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Link to publisher's version: *https://doi.org/10.1016/j.jcsr.2016.03.021*

Citation: Rehman N, Lam D, Dai X and Ashour AF (2016) Experimental Study on Demountable Shear Connectors in Composite Slabs with Profiled Decking. Journal of Constructional Steel Research. 122: 178-189.

Experimental Study on Demountable Shear Connectors in Composite Slabs with Profiled Decking

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Abstract

This paper presents an experimental study on shear strength, stiffness and ductility of demountable shear connectors in metal decking composite slabs through push-off tests. Twelve full-scale push-off tests were carried out using different concrete strength, number of connectors and different connector diameter. The experimental results showed that the demountable shear connectors in metal decking composite slabs have similar shear capacity and behaviour as welded shear studs and fulfilled the minimum ductility requirement of 6mm required by Eurocode 4. The shear capacity was compared against the prediction methods used for the welded shear connections given in Eurocode 4, AISC 360-10, ACI 318-08 and method used for bolted connection in Eurocode 3. It was found that the AISC 360-10 method overestimated the shear capacity while the ACI 318-08 method underestimated the shear capacity of specimens with single shear connector per trough. The Eurocodes method was found to provide a safe prediction for specimens with single and pair demountable connectors per trough. In addition, prediction methods given in both AISC 360-10 and ACI 318-08 for welded shear studs overestimated the shear capacity of specimens with 22 mm diameter demountable connectors that failed in concrete crushing.

Key Words: Demountable shear connectors, Push-off tests, Metal deck flooring, Shear capacity, Ductility, Stiffness

1 Introduction

The increasing rate of carbon emission into the environment has highlighted the issue of sustainability and reuse of materials. This has led to research on the reuse of steel beams in composite construction. Steel-concrete composite beams are a cost effective construction system for multi-storey buildings owing to the composite action between steel beams and composite slabs. In the current construction practice, composite action is achieved through shear studs welded through the profiled sheeting to the steel beam flange and embedded in the concrete slab. Therefore, when composite structures reach the end of their design life, these welded shear connectors make dismantling, adaptation (alteration) and deconstruction of the composite structures almost impossible. In the current practice, steel beams have to go through a recycling process and cannot be re-used straightaway. The recycling process requires a significant amount of energy and produces carbon emission into the environment.

Therefore in this research, a new form of demountable shear connectors is used as an alternative to welded connectors in composite beams. This demountable connector would allow the steel beam to be reused at the end of the structural design life without the need of the recycling process. Bolted shear connectors have been rarely used in construction, apart from rehabilitation work. Unlike welded shear connections, the demountable shear connectors are easy to dismantle, enabling the steel beam to be reused without a recycling process. In addition, a demountable shear connector can be easily installed on site into the predrilled flange of the steel beam and steel profiled decking. The demountable shear connectors have not been widely adopted in construction practice and no design guidance is currently available. Therefore in this study, a number of push-off tests were carried out to assess the capacity of this form of shear connectors.

Eurocode 4 [1] provides a simple procedure for push-off test and equations to predict the shear capacity of shear studs in composite beams. However the push-off test details provided in Eurocode 4 are for welded shear connectors in solid concrete slabs. Mottram and Johnson [2] suggested a geometric adjustment to the standard push-off test for welded headed stud connectors in metal decking slabs. Pavlovic *et al*. [3] studied the M16 Gr8.8 bolted shear connector through push-off tests in solid slabs and compared the experimental results with welded headed shear studs in solid slabs. It was found that the Gr8.8 bolted shear connectors with a single embedded nut achieved about 95% of the shear resistance under static loads, but the stiffness reduced by 50% as compared with the welded headed stud. However, the focus of their research was only focussed on solid slabs with high strength bolts. This is quite different from this research as demountable headed shear connectors are used with cast in-situ metal deck composite slabs. Dai *et al.* [4] performed a series of push-off tests using demountable shear connectors with solid slabs and concluded that the demountable shear connector has a slip of up to 20mm before failure and the shear capacity was about 84% of the welded shear connector at the slip of 6 mm.

