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Viability and performance of demountable composite connectors

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A R T I C L E   I N F O

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A B S T R A C T

Material production, and associated carbon emissions, could be reduced by reusing products instead of landfilling or recycling them. Steel beams are well suited to reuse, but are difficult to reuse when connected compositely to concrete slabs using welded studs. A demountable connection would allow composite performance but also permit reuse of both components at end-of-life. Three composite beams, of 2 m, 10 m and 5 m length, are constructed using M20 bolts as demountable shear connectors. The beams are tested in three-, six- and four-point bending, respectively. The former two are loaded to service, unloaded, demounted and reassembled; all three are tested to failure. The results show that all three have higher strengths than predicted using Eurocode 4. The longer specimens have performance similar to previously published comparable welded-connector composite beam results. This suggests that demountable composite beams can be safely used and practically reused, thus reducing carbon emissions.

1. Introduction: the opportunity to reduce carbon emissions by reusing composite structures

Every year 1500 million tonnes of steel are produced worldwide [1]. Although steel production processes are relatively efficient [2], they still cause emission of significant quantities of carbon dioxide into the atmosphere — approximately 9% of global anthropogenic emissions from energy and processes [3]. The construction industry uses approximately half of steel produced [4] and reuse has been identified as having potential to reduce this tonnage, and hence associated carbon emissions [2].

Addis [5] identifies three characteristics that a component must have to be reusable: it is not worn, yielded or corroded; it is not a superseded technology; and it can still interface with new components. Structural steel beams meet these requirements provided they have not been exposed to fire, seismic or other extreme loading scenarios as their standard sizes and connection technologies have not changed in the past 50 years [5]; thus they are ideal candidates for reuse. If the rate of reuse can be increased (at present it is estimated that 1.5% of steel beams exiting construction in the UK [6] are reused) then there is potential to decrease demand for new and recycled steel.

Composite floors are the most common structural system for multi-storey buildings in the UK, accounting for approximately 40% of such floor area built annually [7]. However composite construction is listed as a barrier to deconstruction [8] with Webster & Costello [9] recommending it be avoided in designs for deconstruction. If a system can be found that permits composite action and also allows deconstruction, then reuse can be enabled and hence carbon dioxide emissions reduced.

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2. Review of published literature

Research into behaviour and prediction of composite construction has used ‘push tests’ extensively to verify models and formulae; however push tests have recently been shown to have poor correlation with actual construction practice. Most papers to date have examined welded connectors.

2.1. Review of literature on traditional, welded-connector composite beams

Engineering understanding of composite steel–concrete construction systems has evolved over the past century mainly based on ‘push tests’ supplemented by modelling. Recent research suggests that these tests do not correlate well with beam tests, which are more reflective of the actual use of composite beams. Design guidance has been continually updated to incorporate developments in understanding and research.

‘Push test specimens’ (an example of which is shown in figure B.1 Eurocode 4-1-1 [10]) were developed in the 1930s [11] to determine the behaviour of composite connectors (called ‘studs’). Lloyd & Wright [12] report that at this time composite steel–concrete beams were mainly used in bridge construction, with a flat-soffit slab cast on top of a beam with factory-welded connectors to transfer shear between components. They go on to explain that, as composite slabs were adopted in building construction, profiled steel decking was used as permanent formwork; this eliminated direct contact between the beam and concrete and necessitated the site-welding of studs. That design standards BS 5950-3.1:1990 [13] and BS EN 1994-1-1:2004 [10] provide guidance only for welded connectors is evidence of the ubiquity of this form of composite construction in buildings. Mottram & Johnson [14] recommend geometric adjustments to the standard push test specimen, defined in 1965, to make it suitable for use with profiled decking.

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All literature reviewed uses results from push tests (either new or previously published) to validate theoretical models of composite behaviour and, almost always, to appraise and update design guidance. Hawkins & Mitchell [15] conclude from 23 push test results that connector spacing and geometry greatly impact the failure load; Mottram & Johnson [14] undertake 35 further tests to appraise design formulae. Greater computing power has allowed increasingly detailed modelling of beam and connector behaviour: Johnson & Molenstra [16] input a mathematical model from first principles to calculate strength and slip, while Ellobody & Young [17] and Qureshi & Lam [18] create finite element models to do the same; such models are still validated against push test results however.

Over time an increasing number of failure modes have been identified from push tests: Hawkins & Mitchell [15] describe four (stud shear, concrete pull-out, rib shear, rib punching), Johnson & Yuan [19] identify three more (splitting failure and two combination modes) and Patrick [20] classifies four additional, less common modes. Patrick claims that existing guidance for trapezoidally-profiled decking considerably underestimated strengths and slip capacities for welded studs. Responding to these claims, Hicks [11] performed 6 push tests and 2 beam-bending tests and showed that the two sets of results have a poor correlation. Given that composite floors in buildings are subject to loading in bending, Hicks concludes that the push-test specimen is deficient when evaluating beams with profiled decking; hence the design specifications are still safe (though a few minor corrections are needed). Smith & Couchman [21] concur with Hicks, recommending minor updates to design guidance based on the results of 27 push tests from a rig modified to better correlate with beam tests.

2.2. Review of literature on demountable connectors

Oehlers and Bradford [22] list different composite connector types, some of which are bolted or otherwise demountable. Dallam [23] and Marshall et al. [24] performed tests in the 1960s investigating the behaviour of friction-grip bolts as composite connectors but focused on the effect of pre-tensioning on the connection and did not demount them. More recently, Kwon et al. [25] post-installed bolts to strengthen existing structures, investigating their performance under fatigue loading. In conference papers, Lee and Bradford [26] develop a ‘quasi-elastic mechanics based’ theoretical model for the behaviour of pre-tensioned bolts and validate it against push test results, while Lam & Saveri [27] describe experiments using connectors machined from traditional studs with threads (shown in Fig. 1) so they can be bolted onto a beam and disassembled. Both sets of authors show that the bolted connection performs suitably in a push test, but beam tests were not completed. Although not supported by published research, the Australian building standard AS 2327.1-2003 “Composite structure: Part 1: Simply supported beams” [28] depicts a bolted connector with a comment that they should be treated as if the same as manually welded connectors; however no references to the bolted connector are made in the text.

