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Manuscript title: Stability of light steel walls in compression with plasterboards on one or both sides

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Abstract

In light steel framing, the compression resistance of the walls may be improved by the stabilising effect of plasterboard attached to one or both sides of the wall. This paper presents the results of 28 load tests on walls of 2.4 m height using 75, 100 and 150 mm deep C sections in 1.2 to 1.6 mm thick steel with various types of board. The results were compared to the methodology in BS EN 1993-1-3 and BS 5950-5 taking account of restraint to minor axis buckling. The test results showed that the lateral restraint provided by 12 or 15 mm thick fire resistant or moisture resistant plasterboard fixed to one flange is equivalent to an effective length reduction factor of 0.7 in the minor axis direction for a 100mm x1.6mm C section in a 2.4m high wall. A theory is developed based on distortional buckling of the C sections, which allows the test results to be extended to other wall heights and section sizes. The failure loads were predicted by finite element modelling by considering initial imperfections in the C sections and the torsional restraint due to the boards.

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1. Introduction

The use of light steel framing and modular construction has developed rapidly over the last 15 years and the design of light steel structures has become very efficient. The main market for light steel framing is in residential buildings of 4 to 8 storeys height in which the load-bearing cross-walls are located at 4 to 6m spacing, as illustrated in Figure 1.

The wall panels are pre-fabricated as storey-high units with C sections placed at 300 to 600 mm centres which are fixed into U sections forming a top and bottom 'track'. The walls are generally X-braced to provide overall stability to the building. The C sections are generally 100 or 150mm deep and 1.2 to 2mm thick depending on the number of floors supported. The steel grade is specified as S280 to S450 to BS EN 10346 (BSI, 2009) where the numerical value is the steel yield strength in N/mm^2.

Load-bearing walls may be of two types: Double leaf walls with a nominal cavity in which plasterboards are fixed to one side; or single leaf walls with either directly fixed plasterboard or with acoustically resilient bars on one or both sides to which the plasterboard is fixed. Therefore, the stability of the C sections in compression is dependent on the plasterboard or indirectly though the resilient bars on both sides. Façade walls may have sheathing boards on the outer face and plasterboards on the inside and the sheathing boards may be considered to offer greater restraint, as evidenced by the tests in this paper.

Modular construction is also used for cellular-type buildings of 4 to 17 storeys height, and the nature of this form of construction is that walls are always in double leaf form. The C section sizes are typically 70 to 100mm deep in order to minimise the overall wall width. They are restrained by plasterboard on the inside and often by sheathing board fixed on the outside. However, sheathing boards are not used in all modular systems, as shown in Figure 2.

In parallel with the development of light steel framing technology, BS EN 1993-1-3: Eurocode 3: Design of Steel Structures Part 1.3 – General Rules – Supplementary Rules for Cold Formed Members and Sheeting and its National Annexes came into force in 2010 (BSI, 2005). It is undergoing its first revision. This EN standard has replaced the former BS 5950-5 that continues to be used for smaller projects. Also design to BS 5950-5 leads to higher buckling resistances, which is partly compensated by higher partial factors for loads.

One practical way to improve the practical design of load-bearing walls is to consider how the buckling resistance of C sections can be increased by taking into account the stabilising effect of higher performing plasterboards or sheathing boards on one side of the wall. However, it is important not to over-rely on the contribution of the boards to the load-bearing capacity of the wall, as they may be damaged or replaced at some stage in the future. This is considered later as a lower bound design check.

Fire resistant plasterboard (Type F to BS EN 520 (BSI, 2004)) is required in light steel structures of more than 30 minutes fire resistance, and this type of plasterboard contains fibres to improve its robustness. In recent years, moisture resistant plasterboards (Type H) have been developed for use as external sheathing boards in cladding systems.

There is little practical guidance in design Codes on the stabilising effect of boards attached on one side of a light steel wall, although the ’Direct Strength Method’ was developed (Schafer, 2013)) based on a major study by the American Iron and steel Institute takes into account of the buckling of C sections with torsional restraint from the plasterboard. Most Codes recognise the stabilising effect of boards attached on both sides of the wall, provided the spacing of the fixings is within reasonable limits. However, for buildings subject to high repeated wind loads or dynamic effects, it is not recommended to take account of the stabilising effect of the boards, unless justified by testing.

In practical terms, a single layer of 12.5 mm thick fire resisting plasterboard is sufficient to achieve 30 minutes fire resistance, whereas two layers of 15 mm thick fire resisting plasterboard are required for 90 minutes fire resistance in medium-rise buildings. The minimum case for stability calculations is taken as a single layer of 12.5mm fire resisting plasterboard on one side with fixings at not more than 300mm spacing to the C sections.

2.1 Compression resistance to BS EN 1993-1-3

The compression resistance of cold formed C sections to BS EN 1993-1-3 is based on the rules for hot rolled steel sections given in BS EN 1993-1-1 Clause 6.3.1.2, and is defined by the modified Perry–Robertson buckling formula, as follows:

$$\chi = [\phi + (\phi^2 - \bar{\lambda}^2)^{0.5}]^{-1} \qquad (1)$$

Where:

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \bar{\lambda}^2] \qquad (2)$$

and α is an imperfection parameter = 0.34 for buckling curve '*b*' or 0.21 for buckling curve '*a*'. $\bar{\lambda}$ is the slenderness ratio for the direction of buckling, which is defined separately for major axis (y-y), or minor axis (z-z) buckling.

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For Class 4 sections whose cross-sections are affected by local buckling, the slenderness ratio, $\bar{\lambda}$ is defined as:

$$\bar{\lambda} = \lambda/\lambda_1 \text{ where } \lambda_1 = \pi\sqrt{(E/f_y)(A/A_{eff})} \tag{3}$$

and f_y is the yield strength of steel (the measured steel strength is used for correlation with tests)

E is the elastic modulus of steel = 210 kN/mm^2

λ is the member slenderness, L/i_{yy} or L/i_{zz} respectively,

i_{yy} and i_{zz} are the radii of gyration of the C section in the major and minor axes respectively, in which the minor axis slenderness is reduced when taking account of the stabilising effect of the boards.

A_{eff} is the effective cross-sectional area of the C section taking account of local buckling of the web and flanges at a compression stress, f_y.

A is the gross cross-sectional area that is unreduced by local buckling.

f_y is the yield strength of the steel

The buckling reduction factor, χ , is used to determine the buckling resistance of the C section, as follows;

$$N_{Rd} = \chi A_{eff} f_y \tag{4}$$

In BS EN 1993-1-1, buckling curve 'b' is used for minor and major axis buckling of C sections, but curve 'a' may be used for major axis buckling of C sections placed back to back. The reduction factor, χ obtained from buckling curve 'b' is shown in Figure 3, as a function of $\bar{\lambda}$. Most cold formed C sections in walls have a minor axis slenderness ratio in the range of 0.8 to 1.2, and so χ is approximately 0.5 to 0.7.

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In design to the former BS 5950-5, the equivalent to buckling curve ‘a’ is used in all cases, which can lead to an increase of 5 to 10% in buckling resistance relative to BS EN 1993-1-3 depending on the slenderness of the wall.

2.2 Test programme on light steel walls in compression

The test programme in this paper was developed from a request of the light steel, modular and plasterboard suppliers to investigate the stiffening effects of practical configurations of various types of boards fixed to light steel walls in compression. Four distinct series of tests were carried out, and it was possible to achieve some common parameters in the tests. The tests were performed on 100mm, 150mm and earlier on 75mm deep C sections in which the walls were 2.42m/2.45m long and 1.2m wide. Additional tests were performed on bare steel sections for comparison with the boarded walls.

The 100 mm and 150 mm deep C sections were selected as being produced by most light steel manufacturers and 75 mm deep C sections are the shallowest C sections that are used in practice. A steel thickness of 1.5 or 1.6 mm was used as is typical of most load-bearing wall applications. The tests on the 150 mm deep x 1.2 mm thick C sections provided a lower bound to the lateral restraint offered by the plasterboard on one side, as the web of this C section is relatively flexible.

The series was as follows;

- 100 mm x 50 mm x 1.5 mm thick C sections with 15 mm thick moisture resistant (Type H) plasterboard on one side, and either no boards, or directly fixed plasterboard, or plasterboard fixed via resilient bars on the other side. The C sections were placed at 400 mm spacing so that the wall panel consisted of two internal Cs and two edge C sections. The measured yield strength of the steel was 464 N/mm^2. A total of 8 walls was tested.

- 100 mm x 50 mm x 1.6 mm thick C sections with 12.5 mm thick fire resistant (Type F) plasterboard on one side and with similar configurations of plasterboard on the other side, as in the above series. The measured yield strength of the steel was 428 N/mm^2. A total of 7 walls was tested.
- 150 mm x 50 mm x 1.2 and 1.6 mm thick C sections with 12 mm thick fire resistant (Type F) plasterboard on one side and no plasterboard on the other side. The measured yield strength of the steel was 400 and 385 N/mm^2 respectively for the two thicknesses. A total of 7 walls was tested.
- 75 mm x 50 mm x 1.6 mm thick C sections with two layers of 12.5 mm thick fire resistant (Type F) plasterboard on one side and either no boards or OSB or CPB sheathing boards on the other side. The C sections were placed at 300 mm spacing for the 1200 mm wide wall so that the wall consisted of 3 internal Cs and 2 edge Cs. The measured yield strength of the steel was 400 N/mm^2. A total of 5 walls was tested including two walls with 10mm and 20mm eccentricity at the base.

The dimensions of the C sections are shown in Figure 4. All frames were per-fabricated with U shaped top and bottom tracks to which the C sections were fixed by a 4.8 mm diameter screw to each flange. The first three series of tests were carried out at Oxford Brookes University and the fourth series at the Building Research Establishment (BRE). The BRE tests were aimed at investigating the stiffening effect of sheathing boards used in modular construction.

The tests with boards directly fixed on one or both sides were common to the series on the two types of plasterboard, so that direct comparison could be made between these test series. The modes of failure are: minor axis flexural buckling combined with web bending (known as distortional buckling), major axis buckling, or in the most highly restrained cases, crushing of the cross-section.

Groups of 3 and 4 tests on walls with plasterboard on one side were carried out in order to obtain the characteristic design resistance of the C sections. The characteristic resistance was found to be 85 to 90% of the mean value of the tests in a group. A reduction factor of 0.85 was used in interpreting the results of single tests, and to compare with design resistances calculated to BS EN 1993-1-3.

In all the tests, the plasterboard was fixed using the recommended 3.2mm diameter fixings at 300 mm centres to all C sections. In all cases, the minimum distance of the fixing from the edge of the board to the outer C sections was maintained at 15mm. The boards were fixed a nominal 10 mm short of the ends of the panel so that the boards would not resist compression directly. The failure of the wall with boards on one side was often precipitated by the outer C sections, which are less highly restrained than the inner C sections. In practice, walls are generally continuous and so the tests on discrete wall panels represented a lower bound to the failure load.

