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Developments of Cold-Formed Steel Sections in Composite Applications for Residential Buildings

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Abstract: Cold-formed steel sections are often designed and built to act compositely with other constructional materials, particularly with wood and cementitious materials in order to improve their structural performance. This paper reviews some recent developments in composite applications of cold-formed steel sections in the U.K., and presents a number of design recommendations for these forms of structures in residential buildings.

Key words: cold-formed steel sections, steel-timber floor joists, steel-timber open roof systems, composite lattice joists.

1. INTRODUCTION

There is an increasing demand for more industrialised production of cold-formed steel sections that take advantage of composite actions with other constructional materials. This paper presents some recent developments in light steel floors and roofs in residential buildings which use light steel composites, as follows:

- light steel-timber floor beams
- 'open' roof systems using steel-timber composites
- gypsum composite floors with steel lattice joists
- light steel slim floors

The main application of these composite members is in residential and medium-rise buildings where the benefits of longer spans and lightweight construction can be realised. The structural performance of these members is compared to the proposed guidance on serviceability limits of lightweight floors.

2. TESTS ON STEEL-TIMBER FLOOR JOISTS

A prototype steel-timber floor joist has been developed in which cold-formed steel C or T sections provide the stiff 'flanges' for bending resistance of the floor joist, and a plywood web provides its shear resistance. A range of joist depths can be manufactured by varying the depth of the plywood in order to suit the design requirements, and openings can be provided easily in the web. A series of tests was carried out to investigate the relative performance of different types of board materials for a 300 mm deep steel-timber floor joist. It was shown that a 12 mm plywood gave the optimum results. Four prototypes were investigated in order to assess the structural performance of floor joists, and the the test specimens consisted of pairs of steel-timber floor joists placed at 600 mm centres with a span of 5.75 m between centres of supports. The joist depth of 345 mm was selected based on calculations of predicted performance, and the span was typical of the clear span between cross-walls in a house. Floor boarding was screwed and glued to the joists and the joist ends were stabilised by plywood diaphragms. Figure 1 shows the four different joist configurations that were tested:

Test T1. 50 mm × 50 mm × 1.8 mm thick cold-formed Tee section flanges with a single 12 mm thick plywood web fixed by countersunk screws at 150 mm centres.

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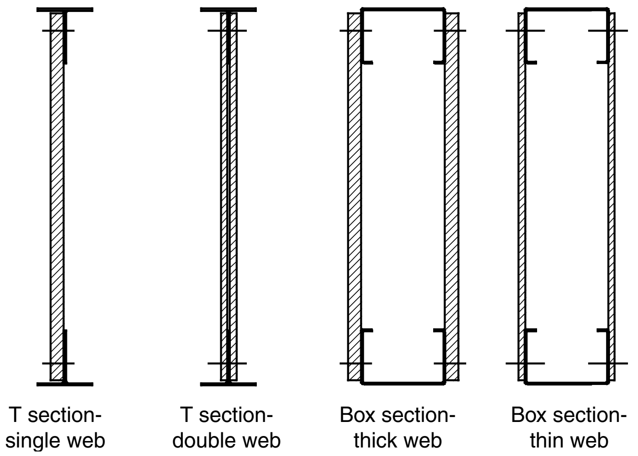

Figure 1. Steel-timber joist configurations using T and C section flanges

Test T2. Tee flanges (as in Test T1) with two 6 mm thick plywood webs fixed by hexagonal head screws at 200 mm centres.

Test C1. 77 mm web × 50 mm flange × 1.6 mm thick cold-formed C section flanges attached to two 12 mm thick plywood webs fixed by countersunk screws at 200 mm centres.

Test C2. C sections (as in Test C1) attached to two 6 mm thick plywood webs fixed by hexagonal head screws at 200 mm centres.

It should be noted that hexagonal head screws were used for the 6 mm thick plywood as countersunk screws caused local damage to it. The board pattern adopted for the double web joists ensured that joints were staggered on opposite sides of the 'box-shaped' joist.

The floor assemblies were subject to a uniformly distributed load, applied using a vacuum test rig. All the tests were carried out incrementally to a pressure of 6 kN/m^2 (the maximum capability of the vacuum test rig) applied to the flooring. No sign of failure was observed in tests C1 and C2, but separation of the boards occurred at approximately 5.6 kN/m^2 in tests T1 and T2. Both tests C1 and C2 were not tested to failure as a load of 6 kN/m^2 exceeded the maximum factored loading that might be applied to the floor joists in practice.