Dallam [5] and Marshall *et al.* [6] investigated the high strength friction-grip (HSFG) bolts. But their main purpose was to investigate the pre-tension behaviour of high friction grip bolts. Dedic and Klaiber [7] and Kwon *et al.* [8] investigated the shear capacity and performance of post installed bolted shear connectors under fatigue loading. The focus of their research was to strengthen the existing non-composite buildings using high strength friction grip bolts as shear connectors. Pathirana *et al.* [9] and Mirza *et al.* [10] carried out research on demountable studs using blind bolts. It was found that blind bolts behaved very similar to welded headed studs in terms of stiffness and strength but the blind bolt had a relatively brittle behaviour. Henderson *et al.* [11] discussed different types of shear connection under dynamic loading and reported that the removable shear connectors had very similar stiffness and strength as welded headed studs in composite beams. Hawkins [12] tested the anchor bolts without embedded nuts in a solid slab using different lengths and diameters of the bolts. It was found that the shear capacity of these anchor bolts was about 80% of the welded shear connectors. Ollgaard *et al.* [13] carried out extensive studies on welded shear connectors. Oehlers and Bradford [14] discussed different types of connectors used for composite beams including the bolted and demountable connectors. It was concluded that the bolts can be attached directly to the flange prior to the casting of concrete; or the concrete slab and the steel beam can be bolted together after casting by using bolts or friction grip bolts. Allwood and Moynihan [15] conducted three composite beam tests using M20 Gr 8.8 bolts; it was found that the Eurocode 4 prediction for the welded shear connector is conservative when compared to their experimental observation.

From the literature review, it is found that previous research on bolted connectors was carried out using high strength bolts with solid slabs. There is little research carried out using metal deck composite slabs. Although a few composite beam tests were carried out with demountable shear connectors using the Gr 8.8 M20 bolts, no push-off tests were carried out using demountable shear connectors with metal deck composite slabs. In this research, a series of push-off tests were carried out to investigate the slip behaviour and the ultimate shear capacity of demountable shear connectors in metal decking composite slabs.

2 Experimental Study

2.1 Test specimens and material properties

To assess the shear capacity, stiffness and ductility of demountable shear connectors, a series of push-off tests, as shown in Figures 1 to 3 with different reinforcement cages were carried out at the University of Bradford. In general, the push-off test arrangements were very similar to the one described in Eurocode 4. It consists of two identical concrete slabs of size 610 × 510 × 150 mm connected through shear connectors as shown in Figures 4 and 5 with a predrilled hole in a steel section (203 × 203 × 52 UB). The test specimens were divided into 6 groups as presented in Table 1. In each group, two replicate specimens were tested. These specimens covered different reinforcement arrangement, concrete strength, connector diameter and types.