Loading was applied concentrically in pure compression though jacks, and it was possible to achieve a relatively slow rate of loading to failure over 15 to 20 minutes to allow some relaxation in loading due to creep in the boards and slip in the fixings.

2.3 Previous research on sheathed light steel walls in compression

Telue and Mahendran (2001) presented results of compression tests on 8 light steel walls with boards on one side and 8 with plasterboards on both sides, which were compared with four tests on bare steel panels. The walls were 2.4m high and the C sections were at 300 or 600mm spacing. Two un-lipped C sections of 75mm depth and 30mm width and 200mm depth and 35mm width in 1.15mm thick steel were tested. Low and high steel grades of S175 and S500 were tested for comparison. In all cases, the 10mm thick plasterboard was fixed at 300mm centres. The test results are presented in Table 1.

The failure load was relatively insensitive to the spacing of the C sections and therefore to the rotational stiffness of the plasterboard. In all of the tests, the increase in the failure load of the C section with plasterboard on one side was a factor of at least 3 times higher than the bare frames with 75mm deep sections, which is due to the low compression resistance of the un-lipped C sections. This factor was much less for the 200mm deep C sections because of the out of plane bending of the web. The steel strength also made a significant difference to the failure loads, particularly for walls with boards on both sides.

Wang, Tian and Lu presented the results of compression tests on walls of 2.44 m height and 1.25 m width comprising three lipped C sections of 90mm depth with 39 or 42mm wide flanges. The steel thickness was 1.5 mm thick in S350 grade. Three different types of sheathing board were fixed on one side of the walls; calcium silicate board (CSB), cement particle board (CPB) and orientated strand board (OSB). No board thicknesses are given in the paper but it is assumed that they are 10mm thick. Loads were applied either uniformly to the top of the wall or by a single point load to the centre of the top of the wall. Fixings to the C sections were placed at 300, 400 or 600mm. The test results are presented in Table 2.

Tests were also carried out on the bare steel frames and it was found that the increase in failure load due to the effect of the boards varied from 31 to 108%. The failure loads for the panels with CPB were marginally higher than those with CSB and OSB, but the differences were small (less than 10%). The failure load increased by about 10% for a fixing spacing of 300mm compared to 600mm. It was also found that the middle C section in the group resisted up to twice the load compared to the outer C sections, which were less restrained by the boards on one side.

Vieira and Schafer (2013) presented the results of compression tests on C sections for walls of various heights and compared the results to earlier work on C section columns by Vieira (2011). The wall tests in compression were carried out using 92 mm deep x 1.73 mm thick C sections in S345 grade steel (50 ksi) which had a measured yield strength of 413 N/mm^2. Plasterboard and OSB sheathing boards were fixed on one or both sides of the walls. The results are presented in Table 3 in terms of the mean failure loads for the 2.44m high walls. It was found that OSB on one side increased the compression resistance of the bare C sections by 55%, and OSB boards on both sides increased the failure load by 92%. Use of plasterboards on both sides of the wall increased the failure load by 71%. The test results are also expressed in terms of characteristic values and are compared with the later theory in this paper.

Vieira and Schafer also quantified the translational and rotational stiffness of the fixings to plasterboard and OSB from the results of small-scale tests. The measured translational stiffness of the fixings were 0.35 and 0.97 kN/mm for plasterboard and

OSB respectively, and the rotational stiffness of the fixings was measured at about 95 kNmm/rad. for both materials.

Guidance on methods of testing of the fixings is given in the American Iron and Steel Institute report 'Sheathing Braced Design of Wall Studs' (2013). In the AISI 'direct strength' method, these stiffness properties may be inserted into a formula for the critical buckling load of the C sections. It was found that the stiffening effect of the boards attached on one side of the C sections was important in determining the buckling load for walls in the range of 2 to 3.5m height. The authors also commented that significant fixity existed due to the C section compressed into the U track.

3.1 Results for bare steel frames in this test series

In this new test series, the control tests on bare steel frames with $100 \times 50 \times 1.5/1.6$ mm C sections failed by torsional flexural buckling combined with flexural buckling, as shown in Figure 5. The failure load per C section was approximately 45 kN. However, it was found by calculation to BS EN 1993-1-3 that the failure load in flexural buckling in the minor axis is close to that in torsional flexural buckling when using an effective length factor of 0.7 for the torsional mode, which results from the torsional restraint provided by the U sections at the ends of the wall.

The theoretical failure load for minor axis flexural buckling was 26 kN to BS EN 1993-1-3 using an effective length equal to the member length and the measured steel strength. This increased to 37 and 40 kN for the two steel thicknesses when using an effective length a factor of 0.85, which gives a better correlation with the test results.

Therefore, it is shown that the effect of compression of the C section into the top and bottom U sections and the restraint by the end fixings leads to a notional effective length factor of 0.85 for bare frames in minor axis flexural buckling. This was observed also by Schafer and Vieira in their tests. However, the failure load is also influenced by the lower imperfections in the tests than assumed in BS EN 1993-1-3.

3.2 Results for wall tests with plasterboard on one side

The 12 light steel frames with plasterboards on one side failed by distortional buckling of the edge C sections followed at high deformation by pull-out of the fixings to the plasterboard, as shown in Figure 6. Tables 4 and 5 present the results of the tests on 100 x 50 x 1.5mm and 1.6mm C sections for the two types of plasterboard. A typical load-displacement graph for a test with boards on one side is shown in Figure 7. The axial displacement at failure was about 7mm and the out of plane displacement of the web of the outer Cs began to increase more rapidly after a load of 80% of the failure load.

Groups of 3 or 4 tests were carried out for the two board thicknesses, and the range of the failure loads was 59 to 65 kN for the 15 mm thick moisture resisting board to 59 to 66 kN for the 12.5 mm thick fire resisting board. This indicates that the results are insensitive to the board thickness or type. If the 7 tests on the 100 x 50 C sections with boards on one side are considered to be one group, this leads to a characteristic failure load of 89% of the mean - see section 4.5. The minimum failure load was 93% of the mean of the tests in the group. Therefore, for single tests, it is considered reasonable to take the notional characteristic failure load as 0.85 x Failure load, when comparing with the proposed design method in section 4.1. This is presented in the Tables for comparative purposes.

When the plasterboard was fixed on one side via resilient bars without boards on the other side, the failure load was 59 kN. This load was at the lower end of the range of the tests of boards directly fixed on one side, which shows that resilient bars on one side offer less restraint than directly fixed boards. The mode of failure in this test is shown in Figure 8. The test with plasterboard on one side and only resilient bars on the other side failed at 65 kN, which was similar to the mean of the tests with plasterboard on one side.

3.3 Results for tests on 100 x 50 C sections with boards on both sides

The results for tests with various configurations of plasterboard and resilient bars on both sides of the C sections are also presented on Tables 4 and 5. The test with moisture- or fire-resisting plasterboards directly fixed on both sides failed at loads of 84 and 86 kN respectively, which shows that there is little difference in performance of these two types of boards. The mode of failure was by major axis buckling, as shown in Figure 9, but for design, it is proposed that the flexural buckling resistance is calculated using the larger of the major axis slenderness or the minor axis slenderness with an effective length factor of 0.5. The design resistance is 67kN using this effective length factor, which is 7% less than the notional characteristic failure load of 72 kN.

For moisture-resistant plasterboard fixed on one side and plasterboard fixed via resilient bars on the other, the failure load was 100 kN per C section. For the test with fire resisting plasterboards fixed via resilient bars on both sides, the failure load was 85 kN. In both cases, the failure mode was by crushing of the C section rather than buckling in the major axis. Although not conclusive, the tests show that plasterboards attached via resilient bars on both sides of the wall can increase the stiffness of the wall relative to directly fixed boards.

3.4 Results for wall tests with 150 x 50 C sections

A further test series was performed on walls with 150 x 50 x 1.2 and 1.6mm C sections and with 12.5mm fire-resisting plasterboard fixed on one side. Comparative tests were also made on the C sections without boards. These tests showed the effect of the lower restraint to buckling of the deeper C sections, as a result of the flexibility of the web. The test results are presented in Table 6. The tests on bare C sections failed again by a combination of torsional flexural buckling and flexural buckling and the same observations on the effective length factor hold as for the test on 100mm deep C sections.

3.5 Results for tests with sheathing boards on one side

The tests on 75 x 45 x 1.6mm thick C sections had sheathing boards fixed on one side and in most tests, two layers of 15 mm fire resistant plasterboard were fixed on the other side. Figure 10 shows a load test on a wall with boards on both sides, which failed by major axis buckling. Table 7 presents the results for this series of tests.

For the test with OSB board fixed on one side, the failure load was 64 kN and so 0.85 x failure load is 54 kN. This equates to a compression resistance in flexural buckling with an effective length factor of 0.5 in the minor axis, which shows that the OSB board offers a higher level of torsional restraint for this shallow section.

For plasterboard on one side and OSB or CPB sheathing boards on the other side, it was found that there was little difference in the restraint offered by OSB and CPB. In both cases, the failure was by major axis buckling at approximately 97 kN. A load of

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0.85 x failure load agrees with major axis buckling resistance with an effective length factor of 0.7, which shows that composite action of boards on both sides is responsible for a considerable increase in the failure load of the C section. This composite action is not considered further here but is a subject of future research.

4.1 Theory of distortional buckling of C section with board restraint on one flange

Consider the distortional buckling of a symmetric C section in which one flange is fully restrained by attachment to boards and the other flange is unrestrained so that its web can bend out of its plane, as shown in Figure 11. The boards to which the C sections are attached are also flexible in bending and so contribute to out of plane bending of the C section.