A further test was carried out on a modified floor joist type C1 using a pair of 225 mm deep box-shaped joists of 4.7 m clear span (or 4.8 m actual length) with a joint in the plywood web at mid-pan. The flanges comprised 65 mm web × 42 mm flange × 1.2 mm thick cold-formed C sections to which 12 mm thick plywood boards were fixed using ballistically-driven nails at 150 mm centres. The nominal diameter of the nails was 1.5 mm. This test gave an opportunity of examining a fixing type more suitable for rapid production, and the span to depth ratio of the floor joist is 21, which is a practical maximum for this type of flooring.

As shown in Figure 2, a central point load was applied to the test specimen in the floor joist in order to create a uniform shear force in the web. The load-displacement graph is presented in Figure 3. The test was terminated when the deflection reached span/60, as shown in Figure 4. At this deflection, it was demonstrated that there was considerable deformation capacity in the fixings, which reached a slip of over 3 mm. The test specimen failed at a shear load corresponding to an equivalent uniformly distributed load of 4.6 kN/m^2 for floor joists spaced at 600 mm, which is acceptable for domestic applications.

Figure 2. Test arrangement on a pair of 4.7 m span steel-timber joists under a central point load

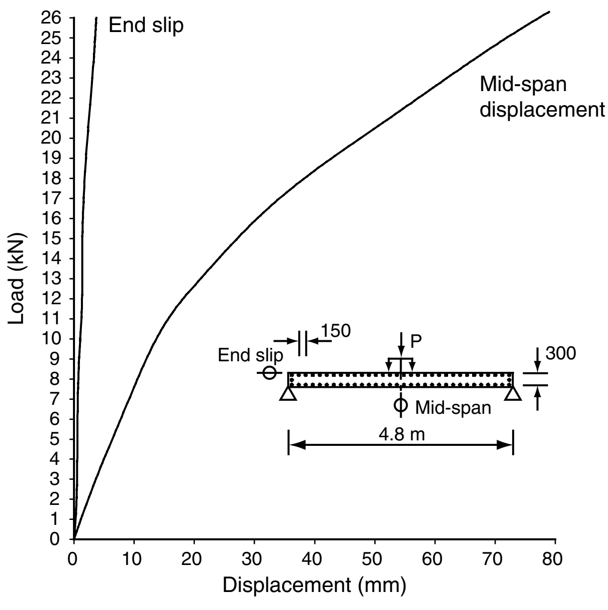

Figure 3. Load–displacement graph for a pair of steel-timber joists

3. ANALYSIS OF PERFORMANCE OF STEEL-TIMBER FLOOR JOIST

The theoretical second moment of area of the floor joists is determined on the basis of the cold-formed steel sections alone, ignoring the contribution from the plywood web, according to:

$$I_{\text{eff}} = A_{\text{eff}}\, d_{\text{eff}}^{2}/2 \tag{1}$$

where

A_{eff} is the effective cross-sectional area of the cold-formed steel section in compression

d_{eff} is the effective depth between the centroids of the sections ($d_{\text{eff}} \approx d - 25$ mm)

d is the joist depth

The shear resistance of a single plywood web is established from the shear flow in the web which is given by:

$$V_{\text{web}} = \frac{P_{\text{d}} d_{\text{eff}}}{S_{\text{x}}}\left(1 + \frac{2}{3}\frac{s_{\text{x}}}{s_{y}}\frac{d}{b}\right) \tag{2}$$

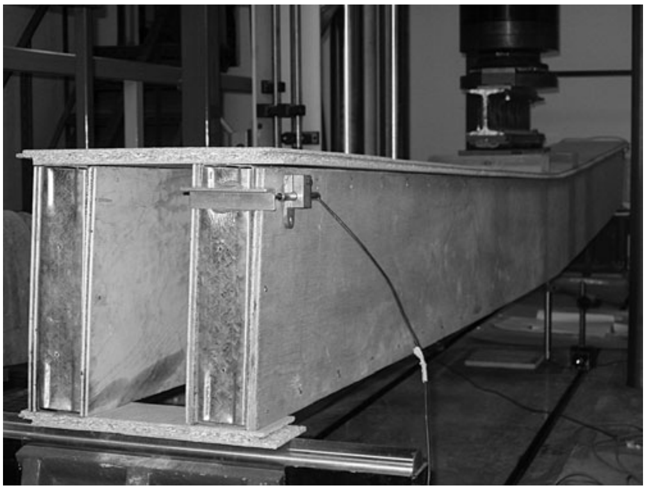

Figure 4. Steel-timber joist at failure under a central point load

where

P_d is the design resistance of a fixing between the board and the cold-formed steel section

b is the board length along the beam length

s_x is the fixing spacing along the beam length

s_y is the fixing spacing vertically along the beam depth

It should be noted that the behaviour of the joists was demonstrated to be 'ductile' and no brittle failure occurred either in the plywood web or its screws. However, the measured deflections of the joists at a service load of 1.5 kN/m^2 were greater than the predicted values under pure bending, as given in Table 1; the difference is due to slip in the screw or nail attachments. Hence, it is necessary to examine the acceptable serviceability limits and they are presented in the next section.