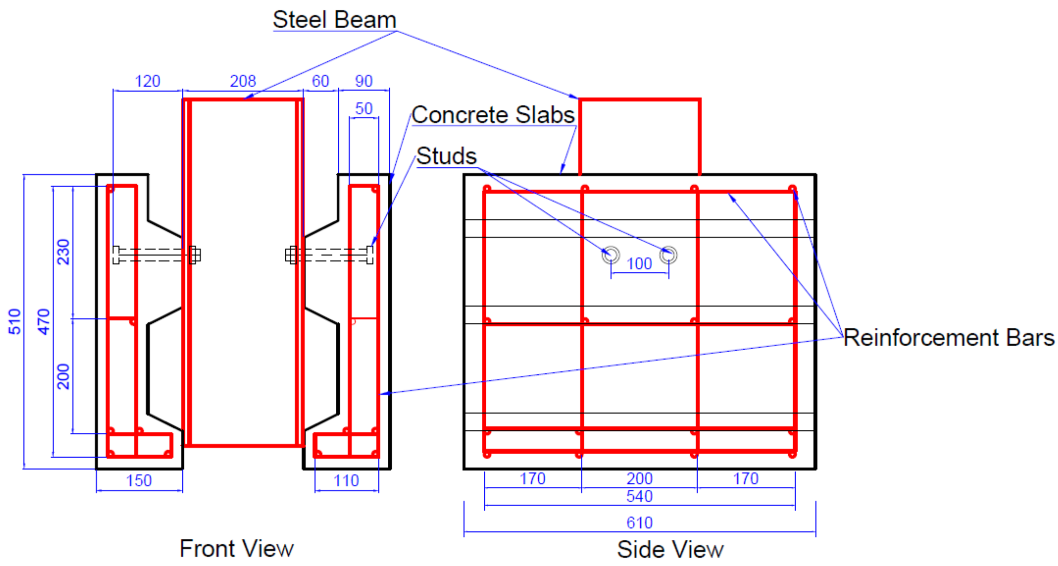

Figure 1. Push-off test specimen with modified reinforcement cage

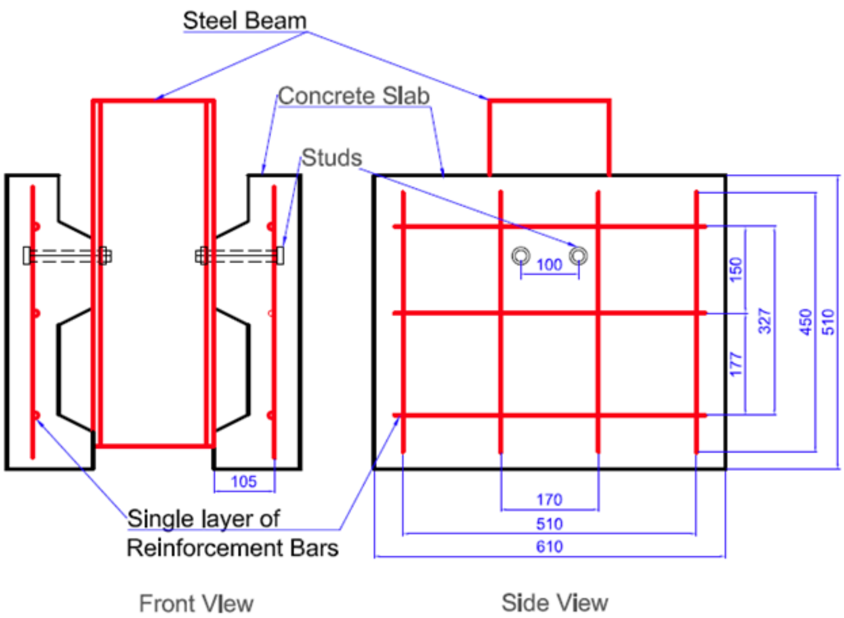

Figure 2. Push-off test specimen with single layer reinforcement cage

Before the improved reinforcement cage (M) as shown in Figure 1 was adopted, single layer of steel reinforcement (S) as shown in Figure 2 was used for push-off test specimens (S1 and

S2), a premature failure due to buckling of the slabs was observed. After these tests, a double layer of reinforcement (D) as shown in Figure 3 was developed and used for specimens D1 and D2 to overcome this premature failure. However the specimens with the two layers of reinforcement developed a local premature failure at the toes of the specimens, yet again preventing the shear connector capacity to be obtained. To prevent this local premature toe failure, the reinforcement cage was modified with toe reinforcement as shown in Figure 1. The main purpose of the reinforcement is to prevent local premature failure due to buckling of the slab or failure to the toe.

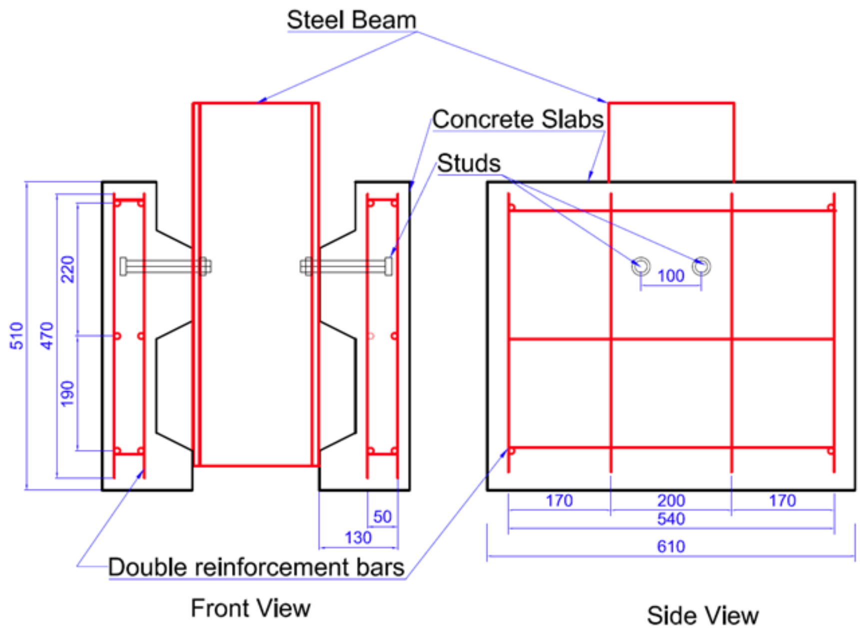

Figure 3. Push-off test specimen with double layer reinforcement cage

The clearance between the hole in flange and collar (shank) of the bolt is 1 mm and 1mm clearance is provided for the hole in metal profile decking. The nuts are tightened using tools used by steel erectors in construction. The nominal height of the connectors in concrete for all the specimens was 120 mm, specimens S1, S2, D1, D2 and M1 to M6 have a shank diameter of 19 mm embedded in the concrete and a 17 mm diameter collar passing through the steel beam flange and a threaded portion of 16mm diameter. The specimen M7 has a shank diameter of 22mm with a collar and a threaded portion of a diameter 20mm. For the specimen M8, a pair of M20Gr 8.8 bolts was used.

