

Composite Action between Cold-Formed Steel Beams and Wood-Based Floorboards

Pinelopi Kyvelou

*Imperial College London, London, UK
pinelopi.kyvelou11@imperial.ac.uk*

Leroy Gardner

*Imperial College London, London, UK
leroy.gardner@imperial.ac.uk*

David A. Nethercot

*Imperial College London, London, UK
d.nethercot@imperial.ac.uk*

Received

Accepted

An experimental investigation has been conducted into the degree of shear interaction between cold-formed steel floor joists and wood-based flooring panels and the resulting benefits derived in terms of composite action. A series of four-point bending tests have been carried out to evaluate the overall system behaviour, while material tests have been performed to accurately define the material properties of each component of the examined system. Two different shear transfer mechanisms were examined: self-drilling screws with varying spacing and structural adhesive. The bare system was also tested to provide a reference response, against which the stiffness and capacity of the composite system could be bench-marked. The experiments showed that significant benefit could be derived as a result of composite action with as much as a 100% increase in bending capacity and 42% increase in stiffness.

Keywords: flooring systems, composite action, cold-formed steel, wood-based floorboard, degree of interaction, shear transfer mechanism.

1. Introduction

Cold-formed steel joists are an attractive option for the construction of flooring systems in commercial and residential buildings due to their high strength-to-weight ratio, ease of prefabrication and rapid assembly on site. The interaction between the steel floor joists and the wooden floor boards, which can have a significant effect on the strength and stiffness of the flooring system, is currently not systematically taken into consideration in design. Although the benefits of composite construction for conventional steelwork and concrete floor slabs are well known¹⁻⁴, this effect has yet to be exploited for cold-formed steel and its associated flooring. Experimental programs have been conducted in the past in order to investigate the possible development of composite action between light gauge steel beams and concrete slabs⁵⁻⁸ but no research appears to have been conducted on the

composite behaviour of flooring systems consisting of wood-based floorboards and cold-formed steel beams.

The aim of this project was firstly to examine the degree of composite action that arises within a typical floor system assembled using current practice. Hence, a system consisting of cold-formed steel swage beam (SB) joists and wood-based particle floorboards was tested. Following this, three similar systems, but with different degrees of shear connection, achieved with screws and/or structural adhesive were tested. The degree of composite action was measured, and the resulting benefits, in terms of strength and stiffness, were evaluated. Comparative tests on bare steel sections were also conducted.

2. Material Testing

Samples cut from the cold-formed steel sections and the wood-based particle boards were tested in order to accurately define the material properties of the examined structural system.

2.1. Tensile coupon tests

A series of tensile coupon tests were carried out. One coupon was cut from the coil prior to the forming of the sections while four flat and two corner coupons were cut from the cold-formed steel sections, at the locations shown in Figure 1. The coupons were then tested to determine the basic material stress-strain behaviour of the sections. Each coupon was labelled such that the coupon thickness and the position from which the coupon was extracted could be identified from its label. For instance, C15-4 refers to the coupon with a thickness of 1.5 mm and cut from location 4 (see Figure 1).

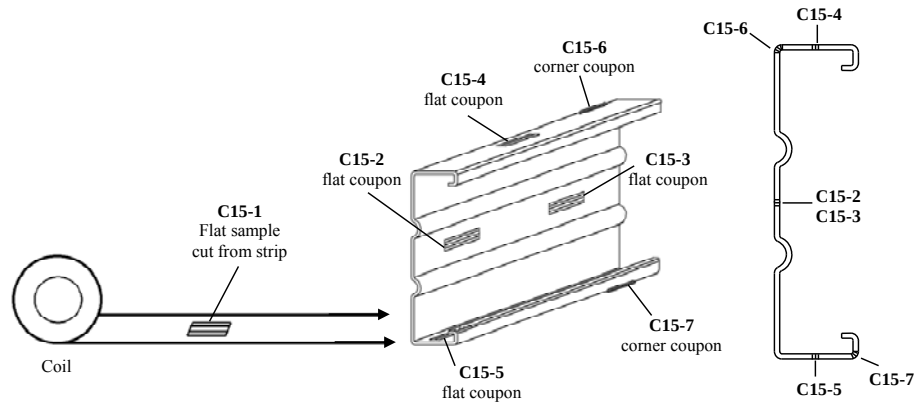


Fig. 1. Positions of extracted tensile coupons.

Two strain gauges were attached to each coupon, one on either side. The tests were conducted using a 25 kN Instron 8801 tensile testing machine and both the axial load and the strain readings were recorded using the Bluehill data acquisition system. Additionally, an optical extensometer was used over a length of 50 mm to provide accurate strain recordings for large strains. In line with BS EN ISO 6892-1⁹, the axial load was applied at a strain rate of 0.00007 s^{-1} prior to the 0.2% proof stress while a strain

rate of 0.00025 s^{-1} was employed beyond the 0.2% proof stress and until failure. A gradual transition from the initial to the final strain rate was ensured by using three intermediate strain rates. The key results obtained are summarised in Table 1, where n and $n'_{0.2,1.0}$ are the exponents of the compound Ramberg-Osgood material model¹⁰. The strain at fracture ε_f was measured over the standard gauge length defined in BS EN ISO 6892-1⁹. The measured stress-strain curves, up to 1% strain, are shown in Figure 2.