Fig. 1. Demountable connectors machined from traditional studs, taken from Lam & Saveri [27].

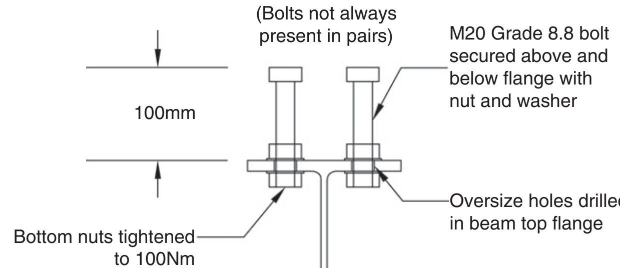

Fig. 2. Demountable, bolted connector.

2.3. Findings from literature review

In the body of published work on composite steel–concrete construction there have been a large number of push tests but few beam tests — despite poor correlation between the two and beam tests being closer to actual use of connectors in construction. None of the research into connector failure modes, for example Yuan & Johnson [29], inherently precludes bolt use as none require moment resistance at the connector base. Of the articles that have examined demountable connectors, few have examined demountability and none present results from beam tests, or on tests using non-preloaded bolts as connectors. This research, therefore, investigates the behaviour of such bolts used as composite connectors in two beam tests.

3. Methodology to test a demountable connector design

Three composite beam specimens, of lengths 2 m, 10 m and 5 m, were laboratory tested to investigate the behaviour of steel bolts as demountable composite connectors. The 2 m specimen was used to test the concept. To compare connector performance with that of welded studs, the larger specimens were constructed to the same specifications as Hicks' [11], who undertook tests on 10 m and 5 m specimens with welded studs in the same laboratory in 2005. UK practice is to use profiled steel decking, so the same commercially available decking (Multideck 60-V2, 0.9 mm thick [30]) as used by Hicks was chosen for all three specimens. The decking was laid on top of the steel beams and connected by 20 mm diameter grade 8.8 bolts through 24 mm diameter holes pre-drilled through the decking and top flanges, then fastened by washers and nuts on either side and tightened to 100 Nm as shown in Fig. 2. Following the procedure of Hicks, fewer than the optimal number studs were installed to ensure they were fully loaded at failure.

3.1. Laboratory testing of demountable connector design in 2 m specimen

A 2 m long specimen was constructed as shown in Fig. 3, with C16/20 concrete poured to form a 140 mm thick slab 0.5 m wide on top of a UB 254 × 102 × 28 S355 steel beam. Two demountable connectors were

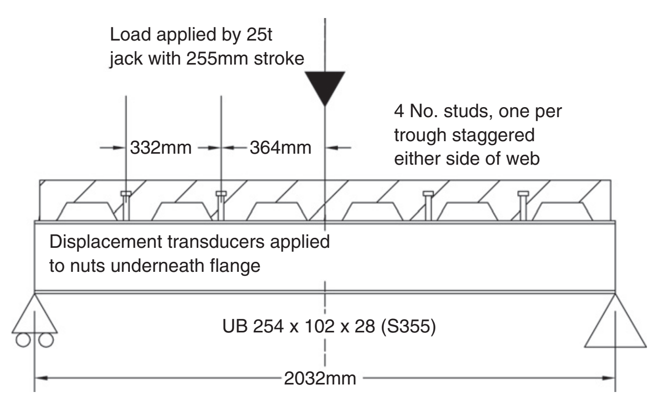

Fig. 3. Geometry and loading setup for 2 m specimen.

Fig. 4. Displacement gauge attached to the underside of beam top flange measuring relative slip of nut.

placed in each half-span in the 'favourable' trough position, staggered either side of the beam web.

Displacement gauges were placed against the lower nut of each connector and fixed to the underside of the flange, as shown in Fig. 4, to measure relative slip. Displacement gauges were also placed at the loading point and the beam midpoint and third points to measure deflection. Loading was imposed at the rate of approx. 2 mm/min via a 25 t hydraulic jack mounted on a rig to subject the specimen to 3-point bending. The beam was initially loaded to a service moment of 4 kNm, equivalent to a uniform distributed load of 6.5 kN/m^2 (a typical office loading as specified by Eurocode 1 [31]). It was then unloaded and demounted – the bottom nuts released and the beam lowered clear of the slab – to test that the bolted connector design did facilitate reuse. The beam was then reattached and reloaded in cycles to increasingly higher loads until failure occurred.

3.2. Laboratory testing of demountable connector design in 10 m specimen

Fig. 5 shows the arrangement of the 10 m specimen, mimicking Hicks' [11] setup: 7 pairs of bolts in one half-span and 15 single bolts (staggered either side of the web to ensure even application of force) in the other; 2.5 m wide slab cast from C16/20 concrete, 140 mm thick on the decking. Following Hicks, the beam was propped at the third-points until testing so the full self-weight was applied to the connectors once the props were struck. Displacement gauges were placed at each nut along one side of the beam and at the nuts closest the support and the middle on the other side. Displacement gauges were also attached to the slab midpoint and third points. Strain gauges were affixed longitudinally at the centre of the flanges and at 45° to the vertical on the web at 15 locations indicated in Fig. 5.

