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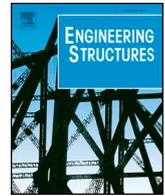
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Elastic behaviour of a tapered steel-concrete composite beam optimized for reuse

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ABSTRACT

Steel-concrete composite beams are widely used in practice because of their simple construction sequence and economic cross-section design. Reuse of traditional composite beams is not possible due to the permanent connection between the steel beam and concrete deck established by welded headed studs. To allow for fast construction, demountability and reuse of composite beams, various demountable shear connectors can be used. In this paper the results of experiments carried out on demountable and reusable tapered composite beams, consisting of a tapered steel beam and large-scale prefabricated concrete decks, are presented. The performance of various arrangements of resin-injected bolt-coupler shear connectors was considered to optimize the beneficial effect of composite action whilst minimizing the number of shear connectors. An advantage of resin-injected bolted shear connectors is that composite action is obtained instantaneously and simultaneously for all connectors. Demountability and reusability of the composite beam were successfully demonstrated experimentally. Experimental and numerical results indicated that the number of shear connectors necessary to fulfil deflection and end-slip limits can be reduced by concentrating them near the supports of a simply-supported beam. Results obtained using finite element models closely matched the experimental results in terms of deflection, stresses and curvature.

1. Introduction

Competitive cross-section design and an efficient construction sequence are the main characteristics of steel-concrete composite beams used as flooring systems in buildings. The composite interaction between the steel section and concrete deck is achieved by shear connectors. Welded headed studs have been used for this purpose for decades, and therefore the design rules related to steel-concrete composite beams are based on this type of connectors.

A downside of welded headed studs is that they do not allow for non-destructive separation of the steel beam and concrete deck during the demolition phase of the structure. The main reason for demolition is obsolescence, i.e. the structure is no longer regarded as useful or significant (Burns [1]). As for many other (commercial) products, reasons for obsolescence of buildings are mainly of aesthetic, social, technological, economic, logistical and functional nature (Burns [1], Cooper [2], Feldman & Sandborn [3]).

Within the circular economy framework, the construction and demolition waste, which is related to the obsolescence of structures, is mitigated or even prevented by maintaining the highest possible value of products and materials in the economic system – a concept closely related to the Inertia Principle of Stahel [4,5]. The use of welded headed studs does not fit in a circular economy framework, because the economic value of the structure is significantly decreased (or even negative) due to destructive demolition. Demountable and reusable

composite structures, however, offer the opportunity to maintain the highest economic value during their complete technical lifetime.

Demountability and reusability of composite structures can be achieved by using demountable shear connectors instead of welded headed studs. A great variety of demountable shear connectors exists, ranging from headed studs that are bolted rather than welded (e.g. Lam & Saveri [6]), (pretensioned) bolted shear connectors (e.g. Liu et al. [7], Pavlovic [8]), embedded bolt-coupler systems with an external bolt (Nijgh et al. [9], Kozma et al. [10], Yang et al. [11]), to more complex alternatives (e.g. Suwaed & Karavasilis [12,13]).

The focus of this paper is on demountable and reusable composite beams consisting of tapered steel beams and large prefabricated concrete decks. The main challenge is to account for the tolerances originating from the fabrication process of the steel beam and prefabricated concrete decks. The size of the prefabricated concrete decks should be as big as practically possible, as this reduces the amount of work at the construction site and therefore improves the execution efficiency. In addition to fabrication tolerances, execution tolerances are necessary to allow for fast execution of the structure. The fabrication and execution tolerances can be accounted for by opting for an oversize bolt hole, in case of a bolted shear connector. However, an increased bolt-to-hole clearance leads to a decrease in effective shear connector stiffness [11]. Injecting the remaining bolt-to-hole clearance with an epoxy resin has the potential to mitigate the decrease in effective shear connector stiffness in composite beams (Nijgh [14]). The stiffness of the epoxy

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resin can be increased by reinforcing it using steel particles [14]. An overview of the mechanical properties of the most commonly used (steel-reinforced) epoxy resin can be found in the work of Nijgh et al. [15] and Xin et al. [16].

Tests on demountable and reusable prismatic composite beams have been carried out by various researchers. For example, Lam et al. [17] and Moynihan & Allwood [18] performed beam tests on composite beams with bolted headed studs, and Wang et al. [19] performed tests on composite beams with (pretensioned) clamping connectors. In addition, Pathirana et al. [20] carried out beam tests with blind bolts as shear connectors. In all previously mentioned beam tests, the focus is on the behaviour at the ultimate limit state (resistance) rather than on the serviceability limit state (deflection and elastic deformations). It could be expected that the serviceability limit state gains more importance in the design of a reusable structure than the ultimate limit state. The importance of the serviceability limit state could be expressed by introducing an increased partial safety factor to ensure the probability of reuse is sufficiently high. Clearly, this concept is to be developed in more detail when the circular economy framework in the construction sector is further implemented.

Tapered composite beams have structural and functional advantages compared to prismatic composite beams (Nijgh [21]). Further structural advantage can be gained by concentrating shear connectors near the supports of a simply supported composite beam, because this reduces the deflection at midspan and the slip at the supports (Roberts [22], Lin et al. [23]). Research of Zona & Ranzi [24] indicates that concentrating connectors near the supports is also effective in limiting the slip demand at ultimate state. Lin et al. [23] have adopted the particle swarm optimization technique to optimize the location of the shear connectors for a given prismatic composite beam design.