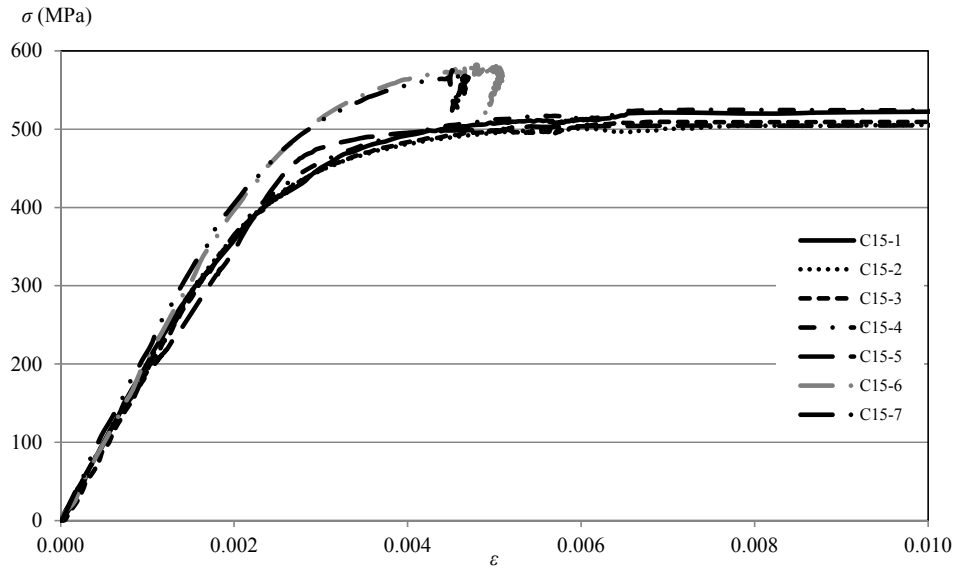


Fig. 2. Stress-strain curves obtained from coupon tests.

Table 1. Coupon test results.

Coupon ID	Elastic modulus E (GPa)	Yield strength $\sigma_{0.2}$ (MPa)	Tensile strength σ_u (MPa)	Strain at σ_u ε_u (%)	n	$n'_{0.2,1.0}$	Elongation at fracture ε_f (%)	Area reduction at fracture Z (%)
C15-1 (flat)	202	502	564	10.8	7.6	2.4	26.0	50.3
C15-2 (flat)	198	489	550	10.5	10.2	2.0	22.0	46.1
C15-3 (flat)	198	492	550	12.9	10.2	2.1	28.0	49.4
C15-4 (flat)	202	505	568	14.1	9.2	3.9	22.0	44.1
C15-5 (flat)	201	498	542	15.9	10.2	1.6	26.0	46.5
C15-6 (corner)	200	573	582	0.5	13.5	2.5	15.0	63.9
C15-7 (corner)	212	—	575	0.5	—	—	12.5	57.9

The average values of Young's modulus E and 0.2% proof strength $\sigma_{0.2}$ for the flat and corner material are shown in Table 2. Note that the corner strength is about 15% higher than that of the flat material; this strength enhancement is due to the plastic deformation arising during section forming¹¹.

Table 2. Material properties of cold-formed steel.

Average elastic modulus E (GPa)	202
Average flat yield strength $\sigma_{0.2}$ (MPa)	497
Corner yield strength $\sigma_{0.2}$ (MPa)	573
Average tensile strength σ_u (MPa)	562
Average value of n for the flat coupons	9.5
Average value of $n_{0.2,1.0}$ for the flat coupons	2.4

2.2. Testing of floorboard material

The basic material properties of the floorboards were determined by means of bending, tensile and compressive material tests, conducted in accordance with BS EN 789:2004¹². The bending specimens were subjected to 4-point bending, and loaded at a constant rate of 5 mm/min to failure using a 750 kN Mayes hydraulic actuator. The tensile and compressive tests were carried out in a 600 kN Instron loading frame at an applied axial load rate of 0.25 mm/min for the former tests and a rate of 0.5 mm/min for the latter tests. In all tests failure occurred within 4-6 minutes, as dictated by BS EN 789:2004. The averaged obtained material properties from three repeated tests are listed in Table 3.

Table 3. Averaged material properties of wood-based floorboards.

Elastic modulus in bending E_{mb} (GPa)	4.1
Elastic modulus in compression E_{cb} (GPa)	2.3
Elastic modulus in tension E_{tb} (GPa)	2.1
Bending strength f_{mb} (MPa)	12.9
Compressive strength f_{cb} (MPa)	12.9
Tensile strength f_{tb} (MPa)	5.8

It may be observed that the obtained elastic modulus in bending E_{mb} is significantly higher than the elastic modulus in compression E_{cb} and tension E_{tb} . This can be explained with reference to the manufacturing process of the floorboards, which results in stratified material properties with more dense, and hence more stiff, material towards the external surfaces of the floorboard and less dense material towards the middle. While the influence of the material density is averaged out by area in the tensile and compressive coupon test results, the denser material, being at the extreme fibres of the boards, has a disproportionately beneficial influence on stiffness in the bending configuration.

3. Composite Beam Tests

3.1. Test specimens

Four 4-point bending tests were conducted to assess the strength and stiffness of flooring systems with different degrees of composite action achieved through different shear transfer mechanisms between the cold-formed steel sections and the wooden floorboards.

A further 4-point bending test of the steel alone was conducted to provide a reference response, against which the composite tests could be assessed.

The shape and average measured dimensions of the tested swage beam sections are presented in Figure 3.

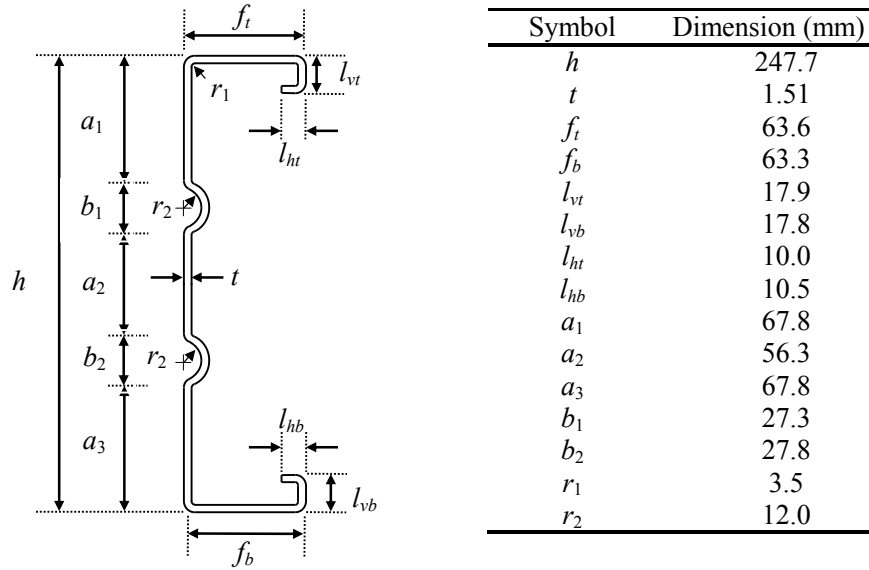


Fig. 3. Dimensions of tested swage beams.