Following the approach of Hicks [11], the beam was loaded in six-point bending using two hydraulic jacks mounted on rigs, each loading two spreader bars. The rate of imposed displacement was approx. 5 mm/min (as measured at midspan), continued until an imposed service moment of 81 kNm was reached, equivalent to a uniformly distributed load of 6.5 kN/m^2 (again chosen as a typical office loading from Eurocode 1 [31]) and then unloaded. After twice repeating this, the bottom nuts were loosened and the slab jacked up approx. 10 mm clear of the beam. The slab was then lowered and beam reattached. The specimen was reloaded to service three times and gauges affixed to either end of the beam to measure relative displacement of the slab. Loading was increased in cycles until failure occurred in one half. To try to force failure in the other half, an end-stop was welded at the left-hand end of Fig. 5 to prevent the left half-span from moving further.

3.3. Laboratory testing of demountable connector design in 5 m specimen

After the procedure described in Section 3.2 was applied, half of the composite beam appeared not to have failed. Following Hicks' [11] methodology, and to gather further data, the beam was then cut in half and the unfailed portion tested as shown in Fig. 6. Clearly this 5 m specimen had the same slab geometry and sensors attached as the parent 10 m specimen. However eight studs were now in the 'unfavourable' location of the trough. A spreader bar was used to load the beam in 4-point bending using a hydraulic jack mounted on a rig, imposing a cyclic displacement until failure, at a rate of approximately 5 mm/min, as measured at midspan.

3.4. Analysis of results and verification

Data were recorded from the displacement and strain gauges along the specimens, and from a loadcell attached to each jack. These were analysed and compared with predictions calculated using Eurocode 4-1-1 [10], informed by results from concrete cube and steel tensile tests performed to obtain the materials' properties. For the 2 m specimen an elastic analysis was used to back-calculate the failure moment to cause crushing strains in the concrete. The results from the larger two specimens were compared with Hicks' [11] previously published results.

4. Results from demountable connector tests

Results are presented for the three specimens tested, all showing moment capacities in excess of those predicted. The results of the two larger beam tests are within 12% of those previously published for welded studs.

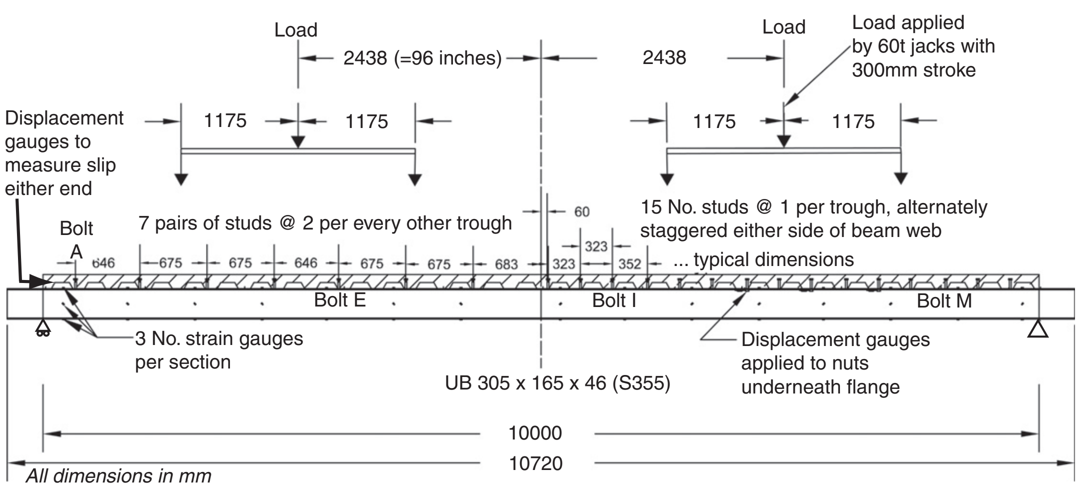

Fig. 5. Geometry and loading setup for 10 m specimen.

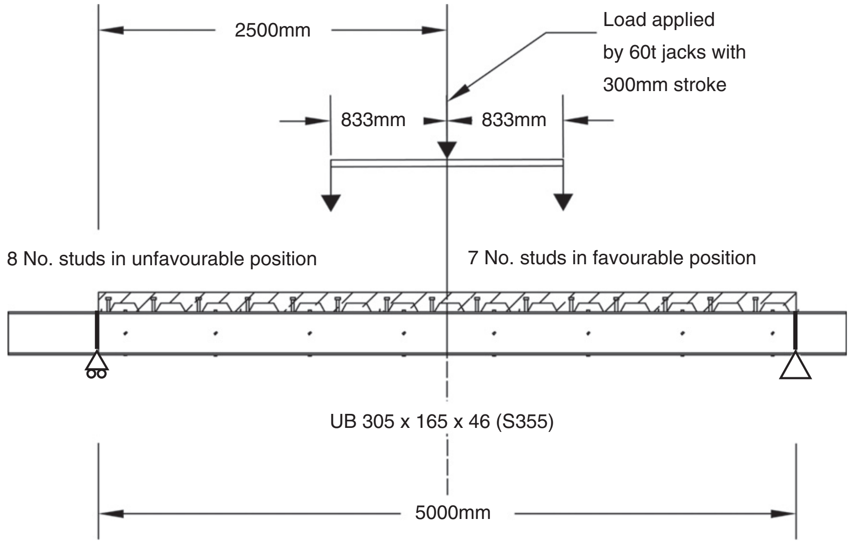

Fig. 6. Geometry and loading setup for 5 m specimen.

4.1. 2 m specimen results

The 2 m specimen was successfully demounted and reassembled. Fig. 7b shows the demounted beam below the suspended slab, contrasted with the initial configuration in Fig. 7a.

Results from the material tests for the 2 m specimen are given in Table 1. Eurocode 4-1-1 [10] calculations with these values predict failure in the concrete at the connector at a moment of 185 kNm.

Failure actually occurred at a moment of 246 kNm (32% greater than predicted) due to compression in the slab at midspan as shown in Fig. 8 (a plastic hinge had already started to form in the steel beam). A failure moment of 248 kNm was calculated from the back-analysis of strains; this is within 1% of the experimental value.

