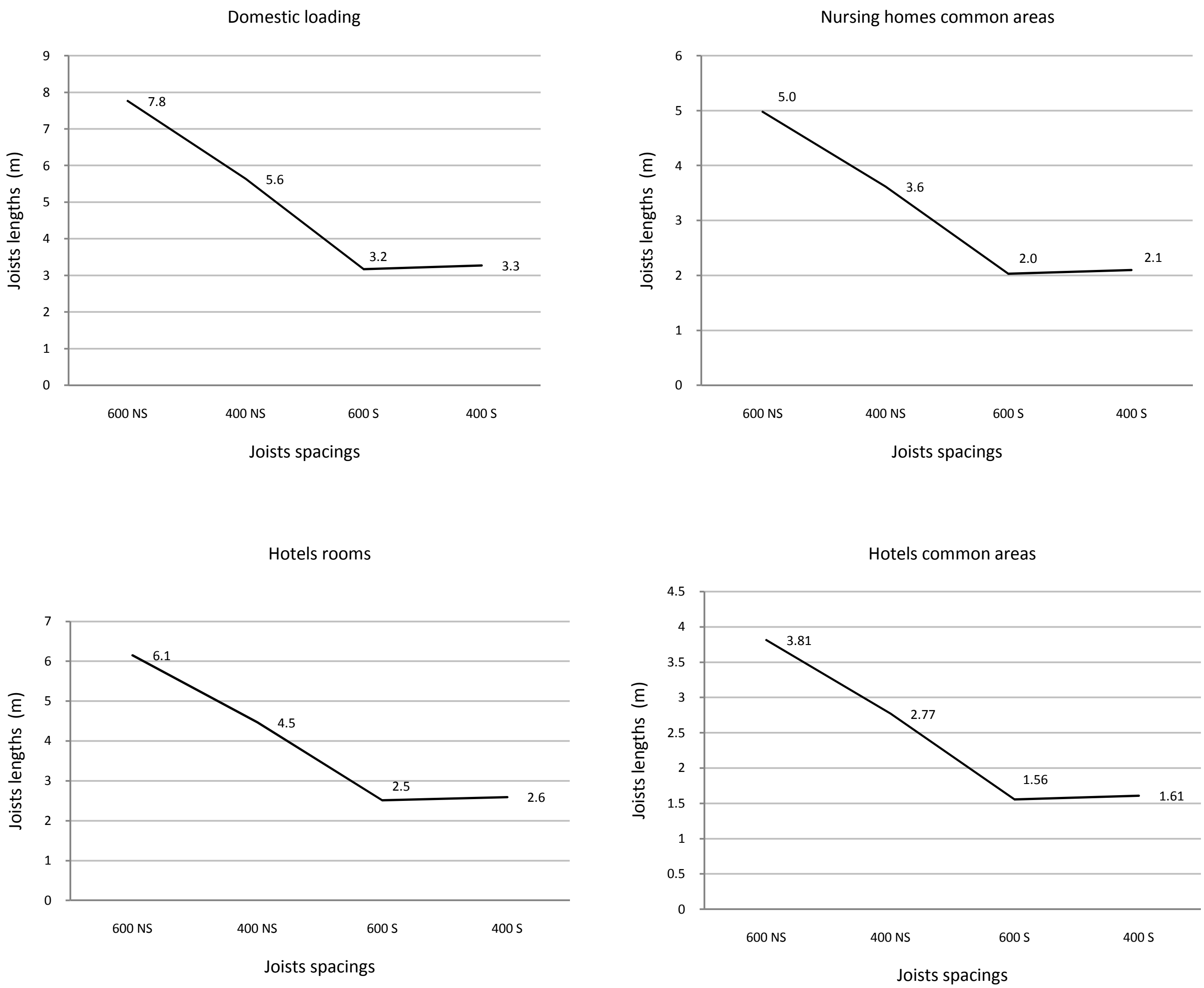

Figure 20: Maximum joists length with mid-span support: (a) case of *continuous* headbinder and head rail