The in –plane bending energy in the unrestrained flange and the out of plane bending energy of the web may be equated to the potential energy of the applied axial load. Torsion is neglected for thin steel sections. The critical buckling load, $P_{cr,zz}$, of a C section in distortional buckling that is rotationally restrained on one flange is taken as twice the compression resistance of the unrestrained flange. It is given as follows:

$$\frac{P_{cr,zz}}{2} = \frac{\pi^2 E_s \left(I_{zz} / 2\right)}{{L_E}^2} + \frac{k_\theta}{\pi^2}\left(\frac{L_E}{h}\right)^2 \qquad (5)$$

where I_{zz} is the second moment of area of the C section in the minor axis

L_E is the member effective length used in distortional buckling

k_θ is the total rotational stiffness provided to the C section, as follows:

$$k_\theta = \left(\frac{1}{k_{w,\theta}} + \frac{1}{k_b} + \frac{1}{k_f}\right)^{-1} \quad (6)$$

$k_{w,\theta}$ is the rotational stiffness due to out of plane bending of the web

The out of plane bending of the web may include a contribution from bending of the top flange. If it assumed that a point of fixity exists in the flange at a distance of 0.5b from the web, the rotational stiffness of the C section due to out of plane bending of the web is given by:

$$k_{w,\theta}=\frac{E_s t^3}{12\left(h/3+b/2\right)}=\frac{E_s t^3}{4\left(h+1.5b\right)} \quad \text{per unit length} \qquad (7)$$

where t is the steel thickness

b is the flange width

h is the section depth

E_s is the elastic modulus of steel

k_b is the flexural stiffness of the board per unit length of the beam. This is taken as the worst case of a board on one side of the C section, as was the case in the wall tests, and is given by:

$$k_b=\frac{2E_b d^3/12}{s}=\frac{E_b d^3}{6s} \quad \text{per unit length} \qquad (8)$$

where d is the board thickness

s is the transverse spacing between the C sections

E_b is the elastic modulus of the board, which may be taken as 2 kN/mm^2 for Type F and Type H plasterboards and 5 kN/mm^2 for OSB.

k_f is the rotational stiffness due to the tensile action of a fixing to the flange that is given by:

$$k_f=\frac{k_{f,ten}}{s_f}\left(\frac{b}{2}\right)^2 \quad \text{per unit length} \qquad (9)$$

where b is the flange width

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s_f is the longitudinal spacing of the fixings

$k_{f,ten}$ is the tensile stiffness of a fixing into the board.

The tensile stiffness of a fixing depends on the board embedment and diameter of a screw. For typical data, $k_{f,ten}$ is 1 to 2 kN/mm in the elastic range, and the pull-out resistance is typically 0.5 kN for a 3.2mm diameter fixing into 12.5mm thick Type F plasterboard. It is observed from the tests that this stiffness is relatively high until after buckling has occurred so this component is ignored in comparison to k_w and k_b. However, it is necessary to check the pull-out resistance of the screws-see section 4.4. Where two boards are fixed on one side of the C section, the fixing density is increased by 50% as the fixings to the inner boards are generally at twice the spacing of the outer boards.

From equation (5) and (6), the minor axis slenderness factor due to distortional buckling is given by:

$$\frac{\lambda_{zz,red}}{\lambda_{zz,eff}} = \left(1 + 0.02\left(\frac{k_{w,\theta}}{1 + k_{w,\theta}/k_b}\right)\left(\frac{h^2}{EI_{zz}}\right)\left(\frac{L_E}{h}\right)^4\right)^{-0.5} \qquad (10)$$

Where $\lambda_{zz,eff}$ is the minor axis slenderness of the C section based on its effective length allowing for end fixity.

From equations (7) and (8), the ratio of the out of plane bending stiffness of the web and the board flexural stiffness becomes:

$$\text{where } \frac{k_{w,\theta}}{k_b} = 1.5\left(\frac{E}{E_b}\right)\left(\frac{t}{d}\right)^3\left(\frac{s}{h + 1.5b}\right) \qquad (11)$$

From this point in the derivation, further simplifications are required to obtain a useable design formula that can be correlated with the results of the wall tests. These simplifications are justified by statistical analysis of test results and by limited finite element modelling. However further work is required to identify other parametric variations outside the limits of the test data.

For a lipped C section with h=2b and edge lips, c=0.2b, the second moment of area in the minor axis direction is given by I_{zz} = 0.62b^{3}t. For a section with h=3b, I_{zz} is given by: I_{zz} =0.7b^{3}t. Therefore, a more general formula for the minor axis inertia of a C section is I_{zz} =0.9 b^3(h/b)$^{0.5}$t. Inserting this I_{zz} value into equation (10), it follows that the stiffening effect of the boards attached on one side of the C section leads to a slenderness reduction factor given by:

$$\frac{\lambda_{zz,red}}{\lambda_{zz,eff}} = \left(1+5.5\text{x}10^{-3}\left(\frac{1}{1+k_{w,\theta}/k_b}\right)\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^2\left[\frac{\left(\frac{b}{h}\right)^{0.5}}{1+1.5b/h}\right]\left(\frac{L_E}{h}\right)^4\right)^{-0.5} \qquad (12)$$

For a range of C sections with b/h in the range of 0.3 to 0.6, the parameter (b/h)$^{0.5}$/(1+1.5 b/h) tends to a value of 0.4 and so the slenderness reduction factor is further simplified to:

$$\frac{\lambda_{zz,red}}{\lambda_{zz,eff}} = \left(1+2.2\text{ x}10^{-3}\left(\frac{1}{1+k_{w,\theta}/k_b}\right)\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^2\left(\frac{L_E}{h}\right)^4\right)^{-0.5} \qquad (13)$$

Partial end fixity of the attachment of the C sections to a U top and bottom track is taken into account by including an effective length factor of 0.85, as observed for the bare frame tests. If equation (13) is expressed in terms of the slenderness of the C section based on its full length, the coefficient of 2.2x10^{-3} becomes 2.2x10^{-3}x0.85^4 =1.2x10^{-3}, and so equation (13) becomes:

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = 0.85\text{x}\left(1+1.2\text{x}10^{-3}\left(\frac{1}{1+k_{w,\theta}/k_b}\right)\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^2\left(\frac{L}{h}\right)^4\right)^{-0.5} \qquad (14)$$

Where λ_{zz} is the minor axis slenderness of the C section based on its full length, L.

For 12.5mm thick plasterboards connected to 100x50x1.5mm thick C sections at 600mm spacing, $(1+k_{w,\theta}/k_b)^{-1} = 0.5$. Therefore, as a good approximation for design to BS EN 1993-1-3, the effective slenderness of the C section with Type F or H plasterboards or other more rigid boards fixed on one flange becomes:

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = 0.85\left(1+0.6x10^{-3}\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^{2}\left(\frac{L}{h}\right)^{4}\right)^{-0.5} \geq 0.6 \qquad (15)$$

This formula applies for steel thickness up to 1.6mm, which is within the tested range.

4.2 Minimum effective length factor

The minimum effective length factor in equation (15) is taken as 0.6 to avoid pull-out failure of the fixings and to provide stability of the bare C sections if the boards are removed or badly damaged. In this case, it is required that the bare steel frame is stable under un-factored loads. If the overall factor of safety for the wall is taken as 1.45 when using the partial factors in Eurocodes, then the compression resistance for an effective length factor of 0.6 must not exceed 1.45 times the compression resistance for an effective length factor of 0.85 for the bare frame. This is shown to be the case if the maximum slenderness ratio of the bare steel wall is $\bar{\lambda} \leq 1.5$, which applies to walls using 100mmx 50mm C sections that do not exceed 3m height.

The limits of use of equations (14) and (15) are also determined by the tests presented in this paper. The effect of increasing the steel thickness is discussed in section 4.5.

4.3 Application to double boards on one side

For two 12.5 or 15mm thick boards attached on one flange, the combined bending stiffness of the boards in is the range of x 2 to x 8 depending on the effective composite action of the two boards and the greater number of fixings that are used. For the purposes of this derivation, the bending stiffness of double boards of the same specification is taken as 4 times that of single boards, which allows for some interaction between the boards. In this case, the boards are relatively stiff in comparison to the web, and so it is reasonable to take $k_w/k_b = 0.25$ in equation (14). The coefficient of 0.6 x 10^{-3} in equation (15) may be increased to 1.0 x 10^{-3} for two boards attached to one flange. This is shown to be conservative relative to the test result for this case, but more work is needed on the additional stiffening effect of double boards.

4.4 Influence of pull-out of the board fixings

In the tests with boards on one side, pull-out of the fixings to the plasterboard occurred after distortional buckling had resulted in high deformation of the web. In the test panels, there were 9 fixings to each C section at 300 mm longitudinal spacing and the measured distance of the outer line of fixings from the edge of the plasterboard was in the range of 15 to 19 mm. In practice, the outer C sections in a wall would be restrained by attachments to other walls and so failure of these C sections would occur at a higher failure load than in the tests. Nevertheless, the test results give a lower bound to the load-bearing capacity. The following theory may be used to predict when pull-out failure may occur in the distortional buckling mode.

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For the unrestrained flange of a C section with a deformation of the web, ‘v’, at the mid-length of the C section that is expressed as a circular arc, the force per unit length developed in the out of plane direction of the web for equilibrium is given by:

$$F_h = \frac{8v}{L^2}\left(\frac{P}{2}\right) \qquad (16)$$

where the compression force in the unrestrained flange is 0.5P.

This horizontal force is partly resisted by in-plane bending of the bottom flange and partly by distortion of the web in out of plane bending, as shown in Figure 5. The proportion of this force resisted by distortion of the web is given by ($1-P_{E,zz}/P_{cr,zz}$), where $P_{E,zz}$ is the Euler resistance of the C section in the minor axis over length, L, and $P_{cr,zz}$ is the critical buckling load of the C section when stabilised by the plasterboard on one flange given by equation (5).

The horizontal force acting on the web that causes out of plane bending is given by:

$$F_{h,web} = \frac{4v}{L^2} P\left(1-\frac{P_{E,zz}}{P_{cr,zz}}\right) = \frac{4v}{L^2} P\left(1-\left(\frac{\lambda_{zz,red}}{\lambda_{zz}}\right)^2\right) \qquad (17)$$

Where $\lambda_{zz,red}$ is the reduced slenderness defined in equations (14) or (15).

The out of plane deformation, v, may be considered to increase classically from an initial imperfection, δ_o as a function of $P/P_{cr,zz}$, as follows:

$$v = \frac{\delta_o}{1-P/P_{cr,zz}} \qquad (18)$$

where δ_o is the initial imperfection for a C section using buckling curve ‘b’.

The initial imperfection is given approximately by δ_o = L/450 for equivalence with buckling curve ‘b’ representing minor axis buckling of a C section.

The ratio of the applied load to the critical buckling load is obtained from:

$$P/P_{cr,zz} = \chi\bar{\lambda}^{2} \qquad (19)$$

Where:

$\bar{\lambda} = \lambda_{zz,red}/\lambda_{1.}$

λ_1 is defined in equation (3), in which A_{eff}/A is taken conservatively as 0.75 for a 100x50x1.6mm C section.

χ is the reduction factor due to buckling at a slenderness ratio, $\bar{\lambda}$, obtained from Figure 2.

The fixing of the plasterboard is taken as acting at the mid-width of the flange in the general area of the board and so a horizontal force acting at the base of the C section leads to a pull-out force in the fixing, F_{ten}, of:

$$F_{ten} = F_{h,web}\left(\frac{h}{0.5b}\right)s_f \leq F_{Rd} \qquad (20)$$

where s_f is the longitudinal spacing of the fixings

F_{Rd} is the pull-out resistance of a fixing, which for a 3.2mm diameter fixing to Type F or H plasterboard is approximately 0.5 kN.