The test results clearly show that the performance of the single web joists T1 was more affected by the shear force in the plywood web and the slip in its fixings. In tests C1 and C2, the joists behaved well and their shear deflections were relatively small. The modified joist C1 using ballistic nails experienced slightly larger shear deflections than the joists using screws. At failure, the shear resistance of the individual fixings may be calculated from Eqn 2, and is given to be 2.1 kN for test T1 and 2.4 kN for test C1 modified. The slip causing separation of the boards at the failure load is illustrated in Figure 5. At service loading, the shear flexibility of the fixings is estimated at approximately 0.5 mm/kN. The calculated natural frequency of the prototype joists was between 7.9 and 10 Hz. It is concluded that the box-shaped joist type C1 could be developed further to practical realisation using ballistic nails to connect the plywoods to the cold-formed steel section.

Table 1. Comparison of predicted and actual load displacement characteristics under service loads on steel-timber joists

Joist	Configuration	Flange	Fixings (centres)	Pure bending deflection (mm)	Actual deflection (mm)	Difference (shear deflection) (mm)
T1	Single web	T section	Screws at 150 mm cs	4.8	10.0	5.2
T2	Double web	T section	Screws at 200 mm cs	4.8	7.0	2.2
C1	Box	C section	Screws at 200 mm cs	5.5	6.3	0.8
C2	Box	C section	Screws at 200 mm cs	5.5	6.5	1.0
C1 (modified)	Box	C section	Nails at 150 mm cs	7.7	10.5	2.8

Predicted deflection based on the second moment of area of composite section

All deflections are based on a service load of 1.5 kN/m^2 for joist spacings of 600 mm

Figure 5. Local deformation at joint in plywood web

4. DESIGN REQUIREMENTS FOR LIGHTWEIGHT FLOORS

In general, the serviceability performance of light weight floors often controls their design, but this is not adequately covered by BS5950. (1997). The Australian Standard AS 3623 (1993) for domestic metal framing gives a total deflection limit of span/250, but this is coupled with a limiting deflection of 2 mm under a 1 kN point load (see below). The Canadian Standards (CAN/CSA-S16.1-94. 2000; CSSB 51M-91. 1991) specify a maximum deflection limit of span/360 when subjected to a uniformly distributed load of 2kN/m^2. More recently, a criterion (Kraus and Murray 1997) is proposed which broadly represents the acceptable limit for sensitivity to floor vibrations. Similar guidance has been proposed at a European level, where the floor sensitivity is presented in terms of a classification ranking. It should be noted that Class 2 application corresponds to housing, and for this class, a limiting deflection of 1.5 mm under a 1 kN point load is proposed.

On the basis of the testing and their recent tests on lightweight floors (ECSC. 2001), the following static and dynamic design criteria are recommended (Gorgolewski *et al*. 2001) in order to assess the serviceability performance of light steel composite and other types of floor joists:

5. STATIC DEFLECTION CRITERIA

Criterion a) The maximum deflection under dead and imposed loads is limited to span/350, or a maximum of 12 mm, including the effect of composite actions – this generally ensures that the minimum natural frequency is satisfied (see criterion *c*) below).

Criterion b) The maximum deflection under imposed loads is limited to span/450 - this only applies to floor areas under higher imposed loads, such as corridors and public areas, as in general, criterion *a*) will control.

6. VIBRATION SENSITIVITY CRITERIA

Criterion c) The natural frequency of lightweight floors should exceed 8 Hz for the loading case of self-weight plus 0.3 kN/m^2, which represents the permanent loading considered in domestic buildings. This criterion is satisfied by limiting the maximum deflection of the floor to 5 mm for this loading condition. The natural frequency limit should be increased to 10 Hz for corridors and public areas, where impulsive actions may increase.

Criterion d) The local deflection of the floor, using the relevant value of N_{eff} under a nominal 1 kN point load, is limited to a maximum of 1.5 mm or (3/span$^{0.5}$) for spans (in m) exceeding 4 m, based on the criteria suggested by Kraus and Murray (1997). This reduction in deflection limit with span reflects the need for higher stiffness to counteract the increase in the possibility of impulsive actions that may occur in longer spans.