As shown in Figure 4, each composite test consisted of 38 mm thick and 1200 mm wide wood-based floorboards connected to the top flange of two cold-formed swage beam joists by means of self-drilling screws. The span for all the beam tests was 5800 mm, with the beams extending 100 mm beyond the centerlines of supports at each end.

The bare steel section beam was designated as specimen No. 1, as shown in Table 4. Lateral and torsional restraint was provided to the sections by means of 50×50×2 mm steel angles, which were screwed to the top flanges of the steel sections at 150 mm intervals, as shown in Figure 5.

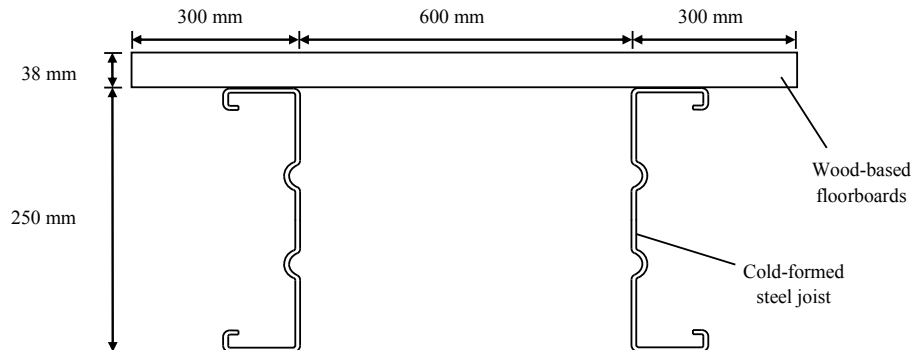


Fig. 4. Typical cross-section of composite specimens.

For the first composite beam test, designated as specimen No. 2, the spacing of the self-drilling screws was 600 mm while for specimen No. 3 the screw spacing was decreased to 150 mm. For specimen No. 4, the spacing of the shear connectors was set to 150 mm and wood adhesive was applied to the board joints in order to minimize the effect of any existing gaps, due to imperfect mating between the boards, as shown in Figure 6. Finally, the highest degree of shear connection was sought for specimen No. 5 by reducing the screw spacing to 100 mm, applying wood adhesive at the board joints and applying strong structural adhesive - epoxy resin - at the interface between the floorboards and the top flanges of the cold-formed steel joists.

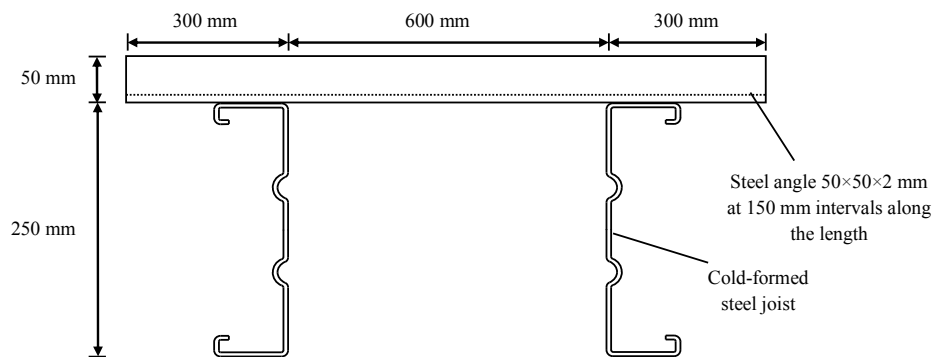


Fig. 5. Typical cross-section of bare steel beam test.

Table 4. Details of composite beam tests.

Specimen No.	Type of shear connection	Screw spacing
1	Bare steel only	—
2	Self-drilling screws	600 mm
3	Self-drilling screws	150 mm
4	Self-drilling screws, wood adhesive at board joints	150 mm
5	Self-drilling screws, wood adhesive at board joints, epoxy resin at the beam-board interface	100 mm

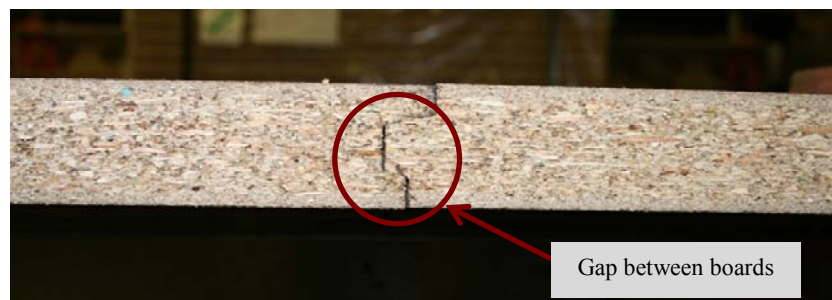


Fig. 6. Gap between boards due to imperfect mating.

3.2. Experimental layout

The overall experimental setup is depicted in Figure 7. All specimens were simply supported on rollers and loaded at their third points through a spreader beam which was hinged at its midpoint. The load was applied through an Instron hydraulic actuator, which had a maximum stroke of 150 mm and a maximum load capacity of 250 kN.

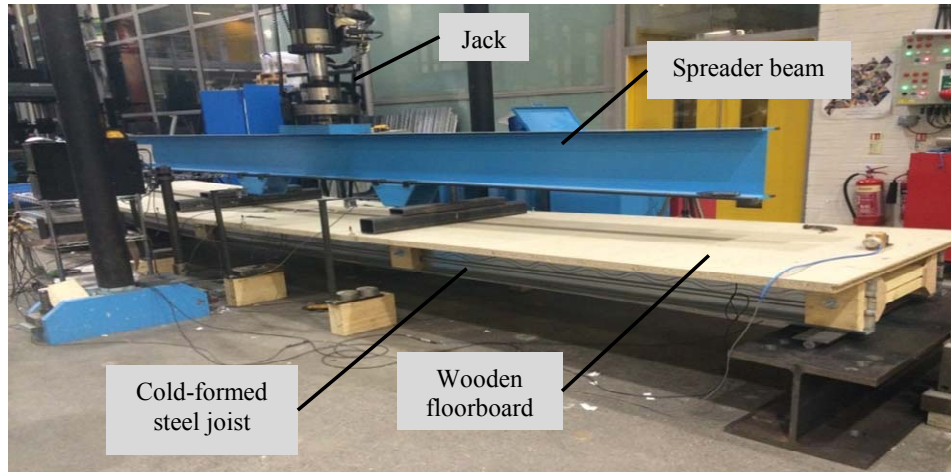


Fig. 7. Experimental setup for composite 4-point bending tests.