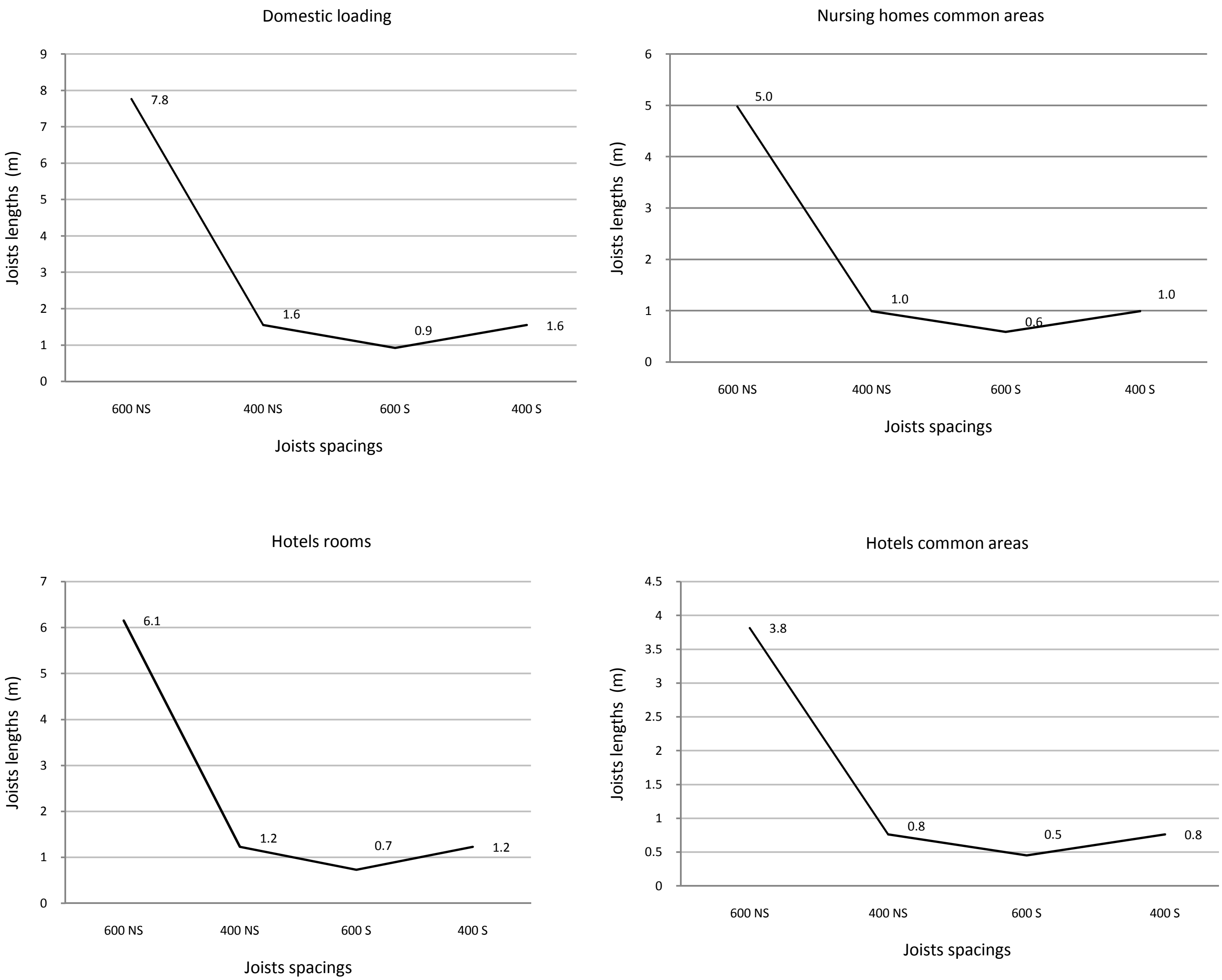

Figure 21: Maximum joists lengths with mid-span support: (b) case of *discontinuous* headbinder or head rail

The results in Figure 20 and 21 are indicative and show the worst possible scenario for walls ranging between 600mm and 5400mm long and supporting the joists at half their length. It is necessary to perform calculations for any specific case of wall panel length and the appropriate joist reaction depending on the position of the supporting wall to derive the appropriate joist length in any particular loading condition as defined by BS6399 Part 1.