From equations (17) to (20), it follows that the maximum compression force, P, that may be applied to a C section when limited by the pull-out resistance of the fixings is obtained from:

$$P \leq 56\,F_{Rd}\left(\frac{L}{s_f}\right)\left(\frac{b}{h}\right)\left(\frac{1-\chi\left(\frac{\lambda_{zz,red}}{\lambda_1}\right)^2}{1-\left(\frac{\lambda_{zz,red}}{\lambda_{zz}}\right)^2}\right) \qquad (21)$$

From the test data: 100 x 50 x 1.6mm C section (f_y = 428 N/mm^2) with L = 2420mm and s_f = 300mm. The failure load by pull-out of the fixings at F_{Rd} = 0.5 kN is obtained as follows for restraint to one flange: Data: i_{zz} = 18.2mm, λ_{zz} = 2420/18.2 = 133, A_{eff}/A = 0.75, λ_1 = 80, and ε = 0.75. From equation (15), $\lambda_{zz,red}$ = 0.72 λ_{zz} = 0.72x 133 = 95,

$\bar{\lambda} = \lambda_{zz,red} / \lambda_1 = 95/80 = 1.19$, and $\chi \approx 0.48$ for buckling curve 'b' . From equation (21), the failure load when limited by pull-out of the fixings is:

$$P = 56x0.5\left(\frac{2420}{300}\right)\left(\frac{50}{100}\right)\left(\frac{1-0.48x1.19^2}{1-0.72^2}\right) \quad = 113 \text{ x } 0.66 = 74 \text{ kN}$$

This exceeds the average test failure load of 61 kN, which suggests that failure did not occur in this mode. The characteristic value of the failure load was 53 kN, which corresponds to a pull-out force of 0.3 kN per fixing when using equation (21).

In comparison, a horizontal force of 2.5% of the compression force in the unrestrained flange is equal to: 0.025 x 0.5 x 53/2.42 = 0.27 kN/m when expressed per unit length, which leads to a pull-out force of 0.33 kN per fixing. It is proposed to use this simple 2.5% rule as the basis of determining the tension forces in the fixings to the boards.

4.5 Statistical correlation of the tests with the simplified theory for boards attached on one side

The design model used to include allowance for plasterboard attached to C sections is based on the theoretical model presented in BS EN 1993-1-3. For use in design, effective length factors are proposed for different plasterboard combinations.

The statistical analysis for tests with plasterboard has been conducted in accordance with BS EN 1993-1-3, A.6.3.2 “Characteristic values for families of tests”. The family of tests approach is applicable to this group of tests because there are more than four tests carried out on a series of similar specimens in which one or more of the parameters are varied.

The tests on 100 mm C-sections and the tests on 150 mm C-sections were treated as two separate groups. The 7 tests in the 100 mm C-sections with plasterboard attached had a variance of 5.6% from the mean and the ratio of the characteristic failure load to the theory is 0.89 for this test configuration. There are 6 tests in the family of 150 mm C-sections with plasterboard attached with a variance of 6.2% and the ratio of the characteristic failure load to the theory is 0.88 for this test configuration.

The statistical analysis of the tests in comparison with the theory is presented in Table 8. For the 100x50x1.5/1.6mm Cs with boards on one side, the mean ratio of Test Failure Load/Theory is 1.19. The theory gives a design resistance that is 8% higher than the characteristic failure load when combining the two groups of tests. For the 150x50x1.2 and 1.5mm Cs, the mean ratio of Test Failure Load/Theory is 1.42. The theory gives a design resistance that is 29% higher than the characteristic failure load. This shows that the theory of distortional buckling is more conservative for the deeper C sections.

For the 100x50x1.5 and 1.6mm Cs with boards on both sides, the mean ratio of Test Failure Load/Theory is 1.27, when using a minimum effective length factor of 0.5 in minor axis buckling. This ratio is higher than for tests with boards on one side and similar to the tests on bare steel sections. This shows that an effective length factor of 0.5 in minor axis buckling is conservative for these tests that failed in their major axis.

For the 75x45x 1.6mm C sections, the effective slenderness with boards on both sides is obtained by including the composite stiffness of the C sections stiffened by the OSB and CPB using an elastic modulus of 5 kN/mm^2. This has the effect of reducing the effective slenderness in major axis buckling by a factor of 0.7, although this effect is not verified for a wider range of C sections.

The application of the design formula in equation (14) to walls of various heights with boards on one side using five typical C sections is presented in Table 9 including the lower bound effective length reduction factor of 0.6 for minor axis buckling.

4.6 Application to thicker steel

The design formula for the effective slenderness in equations (14) or (15) has been calibrated against tested steel thickness of 1.2 to 1.6mm. It may be applied to thicker steel than that tested, but the limiting factor is likely to be pull-out of the fixings based on 2.5% of the compression force in the unrestrained flange of the C section. Therefore it is proposed that the minimum slenderness in equations (14), (15) and Table 9 is modified according to:

$$\lambda_{zz,red} = 0.6\ (t/1.6)\ \lambda_{zz} \text{ but } \leq 0.85\ \lambda_{zz} \qquad (22)$$

where t is the steel thickness in mm (for $t > 1.6$mm).

This implies that the maximum steel thickness is $t \leq 2.2$mm to be able to consider the stabilising effect of plasterboard on one flange. For $t > 2.2$mm, the effective length reduction factor is limited to 0.85 for single boards on one side.

For double boards fixed on one side, the fixing density is increased by 50% and so the pull-out resistance is higher. Using equation (21), it may be shown that the steel thickness may be increased when limited by pull-out of the fixings. Therefore it is proposed that the minimum slenderness in equations (14), (15) and Table 9 for double boards on one side is modified according to:

$$\lambda_{zz,red} = 0.6\ (t/2.0)\ \lambda_{zz} \text{ but} \leq 0.85\ \lambda_{zz} \qquad (23)$$

This implies that the maximum steel thickness is t ≤ 2.8mm to be able to consider the stabilising effect of two layers of plasterboard on one flange of the C section.

5. COMPARISON WITH FINITE ELEMENT MODELS

As a check on the accuracy of the developed theory, comparison of tests using 100x50 C sections was made with a nonlinear elasto-plastic finite element (FE) model using ABAQUS (version 6.14-2). The finite element model was based on the measured centre line dimensions of the cross-sections. The cases considered was 100x50x1.6mm C sections (f_y = 428 N/mm^2) with 12.5mm Type F plasterboard and 100x50x1.5mm C sections (f_y = 464 N/mm^2) with 15mm thick Type H plasterboard. The FE model was used to predict the ultimate load, axial shortening and failure modes of the C sections fixed to plasterboard on one side. Details of the finite element idealisation are shown in Figure 12.

Screws between the flange of the C sections and the plasterboard were idealised by connector elements. The translational stiffness of the connector was taken as 0.35 kN/mm and the rotational stiffness as 0.95 kNm/rad based on previous tests (Vieira and Schafer, 2013).

The load was applied to the reference point which is the centre of one end plate, as shown in Figure 12. The end plates were modelled so that they are effectively rigid when applying the load to all the C sections in displacement control. The C sections were loaded using the RIKS algorithm available in ABAQUS library, through the reference point of the bottom base plate. The RIKS method can predict unstable and nonlinear collapse of a structure, such as post buckling behaviour.

"Surface to surface" contact was used for modeling the interaction between the bottom flange of each C section and the plasterboard. The bottom flange of each C section was modeled as a slave surface, while the top of the plasterboard was considered as a master surface and there was no penetration between the two contact surfaces.

The classical metal plasticity model available in ABAQUS library was used, which implements the von Mises yield surface to define isotropic yielding, associated plastic flow theory and isotropic hardening behaviour. The mean mechanical properties obtained from the tensile tests were used for engineering stress-strain curve. The plasterboard elastic modulus was taken as 2 kN/mm^2 for short term loading.

The C sections and plasterboard were modelled using the linear 4-noded quadrilateral thick shell element S4R, which has six degrees of freedom per node (three translation and three rotational degrees of freedom at each node).The element accounts for finite membrane strains and arbitrarily large rotations. The upper and lower end plates were modelled using rigid quadrilateral shell elements (R3D4). An element size of 5mm square was found to be appropriate for C sections, based on the results of a convergence study. For the plasterboard and end plates, a mesh size of 20mm by 20mm was used. The number of elements along the length of the C-sections was chosen, such that the aspect ratio of the elements was close to one. The MPC beam connector element was used to model the screws.

The test and finite element results were compared for the bare steel frame and for the frame with plasterboard on one side that was attached via ‘spring’ connectors, using a range of stiffness. For the bare frame, the FE model gave a failure load of

44.5 kN, which is in good agreement with the test result of 45.0 kN. Figure 13 shows the output stresses and deflected shape obtained from the finite element analysis of a C section with boards on one side, which shows the effect of end fixity and distortional buckling of the outer C sections. The failure load from the FE model was 59.6 kN per C section when using the Vieira and Schafer connector stiffness in comparison to the average test failure load of 62.8 kN.

The effect of the stiffness of the ‘spring’ connectors between the C section and the plasterboard was also investigated. The failure loads from the FE analysis using the connector stiffness taken from Vieira and Schafer (2013) were compared with a fully rigid connector (upper bound) and also a connector with half the stiffness value (lower bound). The influence of the three connector stiffness in the FE models is compared to the test result in Figure 14 in terms of the lateral displacement of the web (described as in-plane displacement of the wall). It is seen that the C sections with rigid connectors fail at 10% higher load than that of the realistic connector stiffness, but the FE result for half of the realistic connector stiffness failed at only 3% lower load.

A similar study was carried out of the influence of the plasterboard thickness and the results are presented in Figure 15 for 12.5 and 15mm thick plasterboards. The difference in the failure load of using the thicker board with the same fixing stiffness was negligible.

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It is shown that the FE models can predict the behaviour of the light steel walls with plasterboard on one side and so can be used for parametric studies of the wall height, C section size and board stiffness. This will be the subject of a later paper, which is outside the scope of this paper.

6.1 Design to BS 5950-5

The former BS 5950-5 is still used in practice and therefore it is necessary to consider how the method may be adapted to this alternative design approach, which differs in three important respects:

- The higher partial factors for loads in BS 5950-5 lead to approximately 6% higher loads than to BS EN 1993-1-3.
- The effective section properties taking account of local buckling are 5 to 10% higher than to BS EN1993-1-3, depending on the section size.
- The buckling curve to BS 5950-5 is equivalent to buckling curve ‘a’ to BS EN 1993-1-3, which leads to a higher compression strength than to buckling curve ’b’ to BS EN 1993-1-3.