For domestic buildings, the governing criterion is most likely to be criterion *d*) when $N_{\text{eff}} = 2.5$. For separating floors, or for a design imposed load in excess of 1.5 kN/m^2, the governing criterion is likely to be criterion *a*).

7. STEEL-TIMBER OPEN ROOF SYSTEM

A steel-timber composite 'open' roofing system has been developed using a technology similar to that used for the floors, which creates a long spanning and adaptable roofing solution for modern housing. A prototype open roof design is shown in Figure 6. The flanges of the floor beam and the rafters are made from the same C steel sections, and the plywood infills between these sections provide the necessary rigidity and transfer forces at the corners of the roof truss. Any span or slope of the roof can be accomplished by simply modifying the shape and size of the plywood infills. In general, the bottom chord of the roof truss supports loads transferred from the rafters, and therefore its slope is relatively deep.

The prototype steel-timber roof truss under testing is illustrated in Figure 7. Its key dimensions are 8 m span and 35° roof slope, creating an internal habitable space of 4.4 m width and 2.3 m height. The roof trusses are placed at 600 mm centres. The main load-bearing

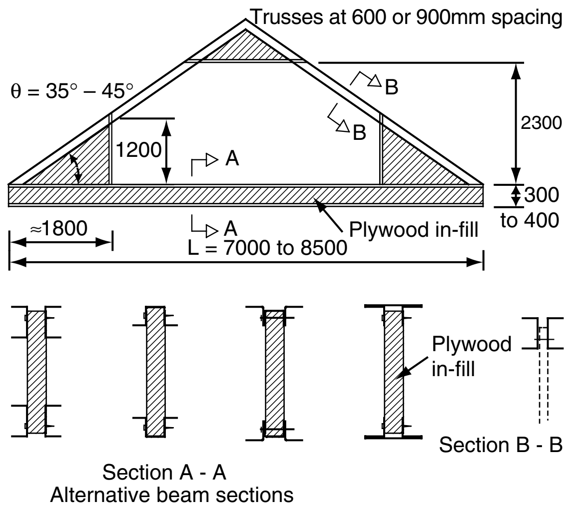

Figure 6. Prototype steel-timber roof system with various types of flanges

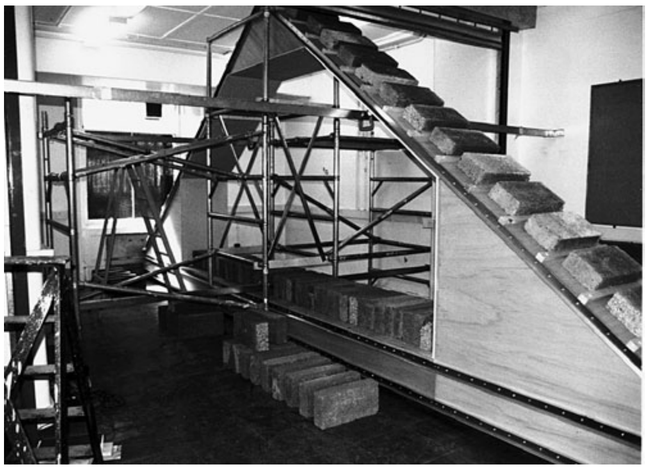

Figure 7. Test arrangement on a pair of 8 m span steel-timber roof truss under distributed loads

component is a 300 mm deep floor beam, which comprises two 60 mm web × 25 mm flange × 1.2 mm thick C sections and their flanges are screwed to 12 mm thick plywoods at 100 mm centres. The rafters use the same C sections placed back to back and act in compression and bending. This roof configuration achieves a clear span with a floor span to depth ratio of 24 and provides for maximum flexibility in space use.

8. LOAD TESTING OF PROTOTYPE ROOF TRUSS

The behaviour of the prototype roof truss is potentially complex and a load test was carried out to assess its performance under service and factored loads. Two roof trusses were tested together to provide stability. Loading was applied by concrete blocks weighing 19 kg each. Because two trusses were tested, the applied load had to be increased by a factor of two in terms of the equivalent load per square metre of the floor or roof in a real structure. The test series and the applied load were as follows:

Test 1 Service load applied to habitable floor space
775 kg or 1.5 kN/m^2
Test 2 Service load applied to roof
604 kg or 0.63 kN/m^2
Test 3 Factored load applied to floor
1229 kg or 2.4 kN/m^2
Test 4 Test to failure by additional load applied to floor
3450 kg or 4.5 kN/m^2