According to Eurocode 4 [25], the longitudinal shear connector spacing may not exceed 6 times the slab thickness nor the value of 800 mm. However, a larger spacing is permitted in case of grouped shear connectors. In that case, the non-uniform shear flow, vertical separation, local buckling of the steel flange, and the local resistance of the concrete slab must be taken into account in the design. Nairane [26] found that vertical separation is non-existent under elastic conditions for composite beams subject to uniformly distributed loads, indicating that the shear connector spacing requirements of Eurocode 4 may not be relevant for elastically designed reusable composite beams. For concentrated forces, however, it was found that vertical separation is negligible in the elastic range, but that it becomes more pronounced as the composite beam exhibits plastic deformation. The latter, however, is by definition avoided if the composite beam is designed elastically. The considerations above imply that, in principle, no vertical restraints are necessary, opening up the possibilities for an economically more viable design.

2. Methodology

2.1. Experimental programme

Experiments were conducted on a tapered steel-concrete composite beam to investigate its demountability and reusability and its elastic behaviour in a four point bending setup. The justification for the limitation to elastic behaviour is that the focus is on demountable and reusable structures, and therefore plasticity in the beam, deck and shear connectors should not be allowed in the design.

The dimensions of the experimental composite beam replicate typical dimensions of a multi-storey car park building (ArcelorMittal [27]) at 90% scale, see Fig. 1. The simply-supported composite beam consists of two prefabricated solid concrete decks of $7.2 \times 2.6 \times 0.12$ m, connected to two symmetrically tapered steel beams using demountable shear connectors. The composite beam spans 14.4 m, and is subjected to bending by applying live loads at 4.05 m from both supports. The centre-to-centre distance of the steel beams is 2.6 m. An

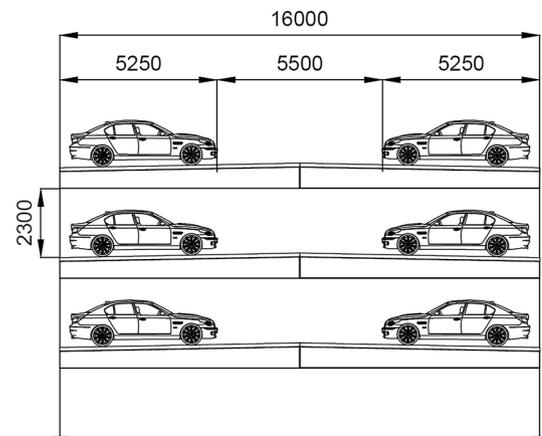


Fig. 1. Typical dimensions (mm) of a multi-storey car park building for one-way traffic circulation [27].

overview of the experimental set-up is provided in Fig. 2a.

The height of the symmetrically tapered steel beam of grade S355 varies linearly between the supports, $h_s(x=0; x=L) = 590$ mm, and midspan, $h_s(x=L/2) = 725$ mm. The thickness and width of the flanges, as well as the thickness of the web, are constant along the beam length. The magnitudes of these geometrical parameters are provided in Fig. 3.

The nominal strength class of the solid concrete decks is C30/37. Angle profiles ($120 \times 120 \times 10$ mm, S355) are placed around the bottom perimeter of the prefabricated concrete decks, to prevent damage to the concrete during the transportation and the assembling and demounting process of the beam, and to act as formwork during casting. In addition, the angle profiles transfer the normal force in the midspan joint without damage to the concrete corners – a phenomenon previously observed by Wang et al. [19]. Fig. 2b illustrates the prefabricated deck prior to casting, showing the reinforcement mats (#8–150 mm with 25 mm cover) and the demountable shear connector system.

The demountable shear connectors consist of an M20 coupler (grade 10.9, DIN 6334) and M20 bolt (grade 8.8, ISO 4017) embedded in the concrete deck, which are connected to the steel beam flange with an external M20 (grade 8.8, ISO 4017/EN 1090-2) injection bolt – see Fig. 4a. The concept behind the over-strength coupler is that damage related to the overloading of a shear connector accumulates in the external bolt, rather than in the embedded coupler, hereby ensuring that the concrete deck is fit-for-use in a subsequent life cycle. The holes in the beam flange have a diameter of 32 mm and are therefore significantly oversized. Through the oversize holes, fast execution of the composite flooring system is possible, despite the fabrication and execution tolerances. The remaining clearance between bolt and bolt hole is injected with RenGel SW 404 + HY2404/HY5159 epoxy resin through the injection channel in the injection bolt.

The load-slip characteristic of the shear connector system (see Fig. 4b) is obtained by experimental and numerical companion push-out tests carried out using bolts, couplers and concrete of the same batch as for the prefabricated decks. Material and damage models from Pavlović [8] and Xin et al. [16] are used to predict the load-slip curve of the shear connector system. Validation of the finite element model is carried out by comparing the experimental push-out test results to the numerical results. Very good agreement was found between the experimentally and numerically obtained load-slip curves, based on which the (initial) shear connector stiffness k_{sc} is determined as 55 kN/mm. The maximum allowable connector slip during the experiments was set to 1 mm to ensure (quasi) linear-elastic behaviour of the shear connector system. Extensive details of the push-out tests and the numerical model thereof are scheduled to be published separately.