When the value of the module shift and the position of the headbinder joint are already known it is possible to derive the appropriate maximum joist length which in the case of a mid-span supported joist will be greater than the lengths given in figure 21.

Another case has been investigated (Figure 22) where the joists are simply supported by two 38x90mm wall panels fitted with double rails. The results are presented in Figure 23 and Figure 24.

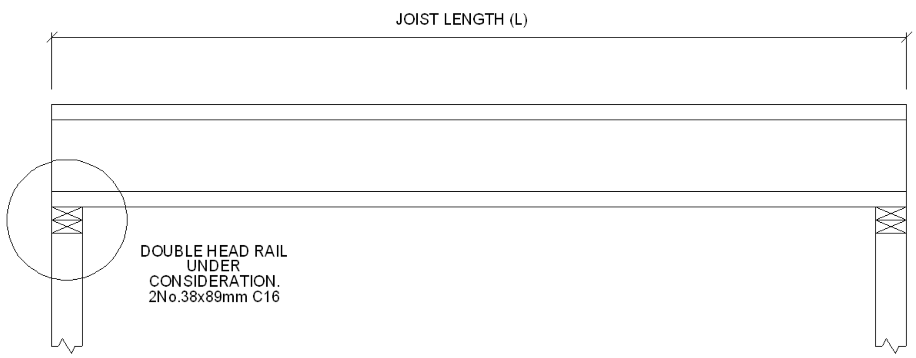

Figure 22: Double rail under single span joist

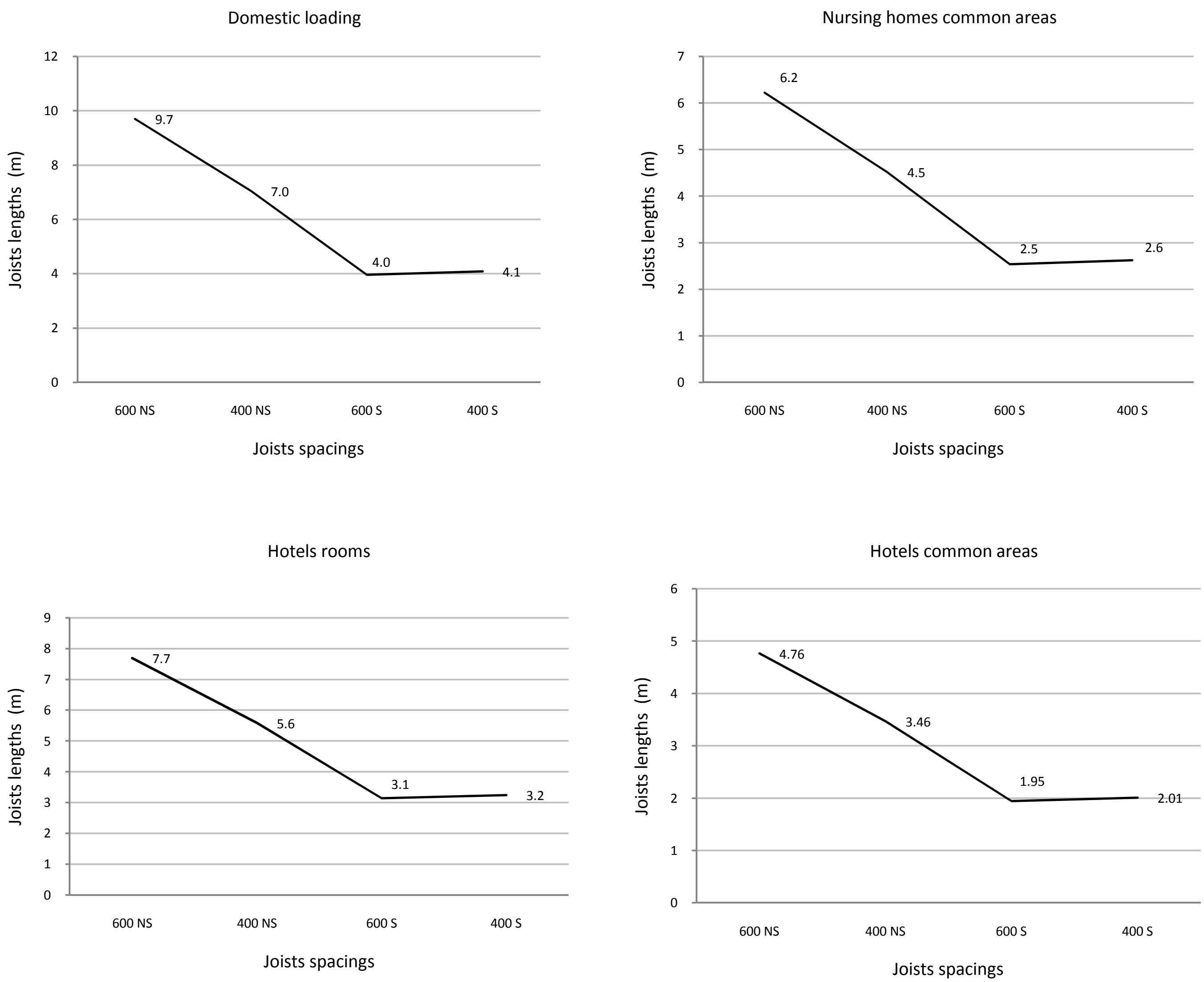

Figure 23: Maximum simply supported joists length: (a) case of *continuous* headbinder and head rail