Four linear variable displacement transducers (LVDT) were employed to measure the maximum slip between the floorboards and the steel joists; these were positioned at each end of the two beams. Three string potentiometers (SP) were installed on each beam, one at midspan and two at the positions of the point loads, to measure vertical deflections, and hence determine curvature. Furthermore, two inclinometers were placed at the ends of the specimens to measure the developed rotations while one extra LVDT was used to measure the closing of the gap between two adjacent boards in the central uniform moment region. The employed instrumentation is presented in Figure 8.

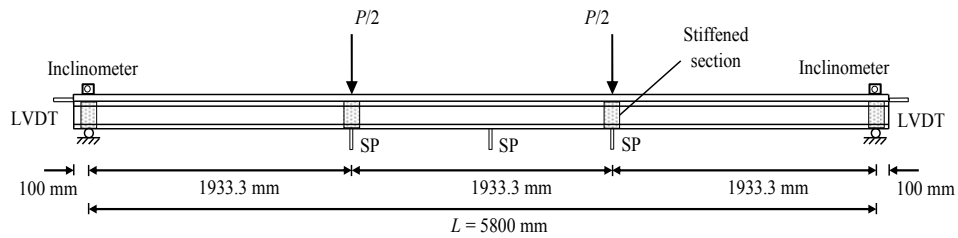


Fig. 8. Experimental layout for composite 4-point bending tests with associated instrumentation.

Finally, as shown in Figure 9, five strain gauges were affixed to the steel section at midspan with two further strain gauges attached to the top and bottom fibres of the board in order to record the strain distribution throughout the testing procedure.

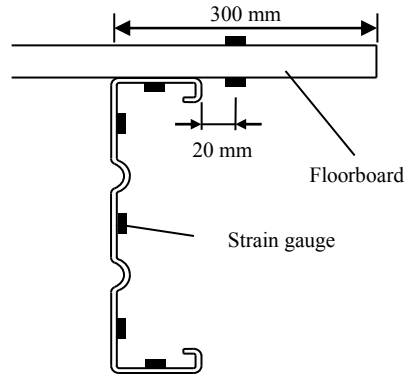


Fig. 9. Positions of strain gauges.

At the positions of the point loads and the supports, both beam sections were stiffened by being connected back-to-back with an additional 150 mm length of the same swage beam section and, as shown in Figure 10, by inclusion of timber blocks also 150 mm in length and a wooden diaphragm, all tied by a threaded bar.

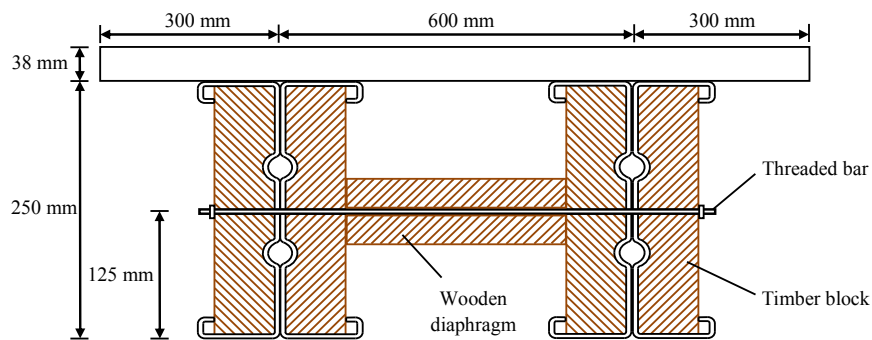


Fig. 10. Stiffened cross-section at positions of supports and point loads.

During each test, two load cycles of 3 kN were initially applied to the specimens to ensure correct function of the instrumentation and settling in of both beams. After these two cycles, the system was loaded until failure using displacement control with a displacement rate of 2 mm/min. The load, slip, deflections and strain readings were recorded at 1 s intervals using the data scan acquisition system DATASCAN.

3.3. Results

Ultimately, as shown in Figure 11, the tested beams all failed in a similar manner, exhibiting distortional buckling between fixings in the constant moment area. The only exception was specimen No. 5 where local buckles developed in the web and top flange of the beam, usually adjacent to the joints between the boards, where very locally, the steel sections may have been carrying more load.



Fig. 11. Failure modes of 4-point bending specimens.

The key experimental results are listed in Table 5, where $P_{u,exp}$ is the maximum load applied to each beam, $M_{u,exp}$ is the maximum attained moment, $\delta_{u,exp}$ is the average (between the two parallel beams) vertical deflection at midspan at $P_{u,exp}$ and $s_{u,exp}$ is the average (between the two parallel beams) slip that occurred at the interface of the steel joist and the floorboard at failure. It should be noted that the self-weight of the steel joists, the floorboards, the steel angles as well as the weight of the loading tubes have been included in the calculation of $P_{u,exp}$ and $M_{u,exp}$.

According to the theory of elasticity, the midspan deflection of a simply supported beam loaded at its third points with two equal concentrated loads P' can be calculated according to:

$$\delta_{max} = \frac{23P' L^3}{648EI} \quad (1)$$

where P' is each concentrated load, L is the length of the beam between the supports, E is the elastic modulus and I is the stiffness of the composite section. Thus, the flexural stiffness $(EI)_{exp}$ of each specimen was determined as the average of the calculated values of stiffness from P - δ data sets up to $0.25P_{u,exp}$ (elastic range), according to Eq. (1).

Table 5. Test results of 4-point bending tests.