The self weight of the roof truss was approximately 0.3 kN/m^2. Deflections were measured at mid-position of the floor beams and the rafters, and the results of Tests 1 to 3 are presented in sequence in Figure 8. The deflection of the floor beam under the service load applied to the roof and the floor was 8.5 mm (span/960), increasing to 15.5 mm under the design factored load. The loads were sustained for 3 weeks, and a small creep deflection was recorded (< 1 mm). The loads were removed without any residual deflection, indicating that the system was not close to failure. The final load test was carried out 3 months later and reached a uniformly distributed load on the floor of 4.5 kN/m^2 (whilst maintaining a constant roof load of 1.1 kN/m^2). No failure occurred at this load despite the 80 mm deflection of the truss, which indicates that the roof system is not subject to a brittle failure mode. This maximum test load corresponds to 3 times the design load for an occupied roof space.

The behaviour of the prototype roof truss can be analysed elastically using the properties of the cold-formed steel sections and the plywoods. Conservatively, all the loads can be considered as applied to the bottom chord of the roof truss acting as a beam. The

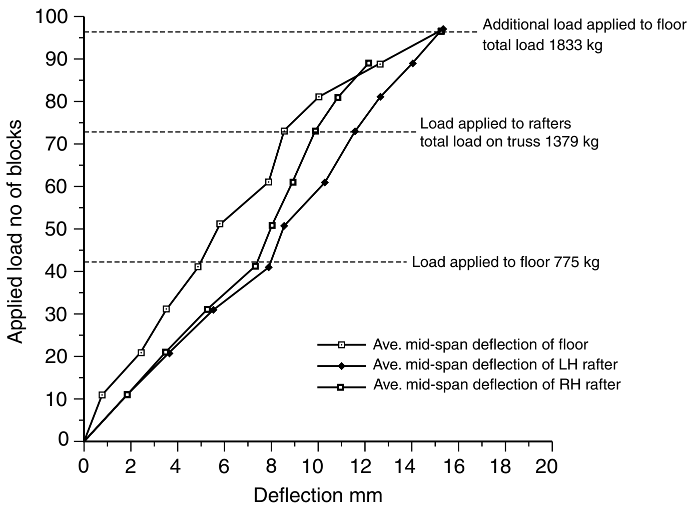

Figure 8. Load-deflection curve for service loads in roof truss tests

rafter can be considered as a fixed-ended strut subjected to local bending and compression. Out of plane buckling was prevented by the roof battens but local buckling of the rafters was observed at approximately 80% of the failure load.

The stiffness of the floor beam is increased due to the end fixity of the plywood 'gussets' and the compression forces developed in the rafter due to the end rotation of the beam. Its measured deflection was approximately 30% of the equivalent simply supported 8 m span 'plyweb' beam and corresponds to an effective span of approximately 6 m (i.e. the actual span less the width of one corner insert). The self weight of this composite roof truss is also 20% less than the equivalent timber open roof. This new open roofing technology is being developed further to determine the most effective shape of the light steel composite sections as well as the fixing technique.

9. TESTS ON COMPOSITE LATTICE JOISTS

Composite design is well developed in terms of the use of concrete slabs and hot rolled steel sections, and potentially it can lead to significant benefits in composite lattice joists using gypsum screeds. Gypsum screeds have definite benefits over concrete slabs in that they are self-evelling, and they can be placed as thin slabs on shallow profiled steel decking, giving relatively lightweight floors. At present, they are being promoted as a means of improving the stiffness of a lightweight floor and reducing its vibration sensitivity without excessively increasing its self-weight. The dry density of the gypsum is approximately 2000 kg/m^2, and so the total self weight of the composite lattice joists and decking with gypsum screeds is typically 1.3 to 1.5 kN/m^2, which is only half of that of a conventional composite slab, and less than 30% of an equivalent concrete slab.

A full-scale test on a pair of 5 m span composite lattice joists was carried out (Surrey report. 2004) to investigate composite action between the lattice joists and a 50 mm thick slab comprising gypsum screeds and a 16 mm deep profiled steel decking as formwork. The lattice joists were 225 mm deep and comprised 65 mm web × 42 mm flange × 1.2 mm thick C sections in S350 steel. The joists were placed at a spacing of 600 mm rather than the normal value of 400 mm due to the higher transverse stiffness of the slab. The shallow profiled steel decking was 0.6 mm thick and was fixed to the lattice joists by 4.8 mm diameter screws at a spacing of 300 mm.