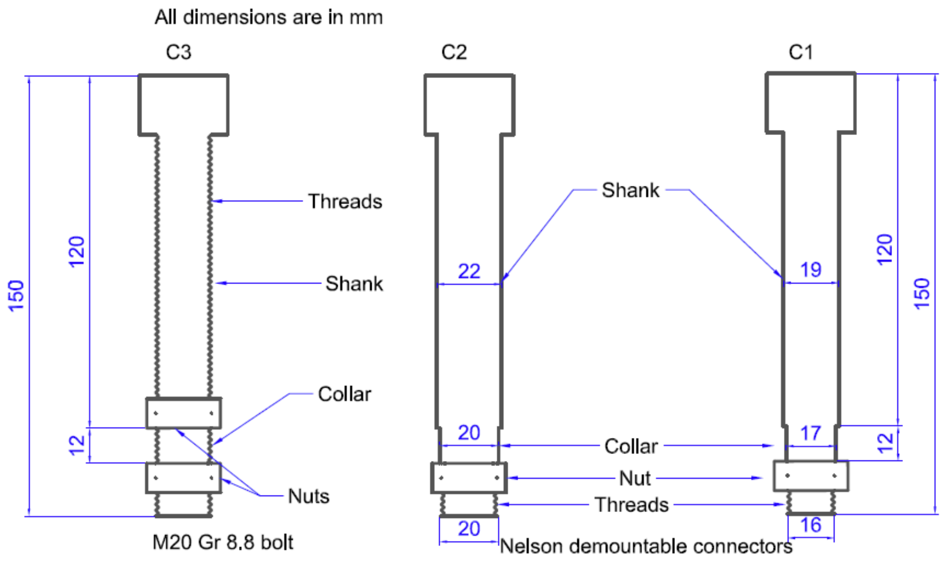

Figure 4. Different type of demountable connectors

Figure 5. Different types of shear connectors

The two composite slabs for each specimen were casted horizontally with the same concrete mix as shown in Table 2 and cured in an open air environment. Ribbed steel bars with a diameter of 10mm were used for the reinforcement cage. The strength of the concrete slab was determined by taking the average strength of six cube specimens cured in the same condition as the tested specimens and tested on the test day.

Table 1. Geometric configuration summary of tested specimens

	Test Specimen ID	Connector type	Reinforcement arrangement ID	Concrete Strength (MPa)	Shear connectors per slab
Group1	S1	C1	S	57.5	2
	S2	C1	S	54.5	2
Group2	D1	C1	D	29.4	2
	D2	C1	D	28.5	2
Group3	M1	C1	M	43.4	2
	M2	C1	M	40.9	2
Group4	M3	C1	M	36.2	1
	M4	C1	M	30.5	1
Group5	M5	C1	M	55.7	2
	M6	C1	M	58.1	2
Group6	M7	C2	M	22.7	2
	M8	C3	M	19.2	2

The transverse spacing between two connectors was 100mm in specimens with two studs per trough. The minimum distance between the shear connectors and the vertical reinforcement cage bar was 50mm. The ultimate strengths of the shear stud connectors and steel reinforcement are 510N/mm^2 610 N/mm^2 respectively. Richard Lees Rib E60 type profiled metal decking with a thickness of 0.9 mm was used with the steel grade of S350.

Table 2. Proportions of different contents in concrete mix design

Group	Cement (kg)	Water (kg)	Fine aggregate (kg)	Coarse aggregate 10mm (kg)	Coarse aggregate 20mm (kg)
Group1	527	195	597	354	707
Group2	286	195	724	395	780
Group3	405	195	640	380	760
Group4	286	195	724	395	780
Group5	527	195	597	354	707
Group6	271	195	690	408	816

2.2 Test setup and instrumentation

Figure 6 shows the test setup of the push-off test. Eight linear variable displacement transducers (LVDTs) were installed at the top of the steel beam and concrete slabs, as shown in Figure 6 to measure the vertical displacements, which subsequently used to calculate the relative slip between the steel beam and the concrete slab. The load versus displacement was recorded by the data logging system.