Fig. 9 shows the moment-displacement profile at the midspan of the 2 m specimen; displaying elastic and plastic regions as expected. Bolt slips, as measured at the underside of the flange, were nowhere more than 2 mm.

4.2. 10 m specimen results

The 10 m specimen was successfully loaded to service, demounted and reassembled; the latter two processes were achieved more easily and quickly than had been anticipated. Fig. 10 shows the test specimen in initial and disassembled states.

The reassembled beam was then loaded until the decking had delaminated from the slab in the left half-span at a midspan deflection of 280 mm. This was confirmed as pull-out failure in a cone shape around the bolts, shown in Fig. 11, once the decking was removed. After testing was complete longitudinal cracks were noticed along the centreline of the slab, further indicating concrete failure initiated at the bolt locations.

The results of the cube and coupon tests for the specimen are given in Table 2. Eurocode 4-1-1 [10] calculations with these values predict failure of the concrete at the stud pairs at a moment of 357 kNm, 5% less than the maximum moment (including self-weight) recorded experimentally: 378 kNm.

After the end-stop was welded, failure was predicted (using the values in Table 2) in the concrete around the single studs at a moment of 375 kNm. Midspan deflection was increased to 490 mm, causing a moment of 434 kNm (14% higher than the predicted maximum) but without causing failure in the right half-span — at this point it was noticed that the end-stop itself had failed and the experiment halted.

Fig. 12 shows the moment-displacement graph at the midspan of the specimen, with self-weight moment and predicted failure moments from Eurocode 4-1-1 [10] indicated. Also plotted are results from Hicks [11], whose displacement values were measured relative to the propped mid-span height and therefore have been uniformly reduced to facilitate comparison (Hicks' predictions are not shown).

Both curves exhibit elastic behaviour initially followed by a ductile plateau, caused by formation of a plastic hinge in the beam approximately under the point-load immediately left of midspan, revealed by Lüder's wedges visible in the web and confirmed by strain gauge

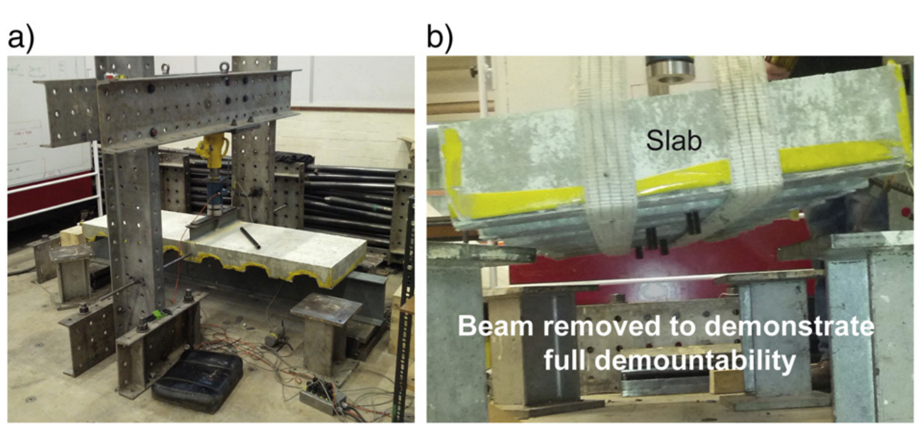

Fig. 7. a) Initial, assembled 2 m specimen and loading rig; b) Demounted slab after loading to service and unloading.

Fig. 8. Crack along a shear-plane, indicating compression failure of 2 m specimen.

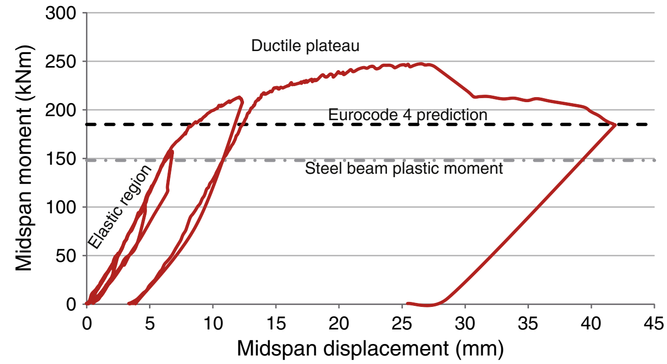

Fig. 9. Moment vs. displacement for 2 m specimen at midspan.

Table 1
Measured material properties for 2 m specimen.

UB 254 × 102 × 28 (S355) steel beam	
Mean flange yield strength	420 MPa
Mean web yield strength	480 MPa
C16/20 concrete slab	
Age at testing	14 days
Mean compressive cube strength ($f_{cm, cube}$)	21.1 MPa
Characteristic compressive cube strength ($f_{ck, cube}$)	20.6 MPa
Characteristic compressive strength (f_{ck})	16.5 MPa[a]

Beam dimensions assumed same as from standard UK catalogue [32].
[a] Calculated from BS EN 1992-1-1 [33]; other concrete properties taken from typical values from this source.

readings. Both beams fail due to concrete pull-out in the left half-span (i.e. with stud pairs) at similar moment values. This is surprising as Hicks reports a concrete characteristic strength 14% higher and a beam axial capacity 3% higher, so the expected difference in maximum moment is 8%. Once end-stops are welded results cannot be compared as Hicks' end-stop was designed differently and did not fail prematurely.

Fig. 13 is an enlargement of the initial portion of Fig. 12 (and omitting Hicks' [11] values) to compare moment-deflection curves just for the service loading cycles before and after demounting. As can be seen the curves are almost identical once initial 'bedding in' occurs after remounting.