The test arrangement is shown in Figure 9 and the load-deflection curve is shown in Figure 10. The pair of joists supported a total load of 27.2 kN applied as a central point load, which was well in excess of the predicted load capacity of 15 kN. A maximum deflection of 200 mm (span/25) was measured in the test, indicating excellent deformation capacity of all the components. The test results are compared to the calculated values in Table 2. The increase in stiffness

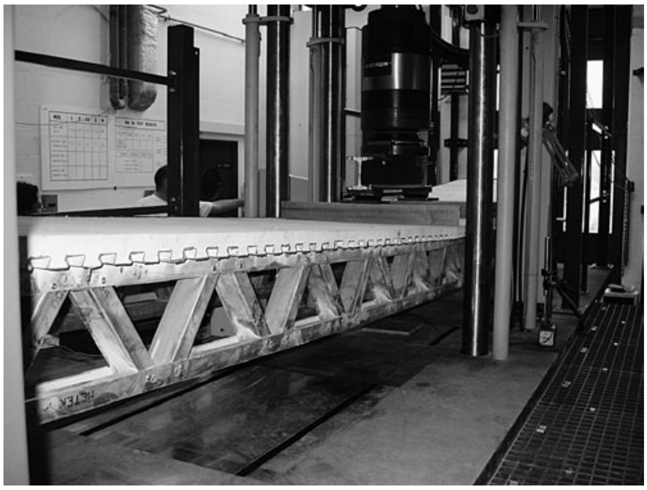

Figure 9. Test to failure of 5 m span composite lattice joists using gypsum screed

Table 2. Calculated and measured properties of composite lattice joists from test

Property per joist	Calculated values: Steel section	Calculated values: Composite section	Measured property from test	Model factor based on calculation method
Bending resistance (kNm)	8.3	10.5	18.1	1.72
Second moment of area (mm^4)	2.3×10^6	6.0×10^6	6.5×10^6	1.08
Shear resistance (kN)	10.1	–	7.7* at test load	1.31
Natural frequency (Hz)	6.3	10.2	10.5	Estimated from measured stiffness

Note: *based on a nominal shear resistance of 5.5 kN for single screw fixing.

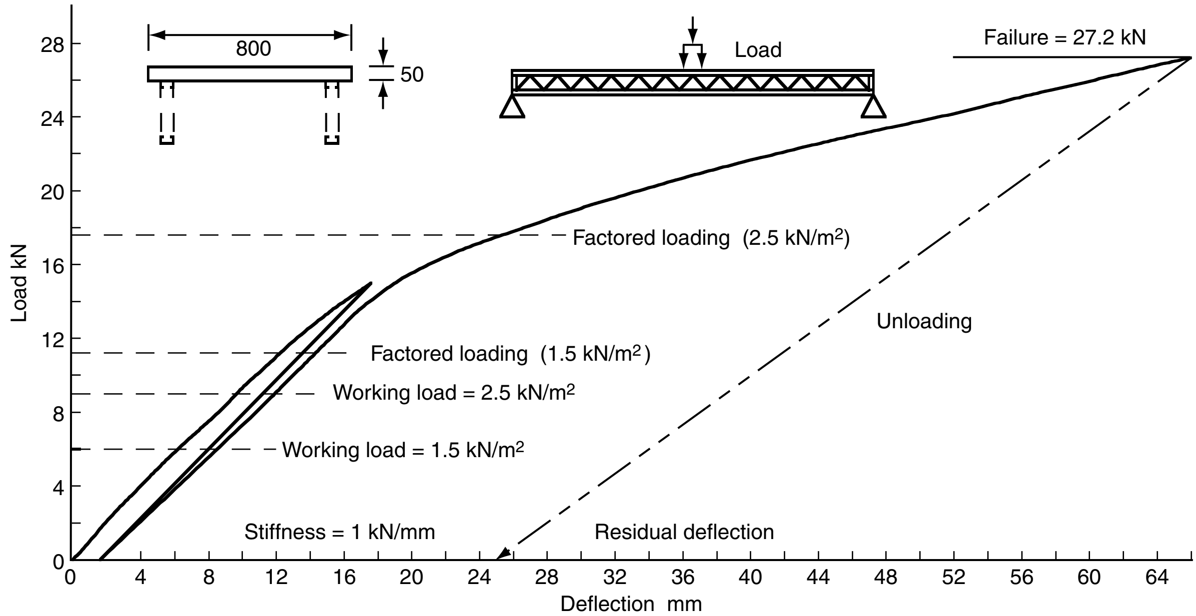

Figure 10. Load-deflection curve for 5m span composite lattice joist using gypsum screed

due to composite action of the gypsum screeds and the profiled steel decking with the floor joists was 180%, which potentially increases the spanning capabilities of the lattice joists by up to 25%.