Therefore, design to BS 5950-5 leads to a higher buckling resistance and so it is proposed that in the previous method, partial end fixity of the attachment of the C sections to a U top and bottom track is taken into account by including an effective length factor of 0.95 rather than 0.85. For design to BS 5950-5, the effective slenderness of the C section due to plasterboards fixed on one flange becomes:

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = 0.95\left(1+0.6x10^{-3}\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^{2}\left(\frac{L}{h}\right)^{4}\right)^{-0.5} \geq 0.65 \qquad (22)$$

The statistical analysis of the tests is presented in Table 10 based on the use of equation (22) with section properties and buckling resistance obtained to BS 5950-5. For the 100x50x1.5/1.6mm Cs as bare frames, the correlation is similar to using an effective length factor of 0.85 to BS EN 1993-1-3. . For the 100x50x1.5/1.6mm Cs with boards on one side, the mean ratio of Test Failure Load/Theory is 1.18 and for the 150x50x1.2/1.5mm Cs, the mean ratio of Test Failure Load/Theory is 1.41, both of which are similar to BS EN 1993-1-3.

Based on these results for design to BS5950-5, the effective length factors for boards on one side may be taken as 1.1 times those in Table 8 with a minimum effective length factor of 0.65(t/1.6), where t is the steel thickness. For directly fixed boards on both sides, it is found that the effective length factor should be taken as 0.6, in which case, the mean ratio of Test Failure Load/Theory is 1.17.

6.2 Application to pairs of C sections

The same approach may be adapted to the case of pairs of C sections restrained by boards on one flange. In this case, the stability of the pair of C sections is improved by their combined stiffness in transverse bending when their webs are fixed regularly along their length.

No method exists in BS EN 1993-1-3 to take account of the reduced slenderness of back to back C sections. However, BS5950-5 clause 6.25 presents a formula for the effective slenderness of compound sections, that is given by:

$$\frac{L_E}{i_{zz,comp}} = \left[\left(\frac{L_E}{i_{zz,2c}} \right)^2 + \left(\frac{s_x}{i_{zz}} \right)^2 \right]^{0.5} \qquad (25)$$

where L_E is the effective length of the C section.

$i_{zz,2c}$ is the radius of gyration of a pair of C sections acting as one section. in the minor axis

$i_{zz,comp}$ is the radius of gyration of the compound pair of C sections in the minor axis.

s_x is the longitudinal spacing of the screws between the Cs, typically at 600mm.

It is generally found that $i_{zz,comp} \approx 1.2\ i_{zz}$, which leads to a 17% reduction in slenderness independent of the stabilising effect of the boards. The relative bending stiffness of the two webs and the board may be taken into account in equation (11). Including the combined action of the two Cs and partial end fixity, the effective length factor is 0.85/1.2 = 0.7 when the slenderness is based on a single C section. It may be shown that for pairs of C sections, equation (14) becomes:

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = 0.7\text{x}\left(1 + 2\text{ x}10^{-3}\left(\frac{1}{1 + k_{w,\theta}/k_b}\right)\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^2\left(\frac{L}{h}\right)^4\right)^{-0.5} \qquad (26)$$

As a conservative approximation, $k_{w,\theta}/k_b = 4$ in equation (26) when considering the stiffness of two webs, and so for design to BS EN 1993-1-3, the slenderness reduction factor is given by:

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = 0.7\text{x}\left(1 + 0.4\text{ x}10^{-3}\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^2\left(\frac{L}{h}\right)^4\right)^{-0.5} \geq 0.5\ (t/1.6) \qquad (27)$$

This shows that the effect of the boards is proportionately less than in single C sections because the majority of the slenderness reduction comes from the combined stiffness of the pair of C sections.

The minimum slenderness reduction for boards on one side may be taken as 0.5 (t/1.6) because of the greater contribution of the transverse bending stiffness of the web, but this should be the subject of further research. The maximum steel thickness is again 2.2mm for use of this method.

The slenderness reduction factors for pairs of C sections are presented in Table 11, based on the slenderness of a single C section. However, if the slenderness is based on the radius of gyration of a pair of Cs, then the coefficient of 0.7 reverts to 0.85 in equation (26). For design to BS 5950-5, the coefficient of 0.7 is replaced by 0.95/1.2 ≈ 0.8, and the minimum slenderness reduction factor is 0.65 (t/1.6).

CONCLUSIONS

The tests on 2.4m high light steel walls with C sections in compression and with fire- and moisture-resisting plasterboards (Type F and H to EN 520) of 12.5 or 15mm thickness attached by 3.2mm diameter screws at 300mm spacing to one or both flanges of the C sections have shown that:

- For these types of plasterboard fixed on one flange of 100x1.5/1.6 mm C sections, the average failure load was 26 to 46% higher than the failure load of the bare steel sections. The characteristic failure load was 53 kN for the tests with 15mm thick boards and 51 kN for tests with 12.5mm thick boards.
- For these types of plasterboard fixed attached on one flange of 150x1.2/1.5 mm C sections, the failure load was 27 to 35 % higher than that of the bare steel sections.

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- For design to BS EN 1993-1-3, a design formula for the effective minor axis slenderness of C sections taking account of boards fixed on one flange is presented in equation (28) using the geometric parameters defined earlier. The method is shown to be conservative relative to the characteristic failure loads obtained from the tests.

$$\frac{\lambda_{zz,red}}{\lambda_{zz}} = A\text{x}\left(1+B\,\text{x}10^{-3}\left(\frac{h}{b}\right)\left(\frac{t}{b}\right)^{2}\left(\frac{L}{h}\right)^{4}\right)^{-0.5} \leq C \qquad (28)$$

The coefficients in this equation depend on the board and C section configuration and are presented below. For a single layer of plasterboard, the maximum steel thickness used in equation (28) is 2.2mm, and the maximum section height is 150mm based on the test data in this paper. Also it is proposed that the maximum wall length is taken as 3m in using this equation.

Case	Board configuration	A	B	C
Single C	Single boards on one side	0.85	0.6	0.6≤C≤0.85 and ≤ 0.6 (t/1.6)
	Double boards on one side		1.0	0.6≤C≤0.85 and ≤ 0.6 (t/2.0)
Pairs of C	Single boards on one side	0.7	0.4	0.5 ≤C ≤ 0.7 and 0.5 (t/1.6)
	Double boards on one side		0.8	

Design to BS EN 1993-1-3 for the slenderness of a single C section in its minor axis

t is the steel thickness

- For design to BS 5950-5, the coefficient A is increased to 0.95 for single boards and 0.8 for double boards, as a result of the different buckling curve compared to BS EN 1993-1-3. The coefficient, C, is also increased from 0.6 to 0.65 and from 0.5 to 0.55.

- For walls with directly fixed plasterboards on both sides, the effective length reduction factor on minor axis buckling may be taken as 0.5 for design to BS EN 1993-1-3 and 0.6 to BS 5950-5, but the slenderness should not be less than for major axis buckling.
- The sheathing boards that were tested provided more restraint than plasterboard, but more work is required to determine the stabilising effect of other types of board.
- This guidance should not be applied to walls subject to repeated or dynamic loads. However, the practical use of this guidance is in residential and similar buildings subject to monotonic loads.

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Notation list

f_y is the yield strength of steel
E is the elastic modulus of steel
i_{yy} and i_{zz} radii of gyration of the C section in the major and minor axes respectively
A_{eff} is the effective cross-sectional area
A is the gross cross-sectional area
L_E is the member effective length
k_θ is the total rotational stiffness
t is the steel thickness
b is the flange width
h is the section depth
E_s is the elastic modulus of steel
d is the board thickness
s is the transverse spacing between the C sections
E_b is the elastic modulus of the board
s_f is the longitudinal spacing of the fixings
$k_{f,ten}$ is the tensile stiffness of a fixing into the board.

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References

American Iron and Steel Institute, AISI S100-12 – North American Specification for the Design of Cold-Formed Steel Structural Members, 2012 Edition

British Standards Institution, BS 5950-5: Structural use of steelwork in building. Code of practice for design of cold formed thin gauge sections, 1998

British Standards Institution, BS EN 520: Gypsum plasterboards. Definitions, requirements and test methods, 2004

British Standards Institution, BS EN 1993-1-1: Design of steel structures – General rules and rules for buildings, Part 1.1 General rules for the structural design of buildings and civil engineering works in steel, 2005

British Standards Institution, BS EN 1993-1-3: Eurocode 3, Part 1-3, General – Cold formed thin gauge members and sheeting, 2006

British Standards Institution BS EN 1993-1-5: Eurocode 3. Part 1.5, Design of steel structures. Plated structural elements, 2006

British Standards Institution BS EN 10346: Continuously hot-dip coated steel flat products. Technical delivery conditions, 2009

Shafer, B.W., 2013, ‘‘Sheathing Braced Design of Wall Studs’ Final Report to the American Iron and Steel Institute, Washington

Standards Australia AS4600 Cold formed steel structures, 1996

Tian Y.S., Wang J, and Lu T.J., 2007 Axial load capacity of cold formed wall studs and sheathing, Thin Walled Structures Vol 45 Issue 5 pp 539 to 551

Telue Y and Mahendran M., 2001, Behaviour of cold formed steel wall frames lined with plasterboard, Journal of Constructional Steel Research 57 pp 435-452.

Accepted manuscript doi: 10.1680/jstbu.18.00118

Vieira, L.C.M., Schafer, B.W. (2013). “Behavior and design of sheathed cold-formed steel stud walls under compression.” American Society of Civil Engrs. Journal of Structural Engineering 139 (5), pp772-786

Vieira, L.C.M.,(2011). “Lateral stiffness and strength of sheathed braced cold formed studs in compression.” Journal Constructional Steel Research 167 (10), pp1554-1566

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Table 1: Failure loads for 75 mm and 200mm deep un-lipped C sections in two steel grades with plasterboards on one or both sides (Telue and Mahendran)

Walls with plasterboard on one side (L= 2.4 m)		**75mm deep C sections**		**200mm deep C sections**	
		C75x30x1.15 S175 steel	**C75x30x1.15 S500 steel**	**C200x35x1.15 S175 steel**	**C200x35x1.15 S500 steel**
a.	Bare steel C section fixed to U track, and with:	5.5 kN	7.2 kN	9.5 kN	10.7 kN
b.	Boards on one side- C sections at 300mm spacing	16.7 kN	28.3 kN	15.0 kN	18.2 kN
c.	Boards on one side- C sections at 600mm spacing	18.5 kN	26.4 kN	14.3 kN	19.4 kN
d.	Boards on both sides -C sections at 300mm spacing	19.0 kN	36.6 kN	22.3 kN	38.3 kN
e.	Boards on both sides -C sections at 600mm spacing	21.3 kN	35.7 kN	22.0kN	41.7 kN

Table 2: Failure loads for 90 mm deep x 39/42mm wide C sections with sheathing boards on one side (Wang et al)

Wall panel of 2.45m height using 90x 39/42mm Cs	**Bare steel panel**	**Boards on one flange of C section**		
		OSB board	**CSB board**	**CPB Board**
C section fixed to U track, and with:	15.6 kN	-	-	-
Board fixings at 300mm spacing	-	28.2 kN	29.0 kN	32.4 kN
Board fixings at 400mm spacing	-	27.9 kN	-	-
Board fixings at 600mm spacing	-	25.6 kN	-	26.1 kN