Plots of end-slip with moment are given in Fig. 14, showing the left slip only until the end-stop is welded. Ductile behaviour is seen in both sides, but magnitudes are greater on the left side: maximum left slip is 19.8 mm, while Hicks [11] reports a corresponding value of 26.5 mm.

Fig. 15 displays a plot of midspan moment against the slip of four bolts (as measured at the lower nut) taken from different locations along the beam (labelled on Fig. 5). Each bolt has nominally 4 mm of clearance in the oversized holes, thus potentially 4 mm of slip can occur before the bolt must bear on the side of the hole (i.e. the beam flange). Assuming that the bolts are initially randomly positioned in the holes, it is then not surprising that some bolts (e.g. A and I) slip less than 1 mm whereas others slip almost 3.5 mm (e.g. bolt E) — none however slip more than 4 mm. Once the bolt bears directly on the flange little further slip occurs as would be expected; some reverse slip of the nut is seen, for example bolt E, potentially caused by rotation of the bolt as the slab continues to move away from the centre. A shear force of approx. 5 kN is needed to overcome the friction induced by the torque on each bolt; this may explain the small initial gradient to each plot. Concrete pull-out prevents the left half-span bolt slips being correlated with the left end-slip, however right end-slip was 4.5 mm before the end-stop was welded (i.e. while this half-span was unfailed), which is a similar magnitude of slip to the right half-span bolts (e.g. bolt M). Bolts for the 2 m and 5 m specimens showed similar slip patterns.

Strain profiles at midspan are plotted in Fig. 16 for different values of moment. As expected the neutral axis position falls as the slab slips under increasing load. The maximum net axial force in the beam is 771 kN, or 55.1 kN per stud, 54% greater than the 35.8 kN capacity predicted by Eurocode 4-1-1 [10].

4.3. 5 m specimen results

The 5 m specimen (shown in Fig. 17a) was loaded in cycles until deck delamination occurred (shown in Fig. 17b) in the right half-span of the beam – where bolts were in the 'favourable' position (as indicated in Fig. 6) – at a maximum moment of 376 kNm and midspan deflection of 145 mm.

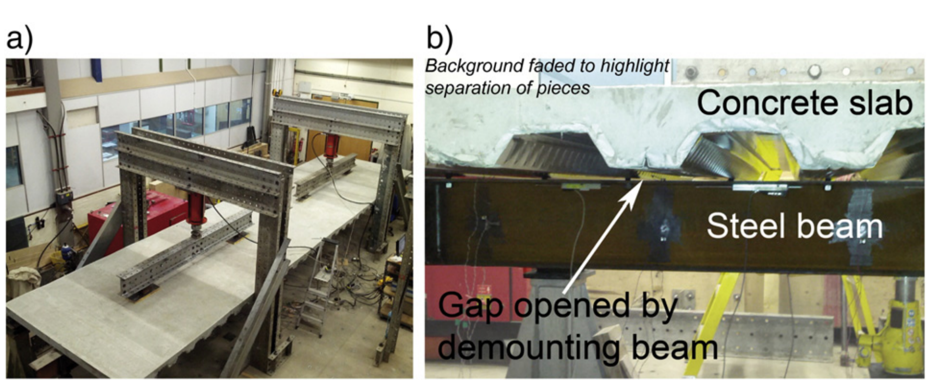

Fig. 10. a) Initial, assembled 10 m specimen and loading rig; b) Demounted beam after loading to service and unloading.

Fig. 11. Cone failure surface indicative of pull-out failure in left half-span of 10 m specimen.

Table 2
Beam and slab properties for 10 m specimen.

UB 305 × 165 × 46 (S355) steel beam	
Mean flange yield strength	376 MPa
Mean web yield strength	395 MPa
Depth of section	303 mm
Width of flange	167 mm
Flange thickness	10.9 mm
Web thickness	6.6 mm
C16/20 concrete slab	
Age at testing	18 days
Mean compressive cube strength ($f_{cm,\ cube}$)	13.8 MPa
Characteristic compressive cube strength ($f_{ck,\ cube}$)	13.3 MPa
Characteristic compressive strength (f_{ck})	10.7 MPa[a]
Secant modulus of elasticity (E_{cm})	24.6 GPa[b]

[a] Calculated from BS EN 1992-1-1 Section 3.
[b] Derived from beam bending stiffness.

Concrete cube and cylinder tests undertaken the same day the specimen failed (68 days after casting) resulted in a characteristic cylinder compression strength of 11.1 MPa. Eurocode 4-1-1 [10] calculations performed using this value and other properties taken from Table 2 predicted failure of the concrete at the studs at a moment of 328 kNm; 13% lower than that found experimentally. Inspection of the slab once the decking had been removed confirmed concrete pull-out failure around the bolts.

Fig. 18 shows a moment-displacement graph for the specimen at midspan, with self-weight moment and predicted failure moment indicated. Also plotted are the results of Hicks' [11] 5 m test (though not his predictions).

Both curves show elastic then ductile behaviour, and both witnessed plastic hinges forming in the beam near the left load point. Unlike this experiment, Hicks observed failure in the half-span with 'unfavourable' stud locations first, then welded an end-stop and failed the other half-span. Hicks reported a concrete strength 10% higher, which Eurocode 4-1-1 [10] calculations suggest should give a maximum moment 7% higher, however the actual value is approximately 12% higher.

Fig. 19 shows the variation of end-slip with midspan moment, displaying ductile behaviour after initial elasticity. Maximum end-slips of 13.3 mm (left side) and 12.0 mm (right side) were recorded — similar to the 12.9 mm of slip Hicks [11] reports for first failure.

Readings from the strain gauges on the 5 m specimen suggested that many no longer gave consistent output, possibly due to over-straining by the large imposed deformation on the 10 m specimen.