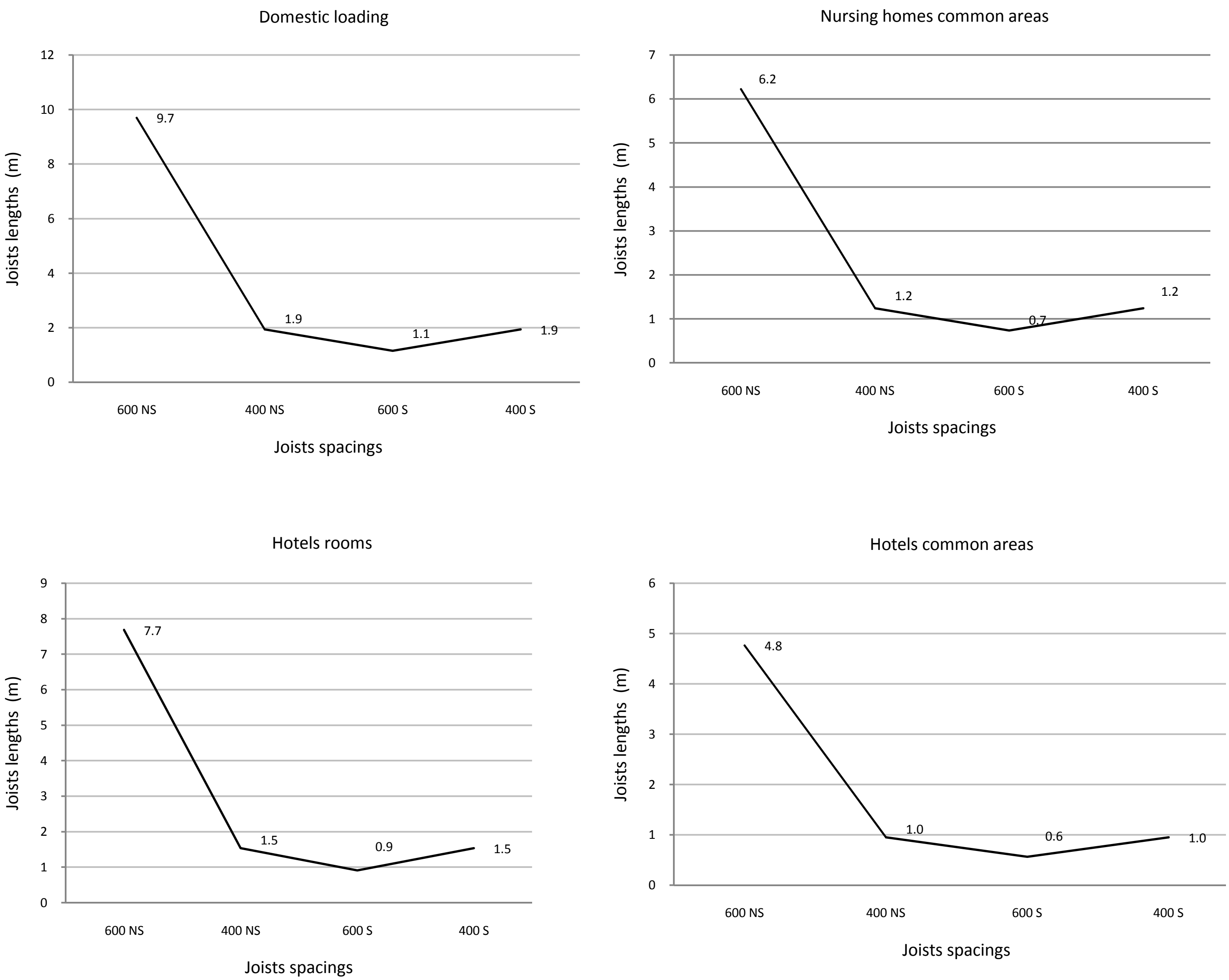

Figure 24: Maximum simply supported joists length: (b) case of *discontinuous* headbinder or head rail

The joist lengths are higher than in Figure 20 and 21 since they yield lower reactions. This example is more appropriate for small size buildings or building with joist span breaks.

4. Conclusions

In this report we have conducted an analytical study of the behaviour of a double rail in different wall lengths when subject to different loading scenarios and floor modules shifts. Results have been drawn for the worst module shifts and in some cases discontinuities within the double rail have been included.

It has been shown that in cases of low loading and a short joist span, the use of continuous headbinders on continuous head rails can be suggested in the case of misaligned joists and studs, as long as adequate structural calculations are performed. *Caution needs to be taken in the case of a discontinuous head rail or head binder - even if the discontinuity occurs on a stud - as this severely impairs the performances of the double rail. It has been found that the most vulnerable position when the break occurs on a stud is at 600mm from both panel edges as this position experiences higher bending stresses in comparison with the other studs locations*.

There will in reality be some unquantifiable redistribution of loads due to element interaction and the plate effect structure of the floor. It can also be reasonably argued that the prescribed live loads stipulated in BS6399-1 are not actually experienced and that the permissible stress values given in BS5268-2 are conservatively low, nevertheless the values for applied loading and permissible stresses are the statutory requirements within the standards and as such should be followed.