Specimen No.	$P_{u,exp}$ (kN)	$M_{u,exp}$ (kNm)	$(EI)_{exp}$ (Nm ²)	$\delta_{u,exp}$ (mm)	$s_{u,exp}$ (mm)
1	19.9	19.1	1.11×10^6	78.9	—
2	21.0	20.0	1.19×10^6	72.7	2.4
3	28.9	27.6	1.27×10^6	97.7	1.5
4	29.9	28.6	1.57×10^6	112.6	3.0
5	39.7	38.1	1.58×10^6	179.5	0.0

The load versus average midspan deflection curves of all tests are plotted in Figure 12. It can be observed that the greater the degree of shear connection, the higher the strength and stiffness of the flooring system.

Load P_{exp} (kN)

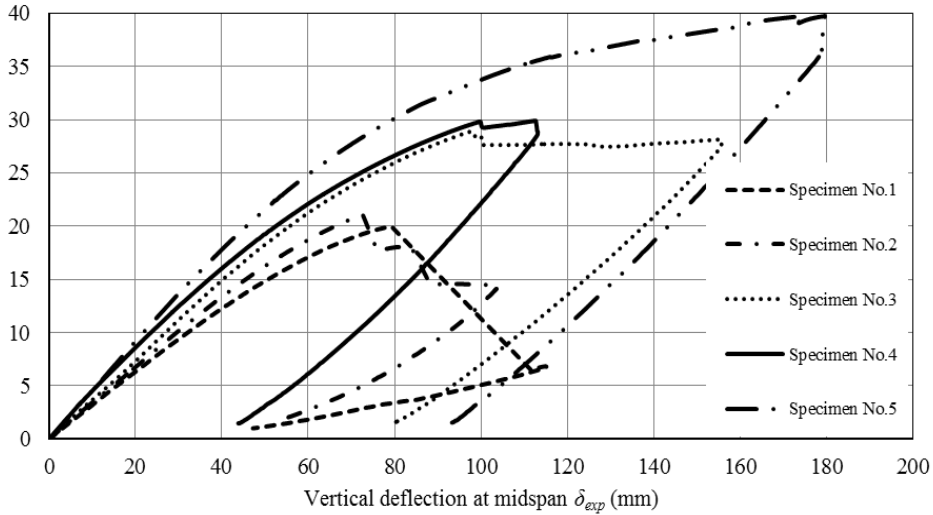


Fig. 12. Load-deflection curves of 4-point bending tests.

The readings of the strain gauges mounted on the midspan cross-section, as shown in Figure 9, showed that the position of the neutral axis lay, for all tests, within the steel section. However, with the increasing degree of shear interaction (from specimen No. 2 to specimen No. 5), the neutral axis moved upward towards the floorboard. Figure 13 presents the strain distributions measured at $0.5P_{u,exp}$ and $P_{u,exp}$ for all tests.

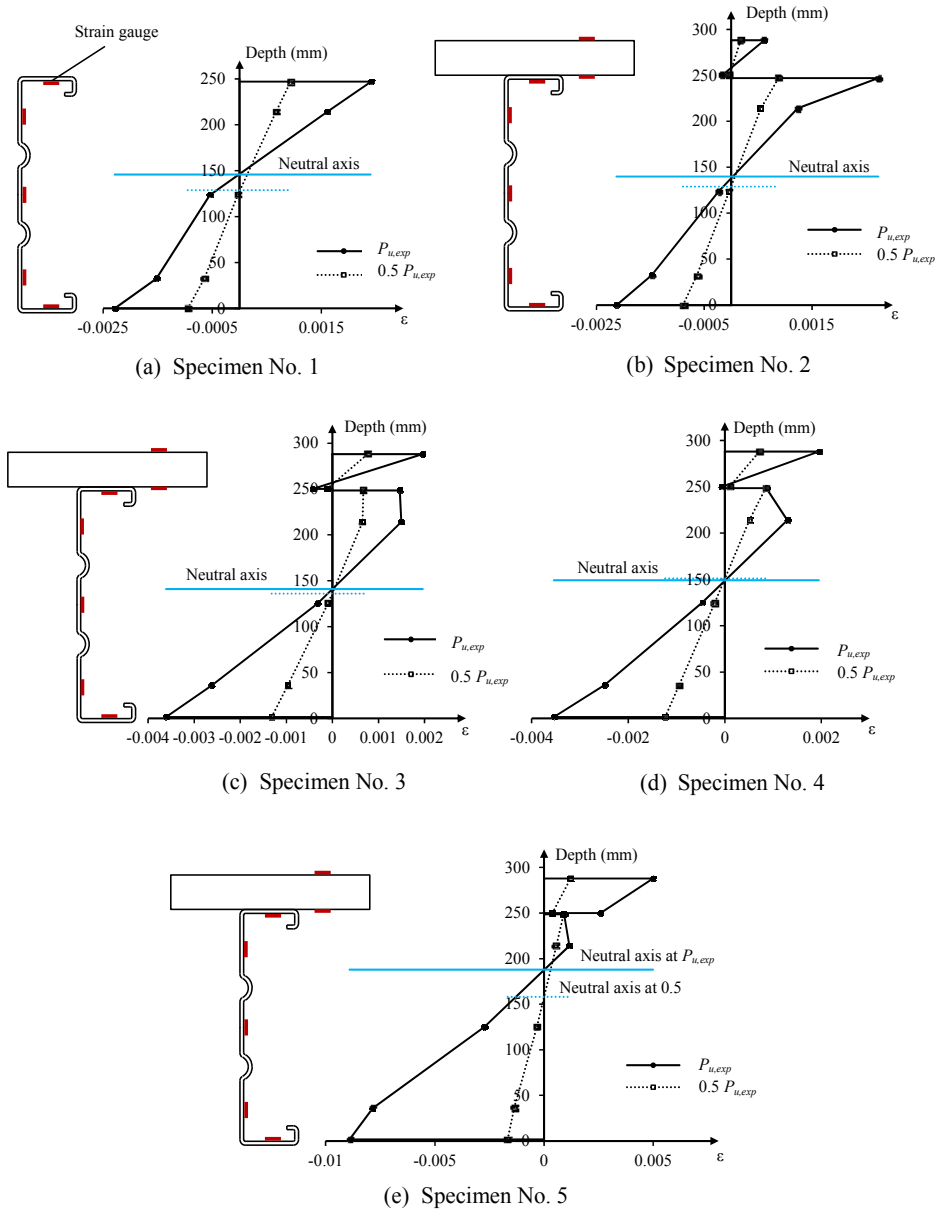


Fig. 13. Strain distributions at midspan sections for all tested beams.