5. Discussion of implications of results

The experimental results demonstrate that a composite beam with bolted connectors performs in a similar manner to such beams with welded studs, predictably meeting the required design and safety standards. Further research would optimise both bolt design and design guidance. The reality of using bolted connectors on commercial projects is explored, finding two challenges and two potential solutions. Policy recommendations are needed to encourage adaptation of demountable and reusable systems in construction.

5.1. Comparison of results with predictions and with welded specimens

The maximum moment resistances reported in Sections 4.1, 4.2 and 4.3 are above the values predicted by Eurocode 4-1-1 [10]. This is expected because design standards such as Eurocode deliberately predict conservatively to allow for uncertainties. The low level of shear connection (20%) may explain the significant under-prediction for the 2 m specimen, as this is below the minimum level for the Eurocode.

The moment capacities of the 10 m and 5 m specimens are 2% and 12% lower than those from Hicks' [11] specimens using welded studs. Two reasons are explored for these discrepancies: material properties and holes drilled in the flange. Despite using an identical mix from the same commercial supplier, a lower concrete strength than Hicks was recorded for both specimens, which causes expected failure moments to be 8% and 7% lower respectively. The holes drilled in the top flange of the beam reduced the plastic moment capacity by 2–3%. Accounting for these two effects, the 5 m specimen's moment capacity is still 3% lower than Hicks' value; however the 10 m specimen's capacity is 8% higher.

The divergence for the 5 m specimens can potentially be explained by the larger strains imposed during the 10 m testing — Hicks' [11] 5 m specimen saw 100 mm less midspan deflection when still part of the 10 m specimen. These larger strains probably invalidated the

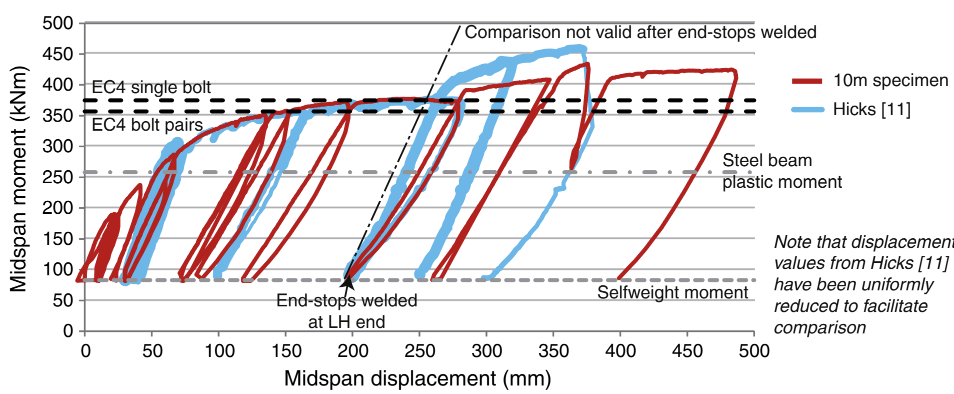

Fig. 12. Midspan moment vs. displacement for 10 m specimen and comparison with published values from Hicks [11].

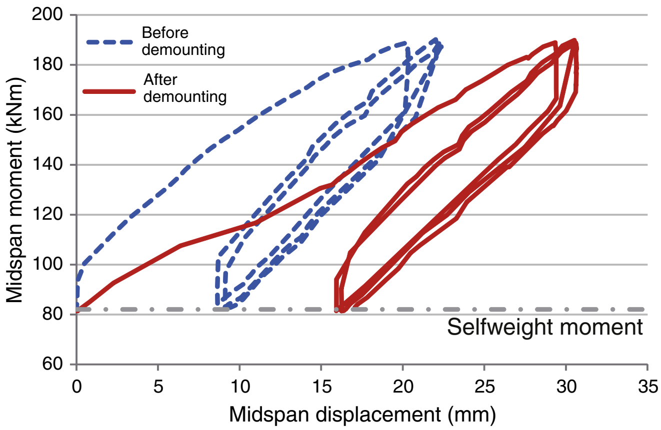

Fig. 13. Comparison of midspan moment vs. displacement of 10 m specimen before and after demounting.

'unfailed' assumption about the 5 m specimen, as shown by the strain gauge failures and the different failure sequence than that reported by Hicks [11]. The latter occurred because the 'favourable' half-span of the 5 m specimen had been more highly stressed (probably causing some failure at the shear connectors) under the large shears in the 10 m experiment, while the 'unfavourable' half-span experienced lower shear, being closer to the middle of the span. The 5 m specimen's ultimate moment capacity remains above predicted values (and almost 50% greater than the plastic moment capacity of the steel beam alone) despite the initial damage, indicating that sufficient shear connection remained.

Although both 10 m specimens failed in similar ways, the results from Section 4.2 exceed predictions whereas Hicks' [11] result was lower than expected. It is not clear why this divergence occurred although Hicks attributes the low result to uplift of the slab between troughs which was not witnessed in the bolted connector experiments — it is possible that the use of nuts and washers more effectively clamped the decking to the beam flange, preventing this phenomenon.

5.2. Avenues for further research

Knowledge about demountable connectors could be increased in three ways: producing tailored design guidance, creating an analytical model of internal interaction, and performing push tests. Performance of demountable connectors could be improved by research in two areas: optimising connector material and geometry, and reducing hole size.

To give confidence to designers when considering bolted connectors, tailored design guidance is required to provide formulae and empirical values suited for demountable connectors because formulae and empirical factors in current guidance, e.g. Eurocode 4-1-1 [10], assume welded studs. Laboratory testing may be required to calibrate these. The finding that bolts slip different amounts before bearing on the beam has implications for the forces in the beam and how these change as bolts slip. An analytical model could be developed to predict these internal forces and compared with experimental results — Lee and Bradford's [26] work could potentially be extended to include this. This phenomenon may also have an impact on beam stiffness — although results in Sections 4.2 and 4.3 indicate stiffness similar to Hicks' [11] specimens. Eurocode 4-1-1 [10] mandates push-tests to verify that ductility requirements are met, these should be undertaken for any bolts used, noting their limitations as discussed in Section 2.1. However push tests by Lam and Saveri [27] and Lee and Bradford [26] indicate that demountable studs perform better than welded studs in such tests anyway.