The opportunity to install external (injection) bolts into the

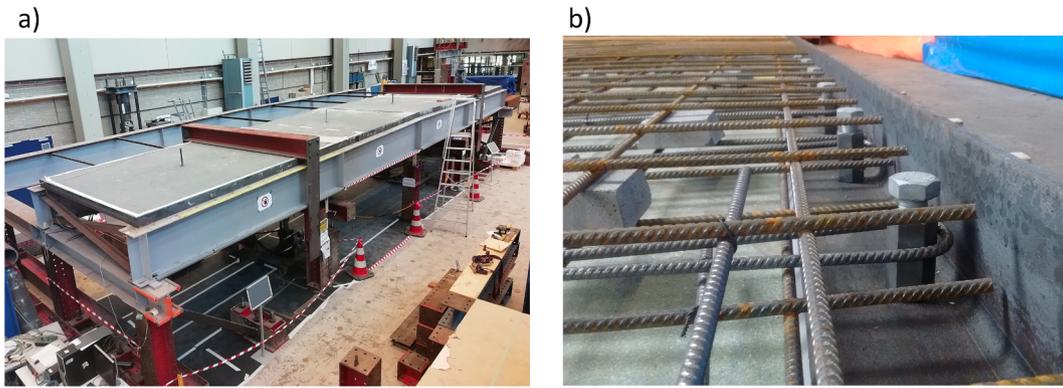


Fig. 2. (a) Overview of experimental set-up, (b) One of the prefabricated decks prior to casting, indicating the reinforcement (two mats of #8–150 mm with 25 mm cover) and embedded couplers and bolts.

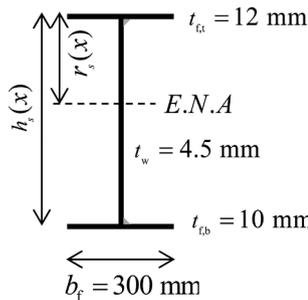


Fig. 3. Cross-section of the tapered steel beams of grade S355, welded at one side of the web. The web and top flange are both cross-section class 4.

embedded coupler exists at a centre-to-centre distance of 300 mm along the beam span, hereby allowing for the investigation of the effects of different shear connector arrangements on deflection and end-slip. The hypothesis is that fewer shear connectors are sufficient to fulfil deflection and end-slip criteria if they are concentrated near the supports rather than uniformly distributed along the beam length. An overview of the shear connector arrangements considered in the experimental programme is provided in Fig. 5. For the arrangements in which the shear connectors are installed near the supports, additional bolts are installed to prevent vertical separation of the steel beam and concrete deck – although this separation is unlikely to occur (Nairane [26]). These additional bolts do not act as shear connectors, because the remaining bolt-to-hole clearance was intentionally not injected and therefore no significant shear force could be transferred by these bolts at the load levels imposed during the tests.

The end-slip between the steel beam and the concrete deck is

measured at the supports by four ETI SYSTEMS LCP8 potentiometers at the steel-concrete interface. The deflection is measured at mid-span and near the point of load application with six Sakae S13FLP50A potentiometers. TML FLA-6-11 strain gauges have been installed to monitor the stresses in the beam and to determine the beam curvature. The strains are measured at the outer tensile fibre of the steel beam, as well as in the web close to both flanges. The strain gauges were installed at 5.0 m from the supports, because the maximum longitudinal stresses resulting from self-weight and applied load were expected in this cross-section.

The detailing of the composite beam at mid-span and at the supports is provided in Fig. 6. Normal forces can, but bending moments cannot be transferred through the deck-to-deck joint at mid-span. Given that the bending stiffness of the concrete deck is an order of magnitude lower than that of the steel beam, it can be derived that the effects of this detailing on the deflection are insignificant.

To prevent lateral torsional buckling and torsional deformations during the assembly and experiments, horizontal and diagonal braces were installed between the two steel beams to ensure uniaxial bending of the steel beams. A total of five bracing sets were installed, one at each support and three along the beam span.

The loads are applied by controlling the stroke of the jacks, which are connected to a loading frame, see Fig. 6. The loading and unloading speeds were set as 0.15 mm/s and 0.30 mm/s, respectively. The composite beam was loaded and unloaded five times during each of the experiments to check the consistency of the results.

2.2. Finite element model

The steel-concrete composite beam is modelled using the

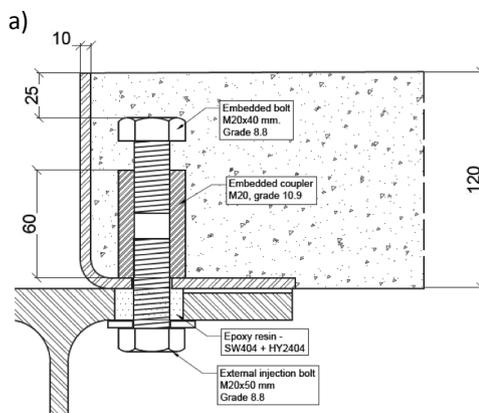
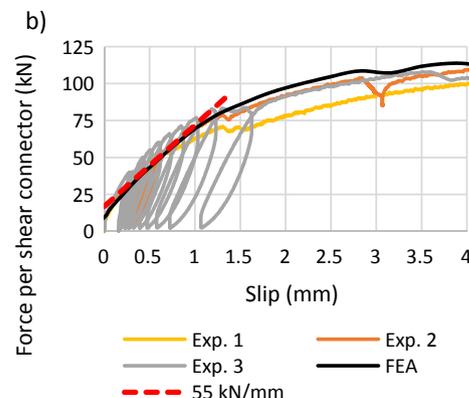


Fig. 4. (a) Cross-section of demountable shear connector, concrete strength class C30/37 (dimensions in mm). (b) Numerically and experimentally established load-slip curves obtained by three push-out tests on the demountable shear connector system, including linear-elastic approximation for the shear connector stiffness.