As shown in Figure 13, for the majority of the cases, the neutral axis moved upwards under increasing loading. This is attributed to local buckling of the steel sections prompting a reduction in stiffness thereof, and closure of gaps between the floorboards, enabling them to become more fully engaged in the latest stages of the tests.

4. Analysis of Results

4.1. Theoretical plastic resistance and flexural stiffness assuming full interaction

Assuming full shear interaction between the cold-formed steel beam and the wooden floorboard and a plastic distribution of stresses, the corresponding theoretical plastic bending resistance of the composite section can be derived. Figure 14 depicts the case where the plastic neutral axis lies within the wooden floorboard while the case of the plastic neutral axis lying in the steel section is illustrated in Figure 15.

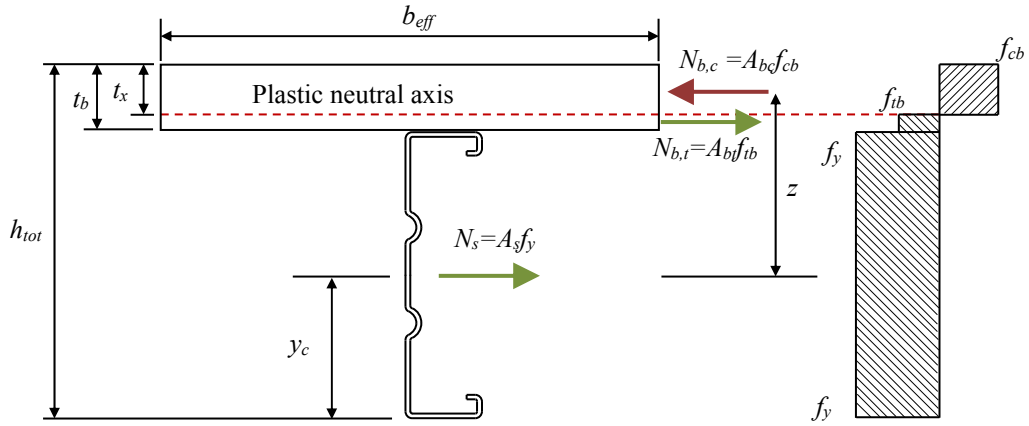


Fig. 14. Plastic stress distribution in a composite section with the neutral axis lying in the floorboard.

The location of the plastic neutral axis can be determined by considering force equilibrium in the cross-section. For the case depicted in Figure 13, the force equilibrium can be expressed as:

$$N_s + N_{b,t} = N_{b,c} \quad (2)$$

Hence, the portion of floorboard in compression of thickness t_x can be calculated according to:

$$A_s f_y + A_{bt} f_{tb} = A_{bc} f_{cb} = b_{eff} t_x f_{cb} \quad (3)$$

where A_s and f_y are the area and the yield (0.2% proof) strength of the steel section respectively, b_{eff} is the effective width of the floorboard (taken as 600 mm) and f_{cb} and f_{tb} are the compressive and tensile strengths of the floorboard respectively. The moment resistance of the composite section $M_{R,pl}$ can then be calculated from Eq. (4), based on the principles of plastic analysis according to Clause 6.2.1.2 of BS EN 1994-1-1¹³, and taking moments about the axis of the compressive force $N_{b,c}$.

$$M_{R,pl} = N_s z + N_{b,t} \frac{t_b}{2} \quad (4)$$

in which:

$$z = h_{tot} - t_x / 2 - y_c \quad (5)$$

where h_{tot} is the total height of the composite section and y_c is the distance from the bottom fibre of the bottom flange to the centroid of the steel section.

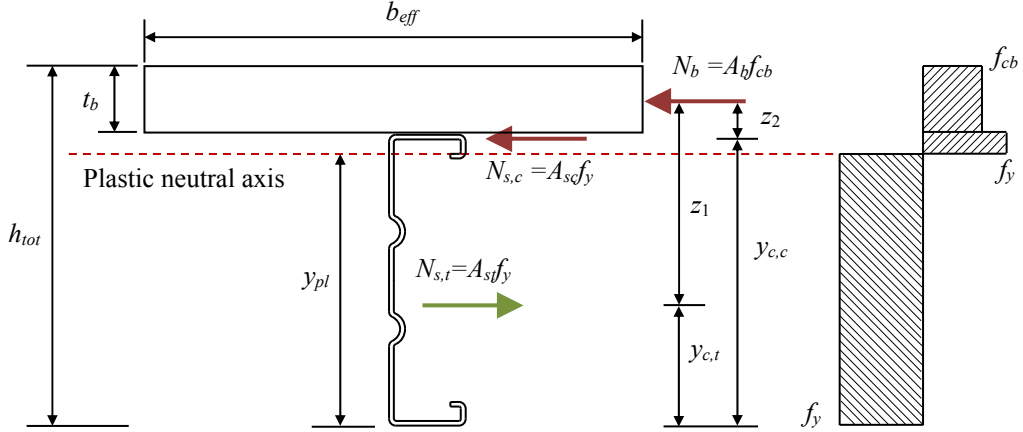


Fig. 15. Plastic stress distribution in a composite section with the neutral axis lying in the steel beam.

For the case illustrated in Figure 15, where the plastic neutral axis lies within the steel section and the top part of the steel section is in compression, equilibrium of forces in the cross-section is again used for the determination of the position of the neutral axis, hence:

$$A_{st} f_y = A_{sc} f_y + A_b f_{cb} \quad (6)$$

where A_{st} and A_{sc} are the areas of the steel section in tension and compression respectively and $A_b = b_{eff} t_b$ is the area of the floorboard. The moment resistance of the composite section $M_{R,pl}$ can be calculated from Eq. (7), by taking moments about the axis of the compressive force in the floorboard N_b to give:

$$M_{R,pl} = N_{s,t} z_1 - N_{s,c} z_2 \quad (7)$$

in which:

$$z_1 = h_{tot} - y_{c,t} - \frac{t_b}{2} \quad (8)$$

$$z_2 = h_{tot} - y_{c,c} - \frac{t_b}{2} \quad (9)$$

with other symbols as defined in Fig. 15.