Research is needed to inform the optimal material properties and geometry for connectors, accounting for ductility as well as strength, and considering that standard practice uses higher-strength concrete. Grade 8.8 bolts (with a nominal ultimate strength of 800 MPa) were used in the present trials, unlike those used by Hicks' [11] whose studs had an ultimate strength of 513 MPa. Size M20 was chosen as similar to the 19 mm welded studs that Hicks used. 24 mm holes were chosen to facilitate demounting but it is possible that demounting could occur with standard 22 mm holes. However, commercially-designed composite beams would typically have higher shear connection resulting in the neutral axis being closer to the flange and reducing the loss in moment capacity, in which case the benefit of having smaller holes may be negligible.

5.3. Implementation of demountable composite beams in industry

The experiments demonstrated that the proposed bolted connector design allows demounting, and therefore reuse, and that the moment capacities can be reliably estimated by Eurocode 4-1-1 and are similar to results from beams with welded studs. Thus the proposed, demountable connector system could potentially be used safely in practice. However, in practice there may be a cost premium when implementing bolted studs on site: the unit cost of grade 8.8 bolts is estimated at three times that of similarly-sized welded studs; additional labour is required to install bolts as one person must be (at height) holding the nut underneath the decking whilst another is tightening it from above. Solutions are suggested to negate these extra costs. Additionally, two advantages of this system may justify any cost premium.

Further research can address the extra unit cost — Lam & Saveri [27] machined a traditional stud into a demountable version, so it is likely that a demountable, cost-efficient (when mass produced) solution can be found. Increased use of prefabrication and 'smart' construction

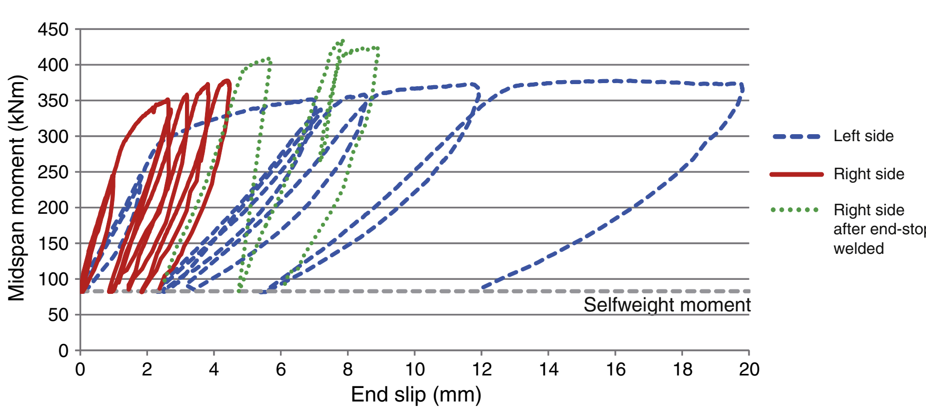

Fig. 14. Midpsan moment vs. end-slips for 10 m specimen.

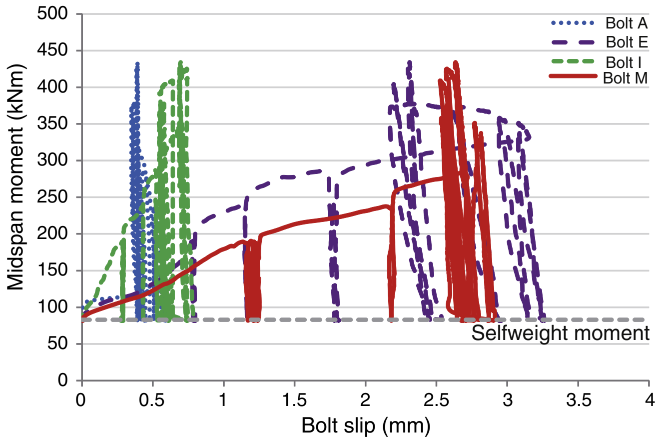

Fig. 15. Moment vs. slip for four bolts from different locations along 10 m specimen.

technology can address the extra labour requirement: the concrete slab could be manufactured off-site with the bolts cast in required locations protruding from the soffit, and then transported to site (a leading UK construction firm already prefabricates concrete units for use on site, giving a programme and cost saving). The steel beam can be predrilled with holes for the bolts as part of the automated fabrication process to ensure a good fit, requiring only one person to tighten the nuts from below. Optimising the bolt design for installation would aid this process, and may reduce the cost of alternative installation methods.

A demountable system would have two advantages over traditional connectors: no welding and increased flexibility. Welding studs alters their material properties, whereas bolts' material properties are unaffected by installation. Welds are susceptible to fatigue under cyclic or seismic loading, so bolts may be preferred in these circumstances — supported by Kwon et al.'s [25] findings. Site welding also involves extra health & safety risks that are avoided when bolting. Using demountable connectors could allow extra flexibility in the finished building as the steel beam can be replaced if the concrete were propped. This would allow a stronger/stiffer beam to be added if extra capacity/damping were required. Clients may be willing to pay a premium to achieve this type of structural flexibility, particularly if it facilitates faster installation/removal of stairways during fit-out between tenants. The evidence from the present trials that the specimens demounted easily suggests that the concept could work in commercial building. The nuts may become difficult to remove after 20 years in place or may damage their bond with the concrete in doing so — so further research is required to understand changes in bolt condition over time.