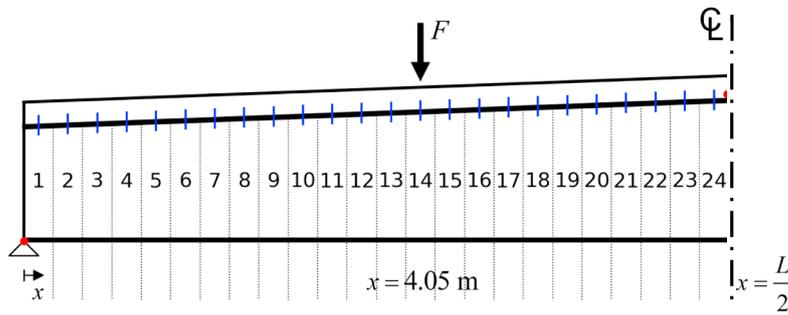


Fig. 5. Shear connector arrangements. Each coloured box indicates a pair of fasteners (one per steel beam). Resin-injected bolts provide shear connection, whereas normal bolts are placed only to prevent vertical separation of deck and beam. “U” denotes uniform connector spacing, “C” denotes concentrated connector spacing near the supports. Beam is symmetric in the plane at $x = L/2$.

Arrangement	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
U-24	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
C-12	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
C-6	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
U-12	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
U-6	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
U-0	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■

■ Resin-injected bolt ■ Bolt (no shear interaction)

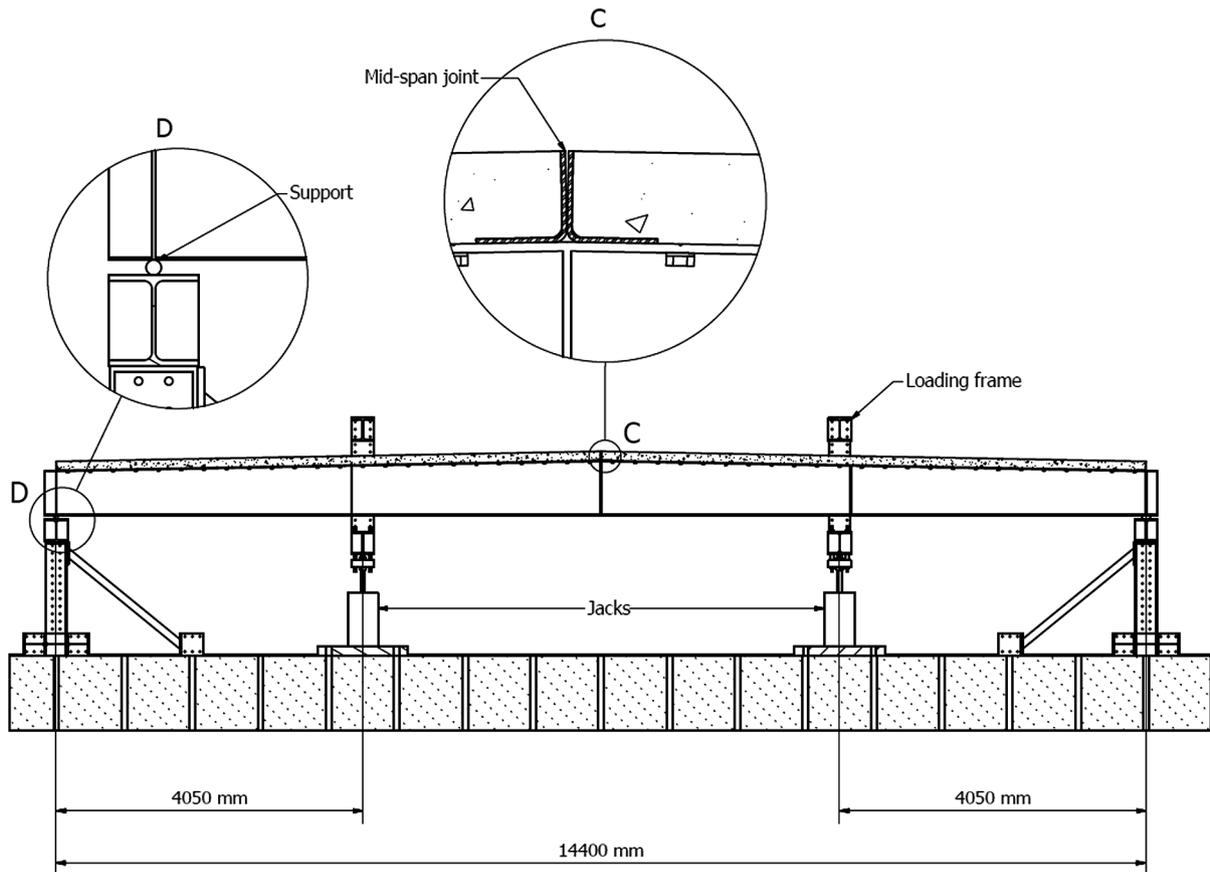


Fig. 6. Cross-section (side-view) of experimental set-up. Two solid concrete decks are supported by two tapered steel beams. The 14.4 m composite beam is subjected to four-point bending at 4.05 m from the supports. The transverse connection between the decks at mid-span is only capable of transferring normal forces. The c.t.c. distance between the steel beams is 2.6 m.