For the determination of the stiffness of the composite beam I_{comp} , it is convenient to first transform the area of the wooden floorboard into an equivalent area of steel, as shown in Figure 16; I_{comp} is then calculated according to:

$$I_{comp} = I_{steel} + \frac{b_{eff} t_b^3}{12n} + A_s (y_{el} - y_c)^2 + \frac{A_b}{n} (y_{el} - h - t_b / 2)^2 \quad (10)$$

where y_{el} is the distance from the bottom fibre of the bottom flange to the centroid of the composite section determined from Eq. (11), y_c is the distance from the bottom fibre of the bottom flange to the centroid of the steel section, I_{steel} is the second moment of area of the steel section, n is the ratio of the elastic moduli of steel to board and A_s and A_b are the areas of the steel section and the floorboard respectively.

$$y_{el} = \frac{y_c A_s + \frac{A_b}{n} \left(h + \frac{t_b}{2} \right)}{A_s + \frac{A_b}{n}} \quad (11)$$

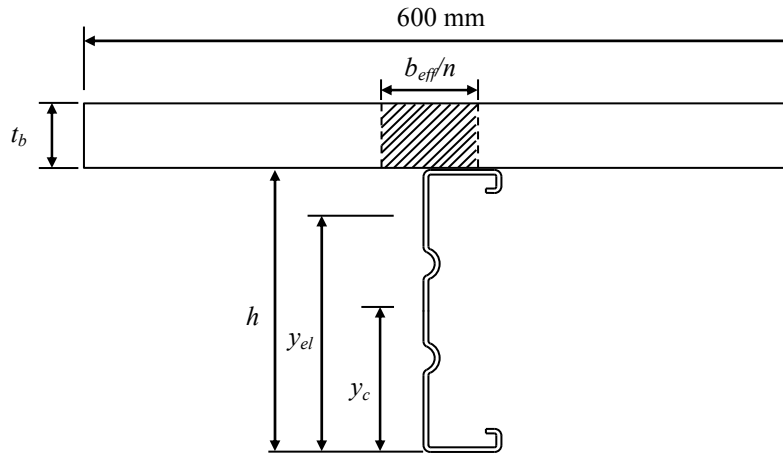


Fig. 16. Transformed section for determination of stiffness of composite beam.

For the composite flooring system examined for this project, the plastic neutral axis lies within the top flange of the steel section. Therefore, the plastic resistance of the fully composite section which was determined according to equation (7) is $M_{R,pl} = 44.9$ kNm while the second moment of area I_{comp} , determined according to equation (10), is $I_{comp} = 943 \times 10^4 \text{ mm}^4$.

4.2. Comparison of experimental results with bare steel and full composite resistances

The ultimate moment capacity $M_{u,exp}$ and flexural stiffness $(EI)_{exp}$ of each of the examined systems normalised by the ultimate capacity $M_{u,expSI}$ and flexural stiffness $(EI)_{expSI}$ of the bare steel system (specimen No. 1) are presented in Figure 17.

It can be observed that the flooring system assembled using current practice (i.e. fasteners at 600 mm spacing), represented by specimen No. 2, offers a minor increase in the strength and stiffness over the bare steel system. However, decreasing the spacing of

the fasteners (specimen No. 3) results in a significant increase in the system's moment capacity while, the application of wood adhesive at the joints between adjacent floorboards (specimen No. 4) yields a significant increase in terms of stiffness. Finally, as expected, the best performance is exhibited by specimen No. 5, with as much as 100% increase in moment capacity and 42% increase in stiffness.

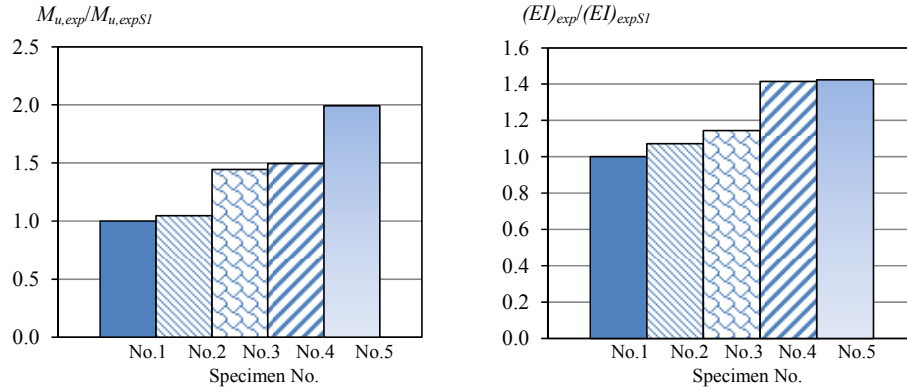


Fig. 17. Moment capacity and stiffness of each system normalised by the capacity and flexural stiffness of the bare steel system (specimen No. 1).

In the bar charts of Figure 18, the ultimate moment capacity $M_{u,exp}$ and flexural stiffness $(EI)_{exp}$ of each of the examined systems are normalised by the theoretical plastic moment capacity $M_{R,pl}$ and flexural stiffness $(EI)_{comp}$ of the fully composite system. Again, as expected, the behaviour of specimen No. 5 outperforms the others, attaining more than 80% of both the plastic moment capacity and stiffness of the fully composite system.

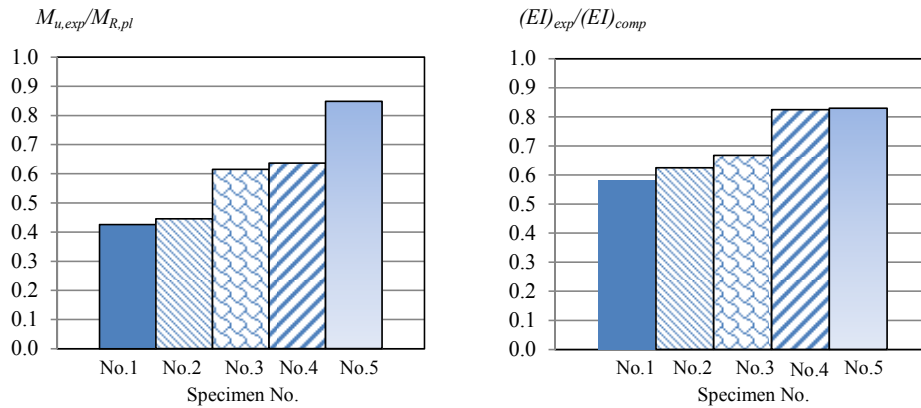


Fig. 18. Moment capacity and stiffness of each system normalised by the capacity and flexural stiffness of the fully composite system (assuming full shear connection).