The maximum compressive stress in the gypsum screeds was calculated to be only 5.2 N/mm^2, which is about 60% of its design compressive strength. Its elastic modulus is approximately 10 kN/mm^2 or one third of that of normal weight concrete. The simple composite model is based on the tensile action in the bottom chord of the lattice joists and the increased effective depth of the joist to the centre of the slab. The screws act effectively as shear connectors and the calculated shear force per screw was approximately 4.5 kN. The model factor on the composite moment resistance was 1.72, indicating that the simple composite model is conservative. The increase in the tensile resistance was due to the tensile strain hardening in the C section of the bottom chord, and to the local composite action of the top chord with the slab, which is not considered in the simplified model.

Similar small-scale bending tests were also carried out on 50 mm deep slabs using the gypsum screeds and the 16 mm deep profiled steel decking of 0.5 and 1 m spans. When analysed as a composite slab to Eurocode 4. (2004), the design shear-bond strength between the gypsum screeds and the shallow profiled steel decking was calculated as 0.13 N/mm^2, when expressed over the plan area of the shear span of the slab between the joists, i.e. with a nominal shear span at span/4. This shear-bond strength is comparable with modern composite slabs using concrete when analysed to Eurocode 4. (2004), and this demonstrates the excellent composite action between the gypsum screeds and the shallow profiled steel decking.

It should be noted that for resistance against a local point load, the calculated value of the effective number of steel joists, N_{eff}, based on the measured stiffness of the 50 mm thick composite lattice joists with gypsum slab was 5.5 for 225 mm deep lattice joists at 600 mm centres. When combined with the increased mass of the floor, the vibration response of these floors will be less than 20% of that of a conventional boarded floor, making this technology suitable for apartments and public-use buildings, where vibration sensitivity is very important. This relatively new composite technology has been used on recent projects and these designs conservatively take account only of the improved stiffness of the composite lattice joists, and did not utilise the improved moment resistance. The test work has been supplemented by a full-scale fire resistance test on a composite lattice joist with gypsum slab, which achieved a fire resistance period of 90 minutes with two layers of 15 mm thick fire resistant plasterboard.

10. COMPOSITE CONSTRUCTION USING LIGHT STEEL SECTIONS

Composite construction may be adapted to the use of cold-formed steel sections, and various possible applications are illustrated in Figure 11. Composite decking may be orientated vertically to form