commercially available finite element package ABAQUS. To limit the calculation time, the model was reduced to a quarter of its actual size by utilizing the symmetry of the experimental set-up. The finite element model of the composite beam is illustrated through Fig. 7

The prefabricated concrete deck is modelled as a solid part using

eight node brick elements with reduced integration (C3D8R) with elastic material properties ($E = 33$ GPa). The steel angle profiles are modelled as part of the prefabricated concrete deck. For the tapered steel beam, four-node elements with reduced integration (S4R) are used. The steel braces are modelled with 2-node linear beam elements

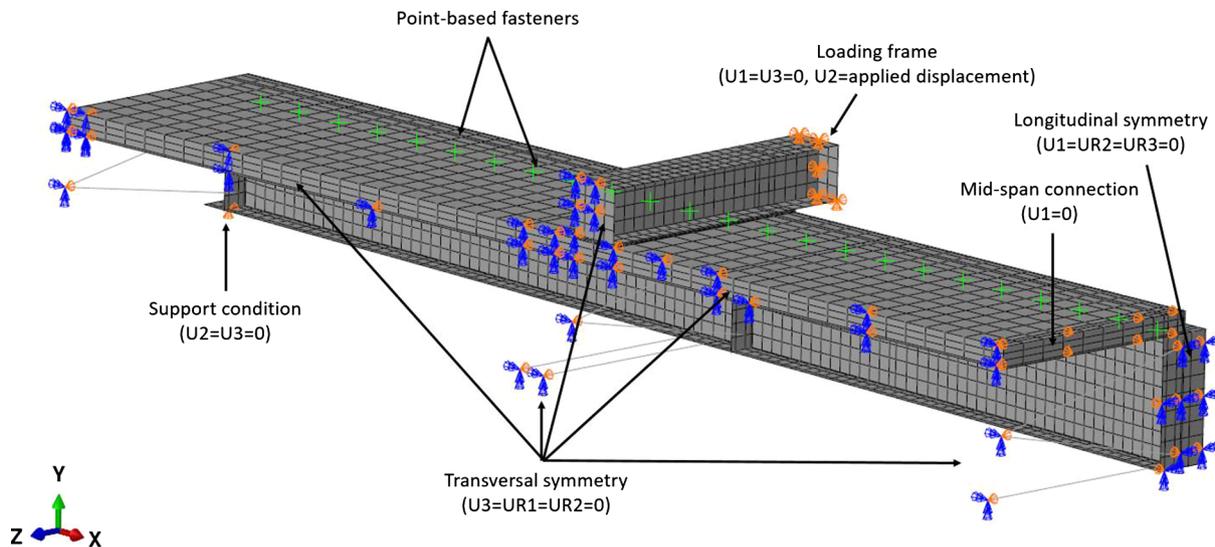


Fig. 7. . Finite element model of composite beam, indicating the boundary and symmetry conditions, as well as the point-based fasteners representing the shear connectors.

(B31) and are tied to the steel beam. All steel elements are modelled using elastic material properties ($E = 210$ GPa).

The discrete longitudinal shear connectors are modelled using mesh-independent, point-based fasteners. The fasteners are defined as coupled on motion, each with a spring stiffness of 55 kN/mm based on the data extracted from Fig. 4b. Rigid springs prevent the vertical separation of the concrete slab and steel beam at the locations where either an injected or non-injected external bolt is present.

The interaction between all the elements is modelled using a *General Contact* definition, characterized by *hard contact* in normal direction and a friction coefficient of 0.3, corresponding to the nominal friction coefficient for a steel-steel interface.

The load on the composite beam is applied by applying a vertical displacement to the loading frame.

3. Results and discussion

The demountability and reusability of the composite beams was successfully demonstrated during the experimental programme. All components of the specimen (concrete decks, steel beams and external injection bolts) were fully demounted, completely taken apart, and reused in the subsequent experiments, without requiring any major revision or repair. Only the resin infills remained within the bolt holes, but removing these was proven a rather simple and quick task.

The load-displacement and load-end slip curves of the experiments are illustrated in Figs. 8 and 9, respectively, per shear connector arrangement. The results of the beam test are evaluated using the effective bending stiffness and the effective shear stiffness of the composite beam, respectively defined as

$$k_{b,\text{eff}} = \frac{\Delta F}{\Delta W(x = L/2)}, \quad (1)$$

$$k_{s,\text{eff}} = \frac{\Delta F}{\Delta u(x = 0)}, \quad (2)$$

in which ΔF is the force increment, $\Delta W(x = L/2)$ is the deflection increment at midspan and $\Delta u(x = 0)$ is the slip increment at the supports. Both the effective bending stiffness and the effective shear stiffness parameters are determined in the interval 15–25 mm of midspan deflection. For connector arrangements U-0 and U-6, the parameters are determined in the last 10 mm interval of midspan deflection instead.

The results of the experiments are summarized in Table 1, together

with the predictions based on the finite element model. Very good agreement is found between the finite element model and the experimental results in terms of the effective bending stiffness: an average deviation of 2% was obtained. The experimentally determined shear stiffness parameters, however, differ significantly from the finite element predictions. The actual slip was 36–52% smaller than expected.