5.4. Further challenges and policy recommendations

While the technology now exists to demount and reuse steel beams, hence reducing carbon dioxide emissions associated with new material production, there is as yet no demand for this option. Policy makers should consider measures to incentivise reuse of construction materials, potentially through schemes that increase the value of materials at the end of structure life or that provide tax benefits for firms that commission demountable structures. The use of demountable studs to allow

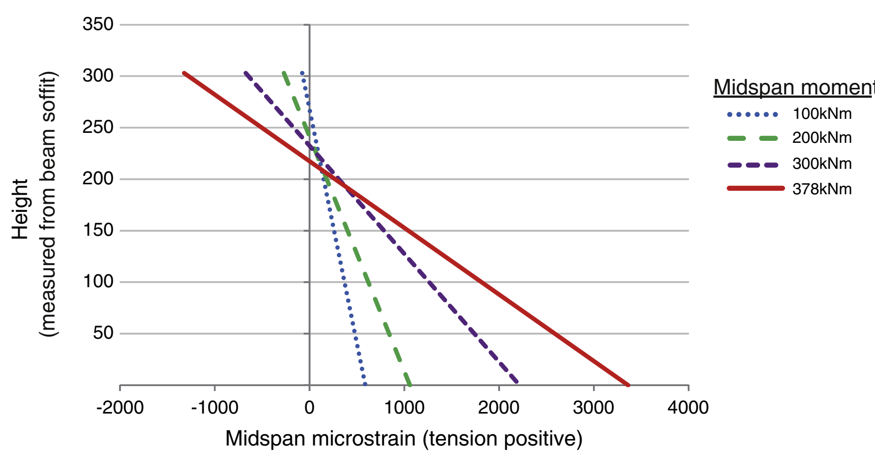

Fig. 16. Midspan strain profiles in 10 m specimen for different moment values.

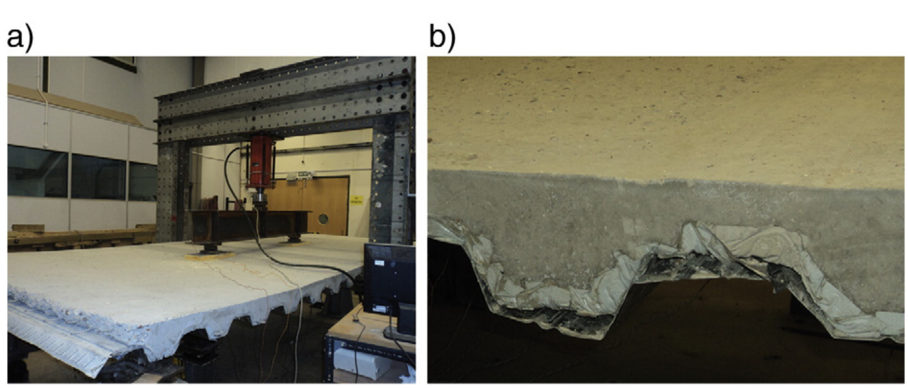

Fig. 17. 5 m specimen a) initially, with loading rig; b) showing decking delaminating from slab.

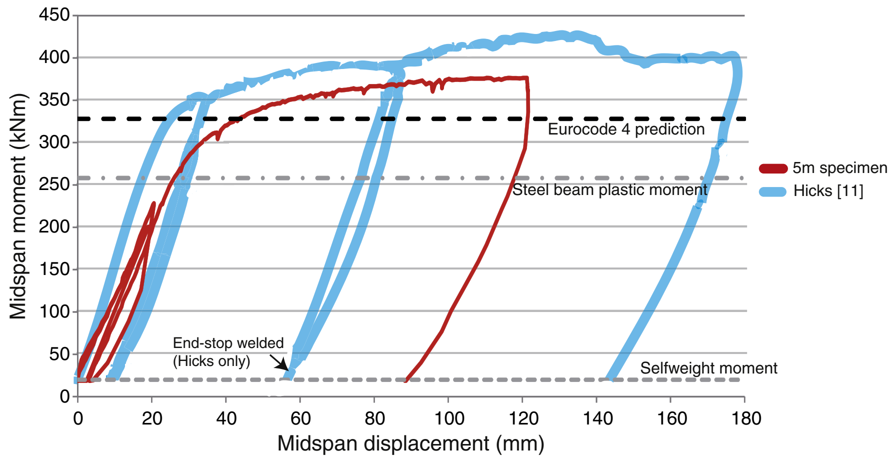

Fig. 18. Moment vs. displacement for 5 m specimen and comparison with published values from Hicks [11].

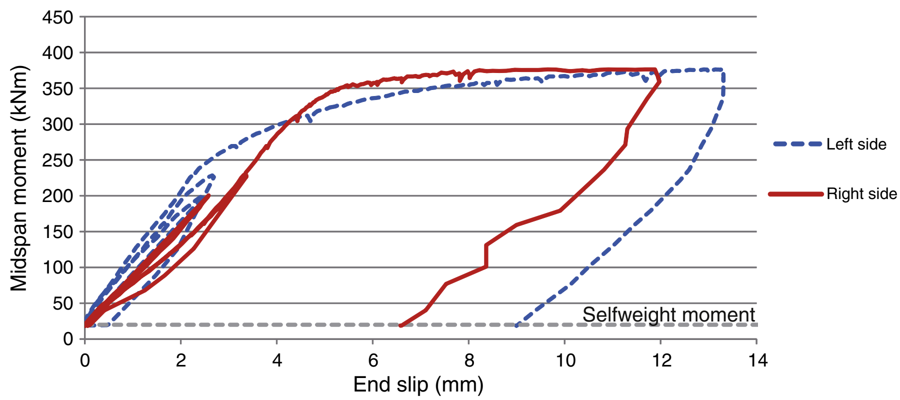

Fig. 19. Midspan moment vs. end-slips for 5 m specimen.

steel re-use points also to the potential to reuse concrete slabs, giving further emissions savings. However there are additional challenges in handling and verifying such re-used slabs, and further examination of this opportunity is required.

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