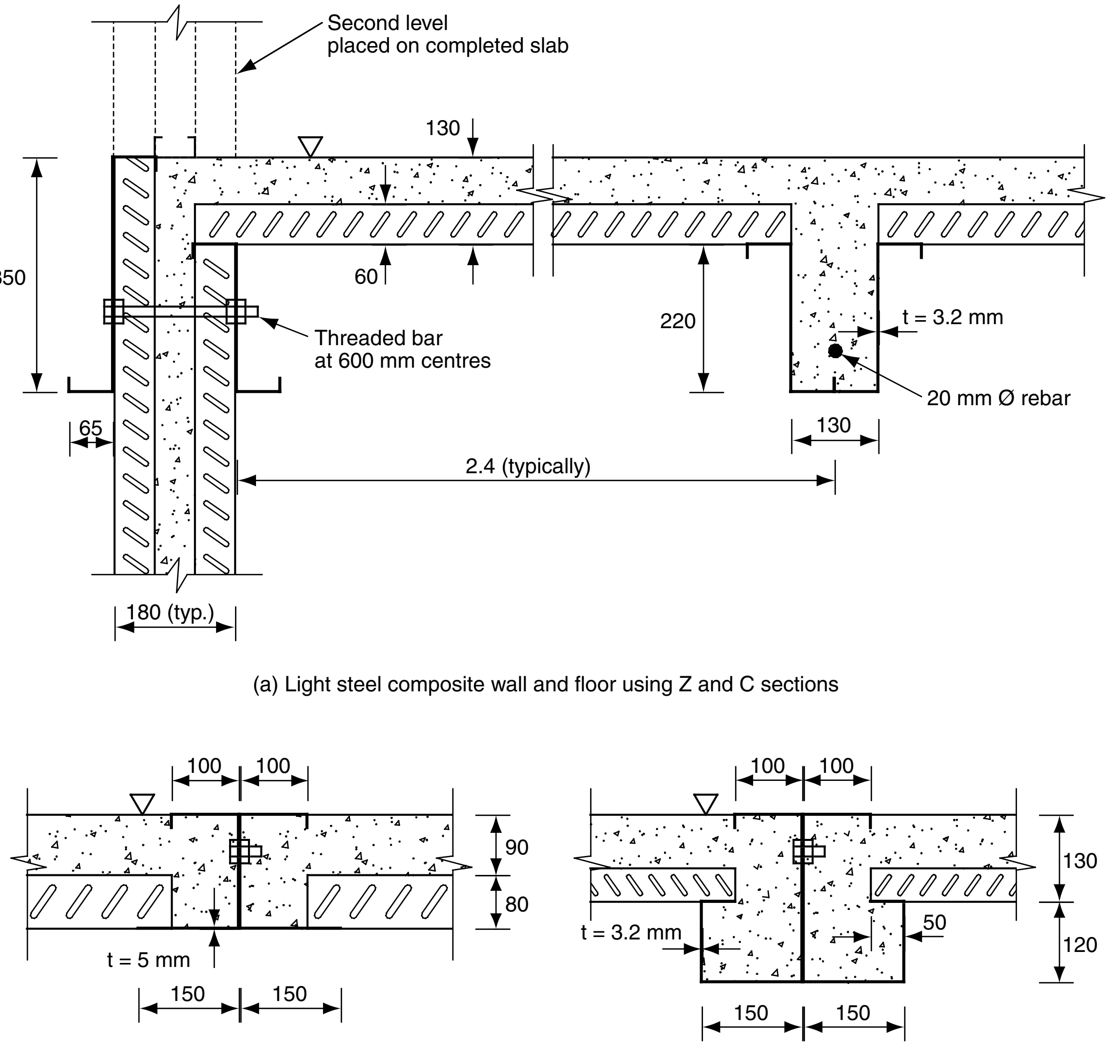

Figure 11. Examples of light steel composite sections using profiled steel decking and slim floor beams

compression members and shear walls. The composite decking resists the lateral pressures produced due to the concrete by tie rods at mid-height as well as at the top of the wall, and this system may be used for thin walls up to 3 m high.

Slim floor construction has attracted interests because the floor and the beam are integrated within the same depth. For moderate spans of 4 to 5 m, it is possible to use double asymmetric C sections of up to 5 mm thick which act compositely with the floor slab using simple shear connecting devices. These forms of construction are being investigated currently.

11. RECENT DEVELOPMENTS ON LIGHT STEEL STRUCTURES AND THEIR CONNECTIONS

It is worthy to note that a number of research studies (Chung and Lawson 2000; Ho and Chung 2004) on bolted connections in cold-formed steel structures may be found in the literature, and the studies aim to promote the effective use of cold-formed steel structures through the provision of rational design of both shear and moment connections in C and Z sections. For more recent researches on cold-formed steel portal frames and deployable structures, refer to Chung *et al*. 2008, Darcy and Mahendran 2008, Kwon *et al*. 2006, 2008, Liew *et al*. 2008, and Vu *et al*. 2006.

12. CONCLUSIONS

New forms of light steel-timber composite floor joists and roof trusses have been investigated and tested. The research activities and their findings are summarized as follows:

i) A double web box-shaped 'plyweb' joist with ballistically nailed plywood to its C section flanges was shown to have optimum characteristics and minimum shear deformation. Based on this test, it was found that the effective flexural rigidity of the composite floor joist may be calculated as 70% of the flexural rigidity of an equivalent I beam, ignoring the web, which takes account of slip in the fixings. More work is required to develop definitive design recommendations taking account of the types of boards and their fixings.

ii) A 'plyweb' roof truss of 8 m span was tested and the maximum test load exceeded the design imposed load by a factor of 3. Assuming that the bottom chord of the roof truss resists all the applied loads and act as a floor beam, its deflection may be calculated from an effective span given by the span of the roof truss less the width of one plywood insert. The span to depth ratio of the floor beam can be increased to 24 for most applications. This prototype roof truss is being developed further with regard to the most efficient method of manufacture.

iii) A test on 5 m span composite lattice joists using a 50 mm thick gypsum screed demonstrated a 180% increase in the stiffness of the joists and a 120% increase in the moment resistance due to the composite action developed between the joists and the gypsum screed. The failure mode was 'ductile', which demonstrated robust composite action due to the screw fixings through the 16 mm deep profiled steel decking to the joists. The shear-bond strength between the gypsum screed and the shallow profiled steel decking was measured as 0.13 N/mm^2 from small-scale bending tests. This composite flooring system possesses excellent flexural rigidity for reduced vibration sensitivity.

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