Due to the growing scale of timber frame projects and their expansion to applications where sizes and loadings have increased considerably, the results show that the allowance of misaligning joists or trusses with studs by introducing a headbinder <u>*does not*</u> represent a viable universal solution. This is best illustrated if we consider a 12m span 35^{o} truss with concrete interlocking tiles with a mid-span intermediate support and typical snow and storage loadings, the truss reaction on the intermediate bearing is of the order of 10kN. If we also consider a deep “I” joist or metal web joist of 11m with a mid-span intermediate support for flats with a domestic live load and a dead load including floating floors, the joist reaction for joists at 600mm centres would be of the order of 9.5kN. The relative comparison of these loads with those given in table 2 highlights the differential.

This report brings forth the recommendation of always aligning joists and trusses with supporting studs thus adjusting studs centres to match joists and truss centres or alternatively, the structural mark ups should indicate where alignment is essential. The capacity of the head binder and the head rail will depend on the bearing stress resulting from the worst case between the supported joist and the supporting stud.

The limitations of the loads in this report do not take into account any contribution to the head rails made by the effect of the sheathing acting as a box beam between studs. The effect of the sheathing cannot be calculated without test data which is unavailable at the time of writing. However the sheathing is only applicable to external walls and racking walls, it is generally not applied to internal loadbearing walls or party walls.

Bibliography

Strength of Materials for Civil Engineers. T.H.G Megson. Nelson Edition. 1980 ISBN 0-17-761081-6

BS 5268-2: 2002 Structural use of timber. Code of practice for permissible stress design, materials and workmanship.

BS 6399-1: 1996 Loading for buildings. Code of practice for wind dead and imposed loads.

NHBC Standards September 2008, chapter 6.2

Timber frame construction 4th Edition. Revised by Huel Twist and Robin Lancashire. Trada Technology Ltd. 2008 ISBN 978-1-900510-56-1

BS EN 1995-1-1:2004 Eurocode 5. Design of timber structures. General. Common rules and rules for buildings.