A similar agreement in terms of effective bending stiffness and disagreement in terms of effective shear stiffness can be found in the work of Gogoi [28]. Gogoi carried out multiple tests on nominally identical (prismatic) composite beams, whilst only varying the condition (e.g. surface roughness) of the steel-concrete interface. The experimental load-deflection curves that were obtained by Gogoi [28] are almost identical for all beams, whereas the relation between applied load and end-slip differs significantly. A similar observation is made by Leskelä [29] in the work of Hanswille et al. [30]. These observations imply that no reliable prediction method for end-slip exists, and that it is unclear how the relatively large difference between the experimentally and theoretically obtained end-slip can be explained. The same method, however, provides a satisfactory prediction in terms of deflection and curvature of the composite beam. Clearly, further investigation is necessary to explain a seemingly complex relation between theoretically predicted and experimentally observed end-slips.

Along the same line as the research of Gogoi [28], one of the two concrete decks was removed to inspect the beam-deck interface. It was found that a resin layer had formed between the solid deck and the steel beam during the injection process, which was intentionally broken during the trial tests. To investigate if adhesion is responsible for deviation in slip between experiments and models, the beam-deck interface in one half-span was thoroughly cleaned and greased to reduce the effect of adhesion and friction. The deck in the other half-span was left in the as-tested condition. An additional experiment was carried out with the C-6 shear connector arrangement. No significant difference in end-slip was observed in either half-span, indicating that adhesion and friction at the interface may only have had a small effect in the original test series. This hypothesis was later confirmed when testing the behaviour of the flooring system without any shear connectors (arrangement U-0) but with the same interface conditions. During this test, the effective bending stiffness of the beam approached that of the steel beam, indicating that no significant shear interaction (e.g. due to adhesion and friction) was present.

Because the number of shear connectors affects the costs of the structure, it is appropriate to reduce the number of shear connectors to

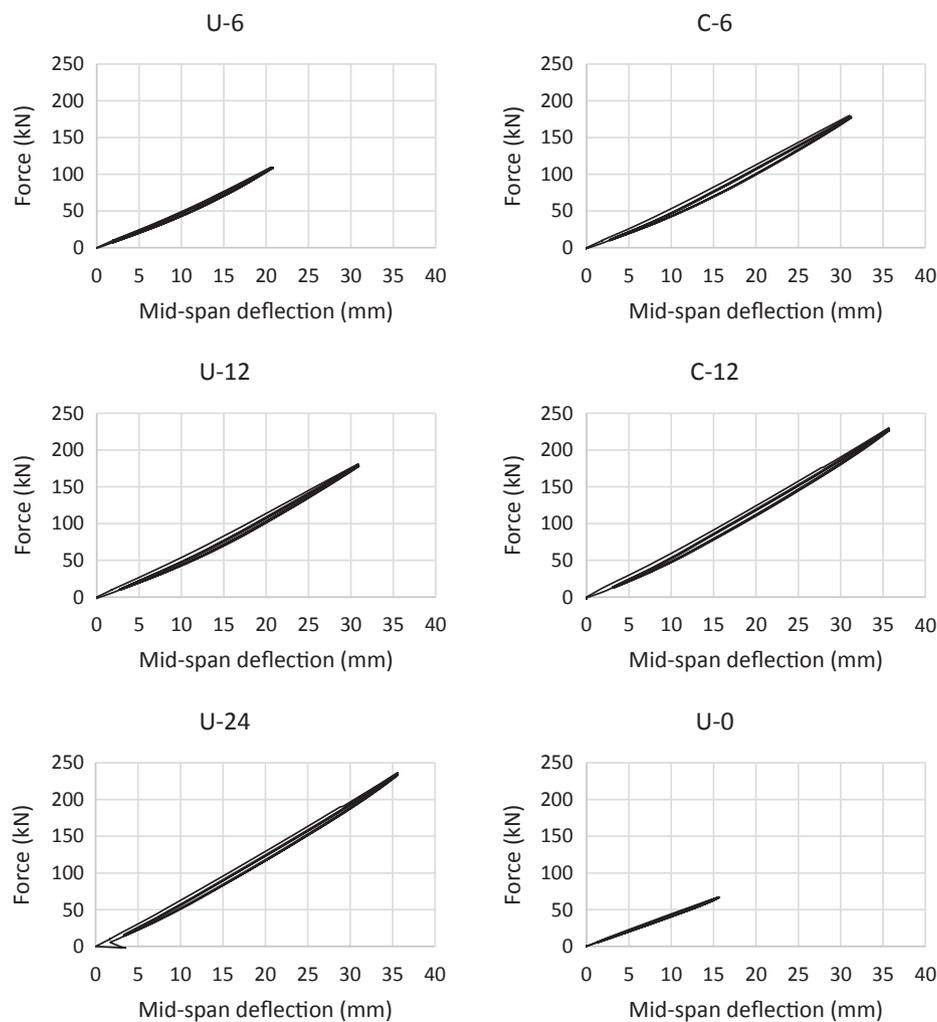


Fig. 8. Experimental load-deflection curves for the considered shear connector arrangements, for the five loading cycles. At equal force, the deflection is reduced in case the same number of shear are concentrated towards the supports.

a minimum. The results presented in Table 1 and Fig. 8 indicate that the number of uniformly distributed shear connectors can be reduced by approximately 40–50% by concentrating the remaining number of shear connectors near the supports, without an increase in deflection or end slip. The same tendency was previously described by Roberts [22] and Lin et al. [23].