Table 3: Failure loads for 92 mm deep x 41mm wide C sections with sheathing boards on one and both sides (Vieira and Schafer, 2013)

Wall panel of 2.44m height using 92x 41x 1.72mm Cs	**Number of tests**	**Mean of test failure loads**	**Coeff. of variation**	**Characteristic failure load**	**Failure load predicted by this paper**
C section fixed to U track, and with:	1	50.1 kN	-	41.1 kN	32.1 kN
12.5mm OSB board on one side	3	78.0 kN	6.3%	63.7 kN	58.5 kN
12.5mm gypsum board on both sides	3	85.7 kN	2.3%	77.1 kN*	66.7 kN[X]
11mm OSB board on both sides	2	95.9 kN	-	86.3 kN*	83.8kN based on major axis buckling
11mm OSB board on one side /12.5mm gypsum board on other side	3	93.3 kN	1.5%	83.9 kN*	

[+]Characteristic value is based on 0.82 x mean failure load, based on the case of boards on one side
*Characteristic value is based on 0.9 x mean failure load
[X] Predicted failure load is based on effective length factor of 0.5 in minor axis buckling

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Table 4: Failure loads for tests on walls using 100 x 50 x 1.5mm C and 15mm moisture resisting plasterboard

Restraint by 15 mm moisture resistant (Type H) plasterboards on one or both sides (L= 2.42 m)		Failure Load, P_{test} (per C section)	Notional characteristic failure load
a.	No boards – plain panel (reference test failing by torsional flexural buckling)	45.2 kN	0.85 P_{test} = 38.4 kN
b.	Light steel wall panel with: Side 1 – 15 mm moisture resistant plasterboard directly fixed Side 2 – No boards	58.7 kN 57.0 kN 61.2 kN 64.5 kN Mean 60.3 kN	53.3 kN (88% of mean)
c.	Light steel wall panel with: Side 1 – 15 mm moisture resistant plasterboard directly fixed Side 2 – 15 mm plasterboard directly fixed	84.2 kN	0.85 P_{test} = 71.6 kN
d.	Light steel panel with: Side 1 – Resilient bars fixed on one side at 400 mm centres Side 2 – 15 mm plasterboard directly fixed	65.7 kN	0.85 P_{test} = 55.6 kN
e.	Light steel panel with: Side 1– 15 mm moisture resistant plasterboard directly fixed Side 2 – 15 mm plasterboard fixed via resilient bars placed at 400 mm cs	100 kN	0.85 P_{test} = 85 kN

Table 5: Failure loads for tests on walls using 100 x 50 x 1.6mm C and 12.5 mm fire resisting plasterboards

Restraint by 12.5mm fire resistant (Type F) plasterboards on one or both sides (L= 2.42 m)		Failure Load, P_{test} (per C section)	Notional characteristic failure load
a.	No boards – plain panel (reference test failing by torsional flexural buckling)	45.3 kN	0.85 P_{test} = 38.5 kN
b.	Light steel wall panel with: Side 1 – 12.5 mm board directly fixed Side 2 – No boards	58.5 kN 63.5 kN 66.3 kN Mean 62.8 kN	51.3 kN (82% of mean)
c.	Light steel wall panel with: Side 1 – 2 x 12.5 mm boards directly fixed Side 2 – No boards	76.0 kN	0.85 P_{test} = 64.6 kN
d.	Light steel wall panel with: Side 1 – 12.5 mm board directly fixed Side 2 – 12.5 mm board directly fixed	85.8 kN	0.85 P_{test} = 72.9 kN
e.	Light steel wall panel with: Side 1 – Resilient bars fixed on one side at 400 mm centres with 12.5 mm board Side 2 – No boards	58.5 kN	0.85 P_{test} = 49.7 kN
f.	Light steel wall panel with: Sides 1 and 2 – 12.5 mm boards fixed via resilient bars placed at 400 mm cs	85.0 kN	0.85 P_{test} = 72.2 kN

Table 6: Failure loads for tests on walls using 150 x 50 C and 12.5 mm fire resisting plasterboards

Restraint by 12.5 mm fire resistant (Type F) plasterboards on one or both sides (L= 2.42 m)		Failure Load, P_{test} (per C section)	Notional characteristic failure load
a.	No boards – plain wall 150 x 1.2 C panel (reference test failing by minor axis buckling)	33.2 kN	0.85 P_{test} = 28.2 kN
b.	Light steel wall panel using 150 x 1.2 C with: Side 1 – 12.5 mm board directly fixed Side 2 – No boards	42.3 kN 44.8 kN Mean 43.5 kN	0.85 P_{test} = 37.0 kN
c.	No boards – plain wall 150 x1.5 C panel (reference test failing by minor axis buckling)	53.5 kN	0.85 P_{test} = 45.4 kN
d.	Light steel wall panel with 150 x 1.5 C: Side 1 –12.5 mm boards directly fixed Side 2 – No boards	58.0 kN 60.8 kN 64.3 kN Mean 61.0 kN	51.8 kN (85% of mean)

Table 7: Failure loads for tests on 75 x 45 x 1.6 mm C using OSB or CPB on one side and two layers of 15 mm Type F plasterboard on the other side

Restraint by *OSB or CPB* on one side and 2 layers of 15mm plasterboard on the other side (L= 2.45 m)		Failure Load, P_{test} (per C section)	Notional characteristic failure load
a.	Light steel wall panel with: Side 1 – 11 mm *OSB* directly fixed Side 2 – No boards	64 kN	0.85 P_{test} = 54.4 kN
b.	Light steel wall panel with: Side 1 – 11 mm *OSB* directly fixed Side 2 – 2x15 mm plasterboard directly fixed	97 kN	0.85 P_{test} = 82.4 kN
c.	Light steel wall panel with: Side 1 – 10 mm *CPB* directly fixed Side 2 – 2x 15 mm plasterboard directly fixed	96 kN	0.85 P_{test} = 81.6 kN
d.	Light steel wall panel with: Side 1 – 11 mm *OSB* directly fixed Side 2 – 2x 15 mm plasterboards directly fixed 10 mm eccentricity of load application	79 kN	0.85 P_{test} = 67.1 kN
e.	Light steel wall panel with: Side 1 – 11 mm *OSB* directly fixed Side 2 – 2x 15 mm plasterboards directly fixed 20 mm eccentricity of load application	62 kN	0.85 P_{test} = 52.7 kN

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Table 8: Comparison of test results with the theoretical buckling resistance to EN 1993-1-3 and the effective length factor obtained from the theory

Test configuration	Section size	Test failure load	Effective length factor	Design load by theory	Test/ Theory	Statistical results for Test/Theory
No boards	100x50x1.5mm C	45.2 kN	0.85	37.2 kN	1.21	Mean = 1.23 Var.=0.086 Char. = 1.03
	100x50x1.6mm C	45.3 kN		40.1 kN	1.13	
	150x50x1.2mm C	33.2 kN		26.9 kN	1.23	
	150x50x1.5mm C	53.5kN		39.9 kN	1.34	
Boards fixed on one side	100x50x 1.5mm C with 15mm Type H plasterboards	58.7 kN	0.68	50.7 kN	1.16	Mean = 1.19 Var.=0.057 Char. = 1.06
		57.0 kN			1.12	
		61.2 kN			1.21	
		64.5 kN			1.27	
	100x50x 1.6mm C with 12.5mm Type F plasterboards	58.5 kN	0.70	52.6 kN	1.11	Mean = 1.19 Var.=0.056 Char. =1.03
		63.5 kN			1.21	
		66.3 kN			1.26	
	150x50x 1.2mm C with 12.5mm Type F plasterboards	42.3 kN	0.81	30.5 kN	1.38	Mean = 1.42 Var.=0.062 Char. =1.28
		44.8 kN			1.47	
	150x50x 1.5mm C with 12.5mm Type F plasterboards	58.0 kN	0.80	44.1 kN	1.31	
		60.8 kN			1.38	
		64.3 kN			1.46	
	100x50x 1.5mm C with 2x12.5mm Type F plasterboards	76.0 kN	0.65	57.0 kN	1.33	
	75 x45x 1.6mm C with 11mm OSB	64.0 kN	0.5	58.1 kN	1.14	
Boards fixed on both sides	100x50x 1.5mm C with 15mm Type H boards	84.2 kN	0.5	67.0 kN	1.25	Mean = 1.27 Var.=0.015 Char. =1.22
	100x50x 1.6mm C 12.5mm Type F boards inc. resilient bars	85.0 kN	0.5	67.0 kN	1.27	
		85.8 kN			1.28	
	75 x45x 1.6mm C 11mm OSB /2x15 mm plasterboards	97 kN	0.5	58.1 kN (or 73.6 kN with composite action of boards)	1.67	Mean = 1.66 (or 1.31 with composite action of boards)
	75 x45x 1.6mm C 10mm CPB /2x15 mm plasterboards	96 kN	0.5		1.65	

Var. =variance of test failure load/theory
Char. =characteristic value of test failure load/theory depending on number of tests in a series
For the bare frame tests, the theory is based on an effective length factor of 0.85 for flexural buckling using buckling curve 'b' to BS EN 1993-1-3.

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Table 9: Effective length factors for C-sections with a single or double layer of Type F or H plasterboards on one side for design to BS EN 1993-1-3

C section size x thickness	Wall height (mm)	Effective Length Factor for Minor Axis with Boards on Flange	
		Single layer of plasterboard	Double layer of plasterboard
100 x 50 x 1.2	2400	0.75	0.72
	2700	0.70	0.67
100 x 50 x 1.6	2400	0.72	0.67
	2700	0.65	0.60
	3000	0.61	0.60 min.
100 x 45 x 1.6	2400	0.69	0.63
	2700	0.63	0.60 min.
150 x 50 x 1.2	2400	0.81	0.80
	3000	0.76	0.75
150 x 50 x 1.6	2400	0.80	0.78
	3000	0.74	0.70

Note: Data for 12.5mm or 15mm Type F or Type H plasterboard fixed at not more than 300mm spacing

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Table 10: Comparison of test results with the theoretical buckling resistance to BS 5950-5 and the effective length factor obtained from the theory