4.3. Attained degree of shear connection

According to Clause 6.2.1.3(3) of BS EN 1994-1-1¹³, the degree of shear connection is defined by the ratio η , given by:

$$\eta = \frac{N_c}{N_{c,f}} \quad (12)$$

where N_c is the compressive force in the board of the examined system and $N_{c,f}$ is the compressive force in the board of an equivalent system with full shear connection, assuming a plastic distribution of stresses. The derivation of N_c was based on the stress distribution of each system at ultimate load derived from the strain distributions presented in Figure 13 while $N_{c,f}$ was calculated as described in Figure 15 (where it was labelled as N_b).

The achieved degree of shear connection for all specimens, derived according to the described procedure, is illustrated in Figure 19. In summary, the attained degree of shear connection η for specimen Nos 2, 3, 4 and 5 was 4%, 14%, 17% and 68% respectively, showing that decreasing the spacing of the fasteners and applying adhesive at the interface between the steel joist and the floorboards offer a significant enhancement to the shear connection, permitting the development of substantial composite action within the flooring system.

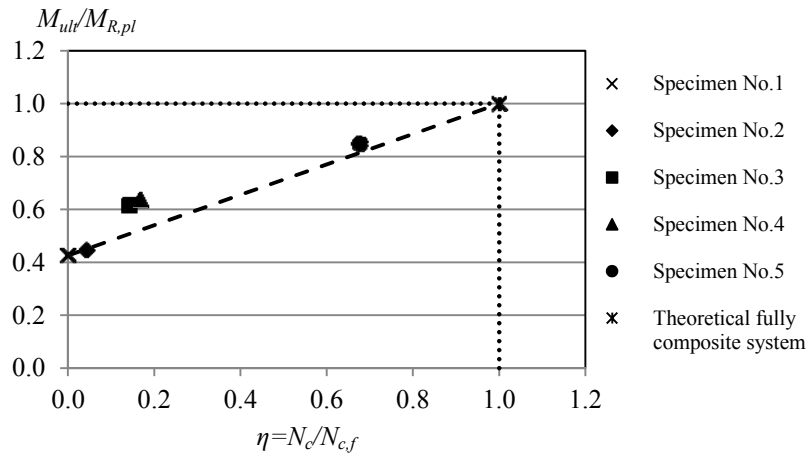


Fig. 19. Attained degree of shear connection

In order to implement the findings of this research, a predictive formula for the degree of shear connection η for each flooring and connection system should be established. This is the topic of ongoing research.

5. Conclusions

Five 4-point bending tests have been conducted to assess the degree of shear interaction within a typical floor system consisting of cold-formed steel joists and wood-based floorboards and the corresponding benefits derived from composite action.

It was found that the spacing of the fasteners as well as the application of adhesive at the interface between the cold-formed steel beams and the floorboards have a significant effect on the moment capacity of the flooring system while the elimination of gaps between adjacent floorboards results in increased stiffness.

Partial shear connection, up to 68% of full interaction, has been achieved and resulted in as much as 100% increase in bending capacity and 42% increase in stiffness. An expression for the determination of the degree of shear connection η for cold-formed steel flooring systems is sought in ongoing research.

Acknowledgments

The authors would like to thank Ayrshire Metal Products for their funding of the experimental programme as well as for their invaluable technical support, and Les Clark and Gordon Herbert for their work in the laboratory.

References

1. Lam D. and El-Lobody E., Behavior of Headed Stud Shear Connectors in Composite Beam, *Journal of Structural Engineering*, **131**(1) (2005) 96-107.
2. Lam D., Elliott K.S. and Nethercot D.A., Experiments on composite steel beams with precast concrete hollow core floor slabs, *Proceedings of the ICE - Structures and Buildings*, **40**(2) (2000) 127-138.
3. Chung K.F. and Wang A.J., Behaviour, analysis and design of composite beams and joints in buildings, *Proceedings of 5th International Conference on Advances in Steel Structures, ICASS 2007*, Singapore, **1** (2007) 99-116.
4. Gardner L., Macorini L. and Kucukler M., The continuous strength method for steel and composite design, *Proceedings of the Institution of Civil Engineers - Structures and Buildings*, **166**(8), (2013) 434-443.
5. Hanaor A., Tests of composite beams with cold-formed sections, *Journal of Constructional Steel Research*, **54** (2000) 245-264.
6. Lakkavalli B.S. and Liu Y., Experimental study of composite cold-formed steel C-section floor joists, *Journal of Constructional Steel Research*, **62** (2006) 995-1006.
7. Wehbe N., Bahmani P. and Wehbe A., Behavior of concrete/cold formed steel composite beams: Experimental development of a novel structural system, *International Journal of Concrete Structures and Materials*, **7** (2013) 51-59.
8. Hsu C.-T.T., Punurai S., Punurai W. and Majdi Y., New composite beams having cold-formed steel joists and concrete slab, *Engineering Structures*, **71** (2014) 187-200.
9. *BS EN ISO 6892-1:2009 Metallic materials – Tensile testing – Part 1: Method of test at room temperature*, 2009.
10. Gardner L. and Ashraf M., Structural design for non-linear metallic materials, *Engineering Structures*, **28**(6), (2006) 926-934.
11. Afshan S., Rossi B. and Gardner L., Strength enhancements in cold-formed structural sections – Part I: Material testing, *Journal of Constructional Steel Research*, **83** (2013) 177-188.
12. *BS EN 789:2004 Timber structures – Test methods – Determination of mechanical properties of wood based panels*, 2004.
13. *BS EN 1994-1-1:2004 Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings*, 2004.