Concentrated shear connector arrangements in the support regions could easily be implemented in practice if the only goal of the shear connection is to fulfil deflection and end-slip criteria under serviceability loads. The design load in the ultimate limit state is then to be resisted by the steel beam alone. A suggestion for this design approach was made by Crisinel [31]. Previous research [26] has indicated that there is no need for connectors to prevent the vertical separation between beam and deck in the elastic region – this claim is further investigated in an on-going experimental study.

Fig. 10 illustrates the longitudinal stresses over the height of the steel beam at 5 m from the supports. Numerically and experimentally obtained results show excellent agreement: an average difference of 4.9% for the stresses and 0.7% for the curvature was obtained. It was observed that, in line with expectations, the elastic neutral axis in the steel beam shifts upwards as the effective bending stiffness and the effective shear stiffness increase.

Although the current work addresses the application of demountable and reusable composite beams in buildings, the application could also be extended to bridges. On-going research addresses the behaviour

of the shear connectors and resin under cyclic loading to allow for the broader field of application.

4. Conclusions

The main outcomes of the experimental and numerical assessment of demountable and reusable tapered composite beams are:

- Demountability and reusability of a composite flooring system, consisting of tapered steel beams and prefabricated concrete decks, connected by resin-injected bolt-coupler shear connectors, was successfully demonstrated.
- Initial slip of an external bolt ($\text{Ø}20$ mm) in a significantly oversized bolt hole ($\text{Ø}32$ mm) can be prevented by injecting the remaining bolt-to-hole clearance with RenGel SW 404 + HY2404/HY5159 epoxy resin. The principle of resin-injected bolted connections allows for greater positioning tolerances and geometrical deviations for composite structures, whilst the composite action is achieved instantaneously and simultaneously under live-load.
- The results of the finite element analysis closely match the experimental results both in terms of deflection (average deviation 2%), curvature (average deviation 0.7%) and stresses (average deviation 4.9%).
- The end-slip observed in the experiments is significantly smaller (36–52%) compared to the finite element predictions. The results

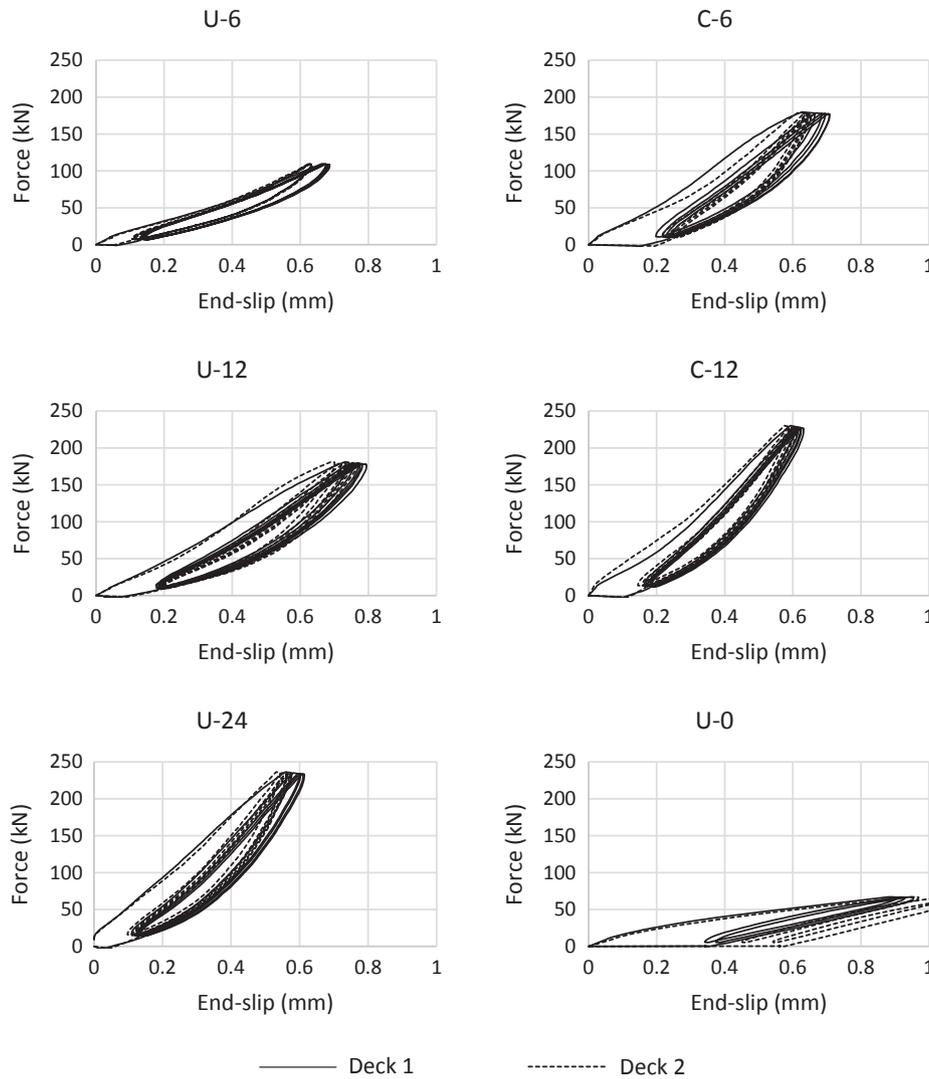


Fig. 9. Experimental load vs. end-slip curves for the considered shear connector arrangements, for the five loading cycles. The results have been averaged per deck (half-span). At equal force, the end-slip is significantly reduced in case the same number of shear connectors are concentrated towards the supports.