Test configuration	Section size	Test failure load	Effective length factor	Max. load by theory	Test/ Theory	Statistical results
No boards	100x50x1.5mm C	45.2 kN	0.95	35.0 kN	1.29	Mean = 1.26 Var.=0.092 Char. = 1.04
	100x50x1.6mm C	45.3 kN		37.5 kN	1.21	
	150x50x1.2mm C	33.2 kN		28.5 kN	1.16	
	150x50x1.5mm C	53.5kN		39.1 kN	1.37	
Boards fixed on one side	100x50x 1.5mm C with 15mm plasterboards	58.7 kN	0.76	51.0 kN	1.13	Mean = 1.26 Var.=0.092 Char. = 1.05
		57.0 kN			1.12	
		61.2 kN			1.20	
		64.5 kN			1.26	
	100x50x 1.6mm C with 12.5mm plasterboards	58.5 kN	0.78	51.9 kN	1.13	Mean = 1.19 Var.=0.049 Char. = 1.05
		63.5 kN			1.22	
		62.8 kN			1.21	
	150x50x 1.2mm C 12.5mm boards	42.3 kN	0.91	30.7 kN	1.38	Mean = 1.42
		44.8 kN			1.46	
	150x50x 1.5mm C 12.5mm boards	58.0 kN	0.89	43.3 kN	1.34	Mean = 1.41 Var.=0.07 Char. = 1.20
		60.8 kN			1.40	
		64.3 kN			1.48	
	100x50x 1.5mm C 2 x 12.5mm boards	76.6 kN	0.74	56.8 kN	1.28	
	75 x45x 1.6mm C with 11mm OSB	64.0 kN	0.6 min	55.4 kN	1.16	
Boards fixed on both sides	100x50x 1.5mm C 15mm boards	84.2 kN	0.6 min	69.1 kN	1.22	Mean = 1.18 Var.=0.029 Char. = 1.09
	100x50x 1.6mm C 12.5mm boards inc. resilient bars	85.0 kN	0.6 min	73.6 kN	1.15	
		85.8 kN			1.16	
	75 x45x 1.6mm C 11mm OSB /2x15 mm plasterboards	97kN	0.6 min	55.2 kN or 72.0 kN inc. composite action of boards	1.75	Mean =1.74
	75 x45x 1.6mm C 10mm CPB /2x15 mm plasterboards	96 kN	0.6 min		1.74	

P_k = characteristic failure load based on the group of tests
P_{av} = average failure load of the group of tests
σ=variance of test results from average
For the bare frame tests, the theory is based on flexural buckling with an effective length factor of 0.95 using the buckling curve to BS 5950-5

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List of figures

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Fig 1.jpg

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Fig 2.jpg

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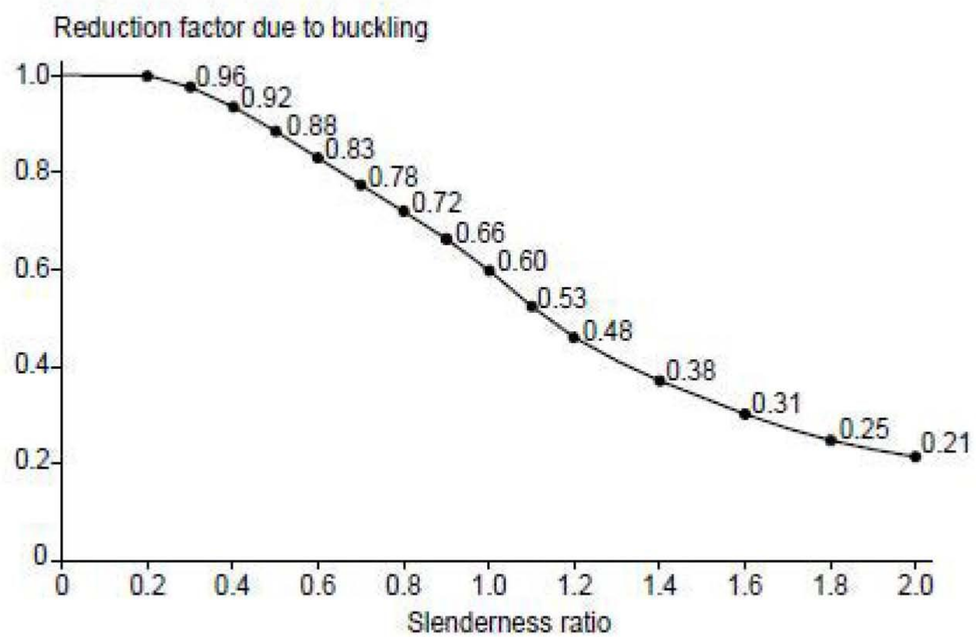

Fig 3.jpg

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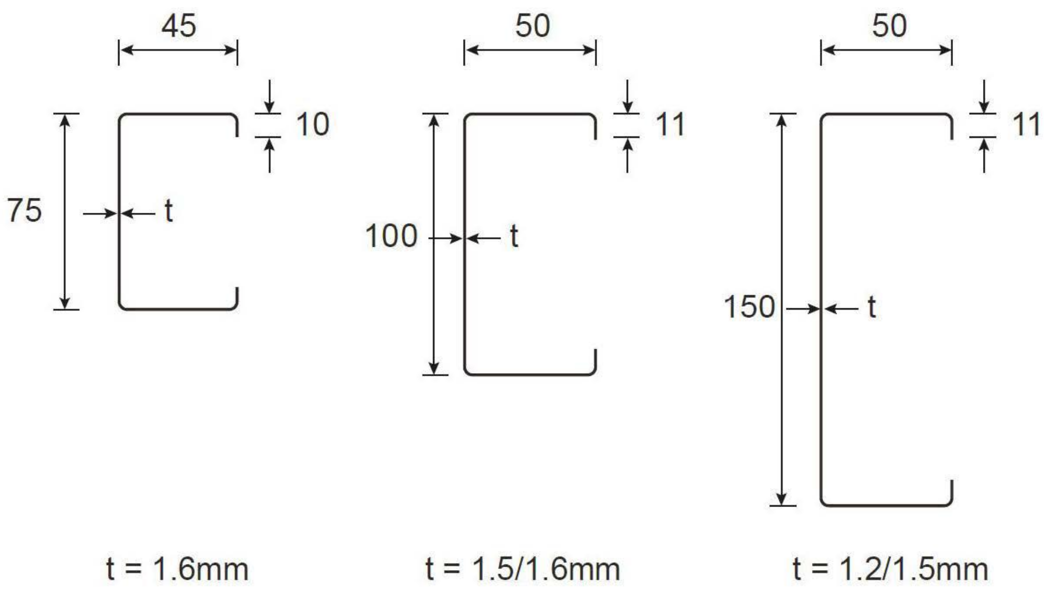

Fig 4.JPG

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Fig 5r.jpg

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Fig 6r.jpg

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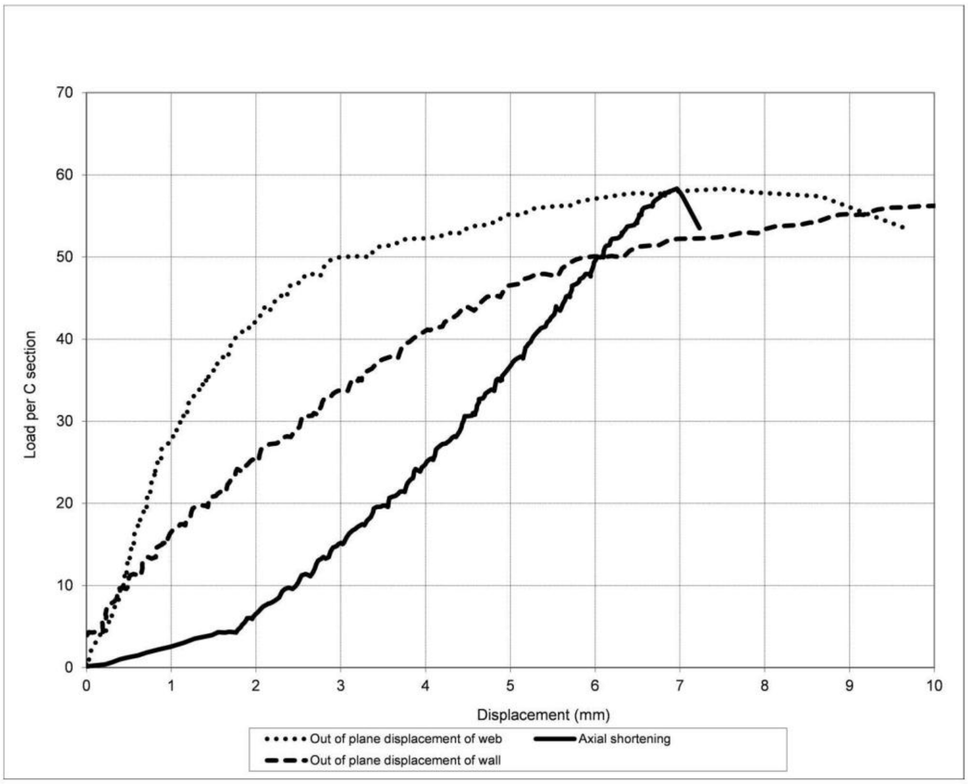

Fig 7r.tif

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Fig 8r.jpg

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Fig 9r.jpg

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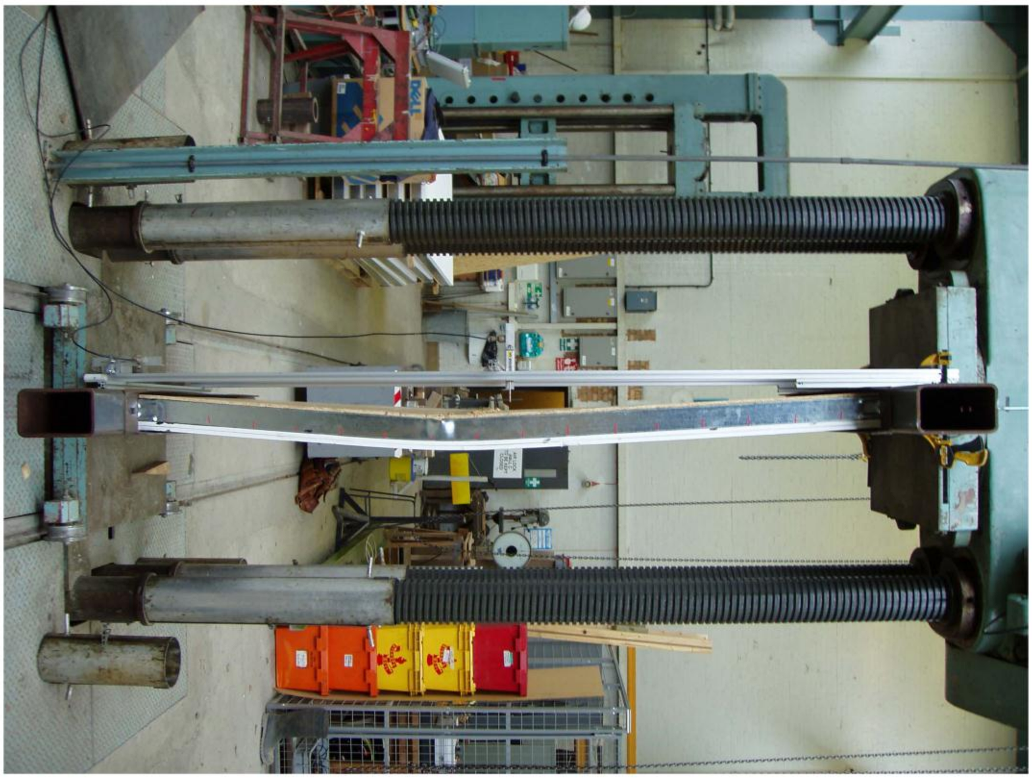