Table 1
Effective bending stiffness and effective shear stiffness for the various shear connector arrangements, based on the experimental results and the numerical models.

Arrangement	$k_{b,eff}$ (kN/mm)		$k_{s,eff}$ (kN/mm)	
	Experiment*	FE model	Experiment*	FE model
U-24	6.89	7.13 (+ 3.5%)	514	301 (- 41%)
C-12	6.69	6.96 (+ 3.9%)	487	294 (- 40%)
C-6	6.18	6.28 (+ 1.7%)	389	188 (- 52%)
U-12	6.35	6.53 (+ 2.9%)	330	190 (- 42%)
U-6	5.82	5.87 (+ 0.9%)	199	128 (- 36%)
U-0	4.10	4.07 (- 1.0%)	98	51 (- 48%)

* Average result over the number of load repetitions, taken over the interval 15–25 mm deflection at mid-span. For arrangements U-0 and U-6, the results were taken over the last 10 mm of deflection.

obtained with an as-delivered steel-concrete interface do not significantly differ from those obtained with a greased interface, indicating that the deviation is not likely caused by friction nor adhesion. This statement is supported by tests on the composite beam without any shear connectors, in which the bending stiffness of the non-composite beam numerically predicted and experimentally obtained. Literature study showed that this deviation has occurred in other experiments as well, and that yet no method or explanation exists to explain the discrepancy in end-slip.

- For a simply supported tapered composite beam, the number of shear connectors required to meet the deflection criterion or the elastic-slip criterion in the relevant limit state can be minimized by redistributing those connectors closer to the supports.

5. Further research

Additional investigation of the end-slip observed in (tapered) composite beams is necessary to understand the reason for the discrepancy between experiments and finite element models.

The behaviour of the resin-injected bolt-coupler shear connector is investigated under cyclic loading to extend its application in the field of bridge engineering.

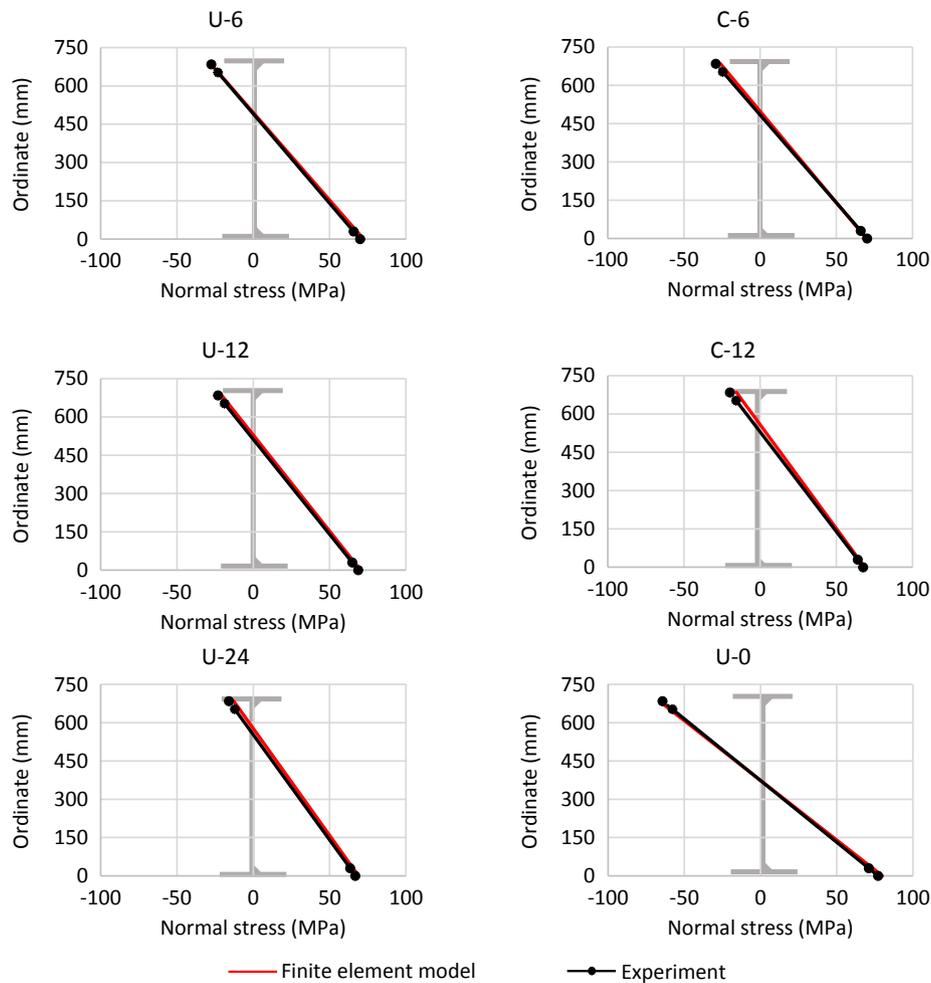


Fig. 10. Normal stresses over the height of the steel beam for the cross-section at $x = 5.0$ m, for applied point loads of 100 kN. The experimental results for U-0 and U-6 have been extrapolated for comparison. The elastic neutral axis shifts upwards as the effective bending stiffness and the effective shear stiffness increase.

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