Fig 10r.jpg

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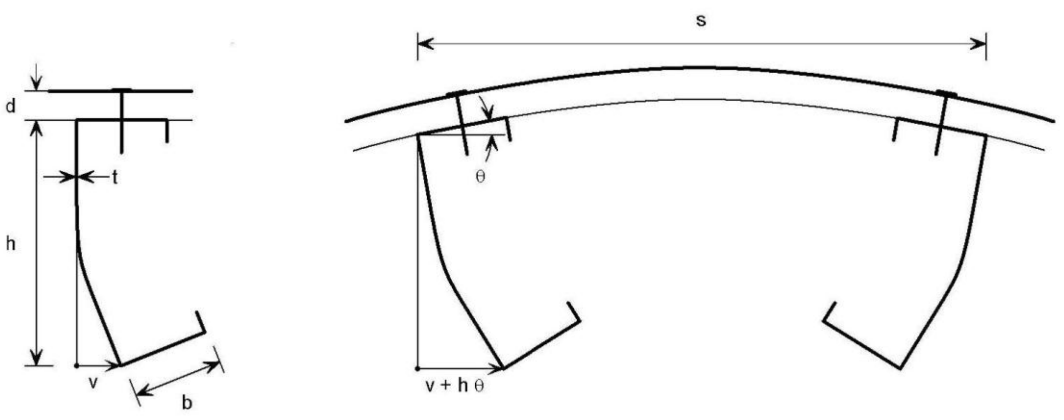

(a) Rigid connection to board

(b) Flexible connection to board

Fig 11r.jpg

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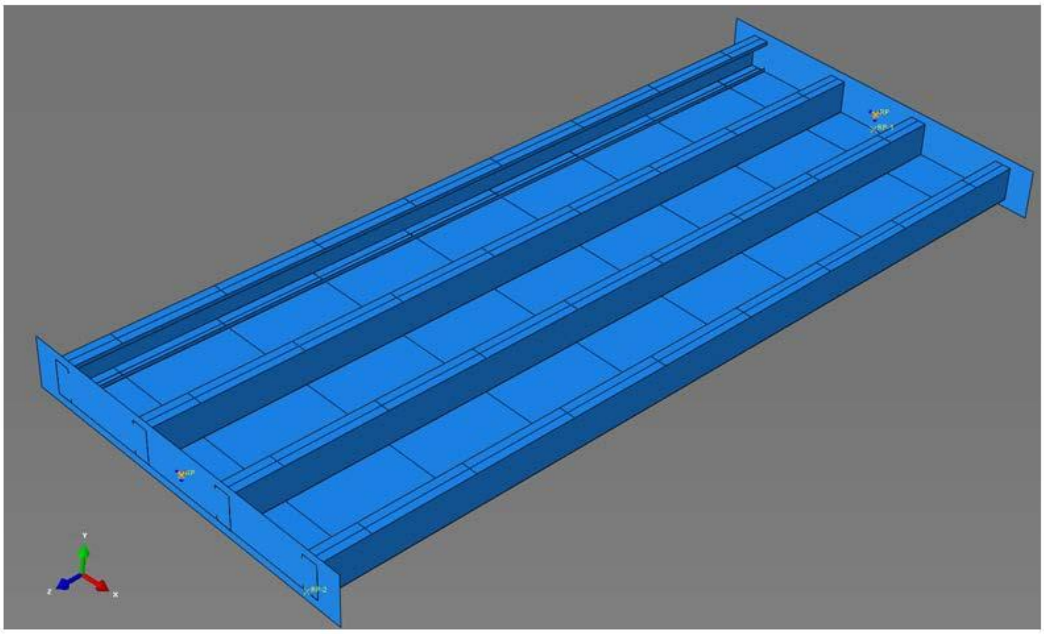

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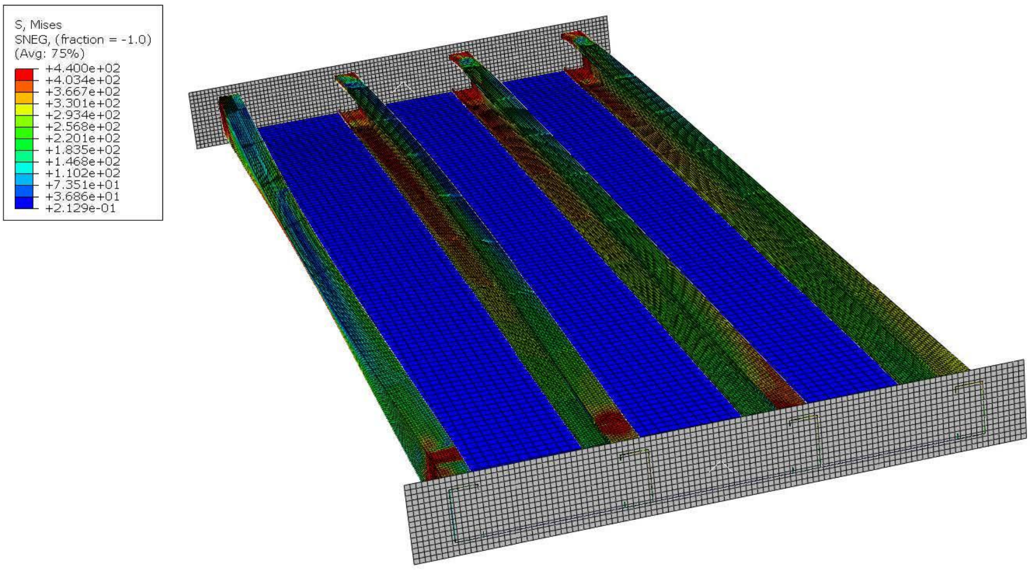

Fig 13r.tif

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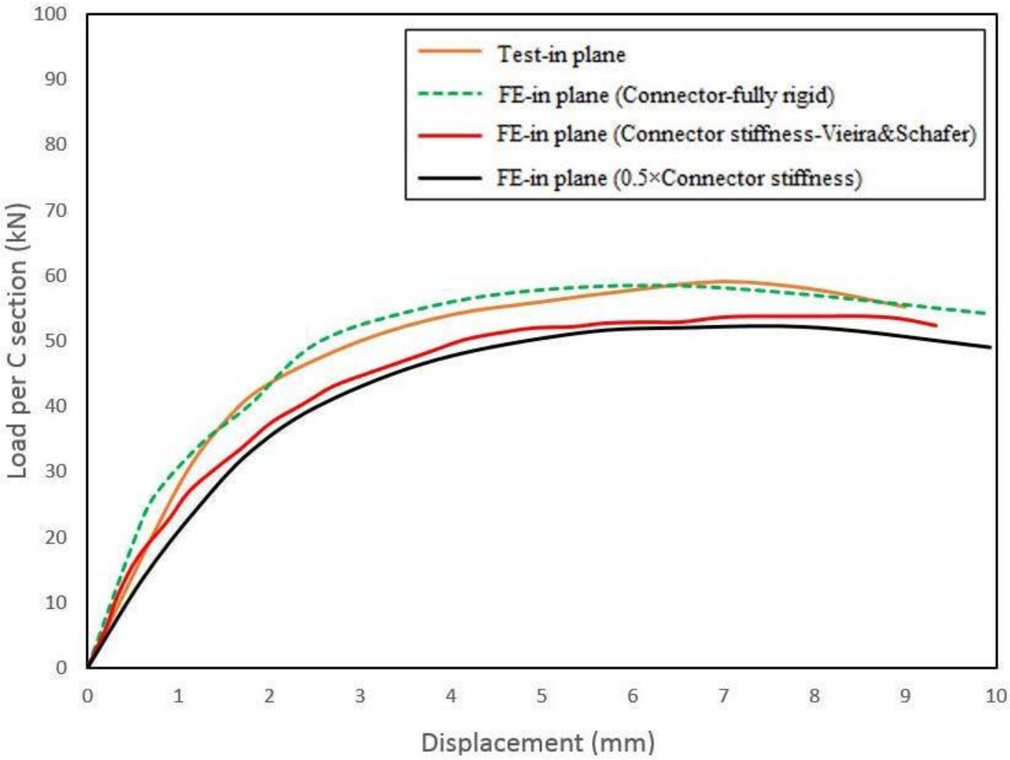

Fig 14r.JPG

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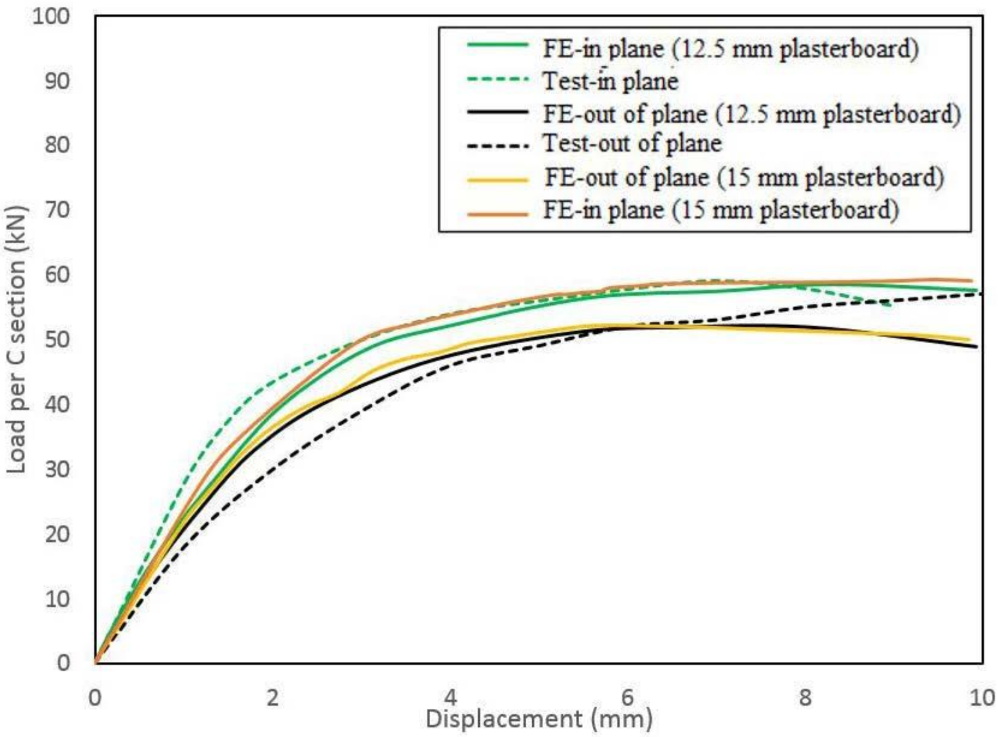

Fig 15r.JPG