During the test, the applied load was increased by 5kN interval; at each interval, a further 5 minutes between loading is allowed for the load to settle before the next load increment is applied. When the applied load reached 40% of the predicted failure load based on the Eurocode 4 equations, then the load-control method was changed to the displacement-control method, in which a constant increment rate of 0.2mm/min was adopted until the failure of the specimens was observed, i.e. rapid reduction of the load capacity.

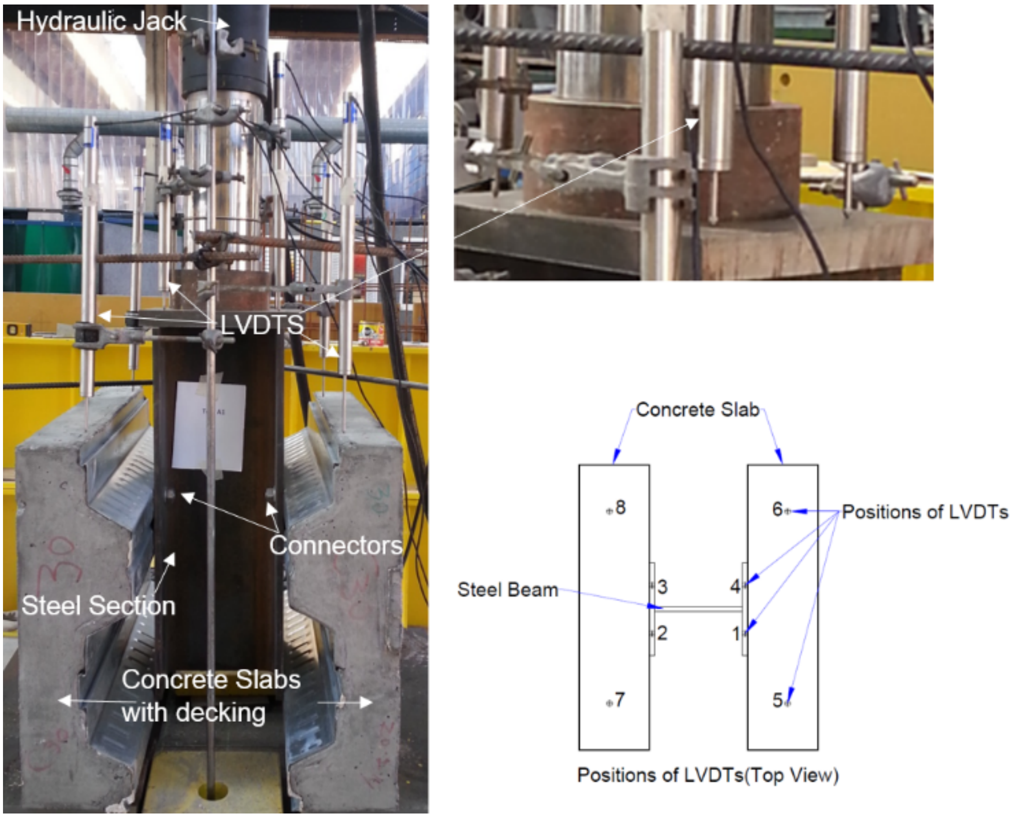

Figure 6. Push-off test set up and monitoring positions

3 Experimental Results

3.1 Modes of failure

Two main failure modes were observed in these tests. The first mode of failure is concrete cone failure with concrete crushing and cracks where no connector fracture was observed. In this type of failure, the concrete around the connector failed in compression before the connector is yielded. Crushing of concrete started from the vicinity of the connector head and cracks developed through the depth of the concrete slab forming a cone shape around the shear connectors as shown in Figures 7 and 8.

Cracks on the outer surfaces of concrete slab were observed in tested specimens. These cracks were more evident in specimens with low strength concrete than those with higher strength concrete. The transverse concrete cracks first appeared on the outer surface of the slabs just near the middle of the slabs as shown in Figure 9, but they did not propagate inside the slab due to the presence of the reinforcement cage.

Figure 7. Concrete pull-out failure mode observed from test specimen M7

Figure 8. Combination of a concrete cone failure and the connector fracture

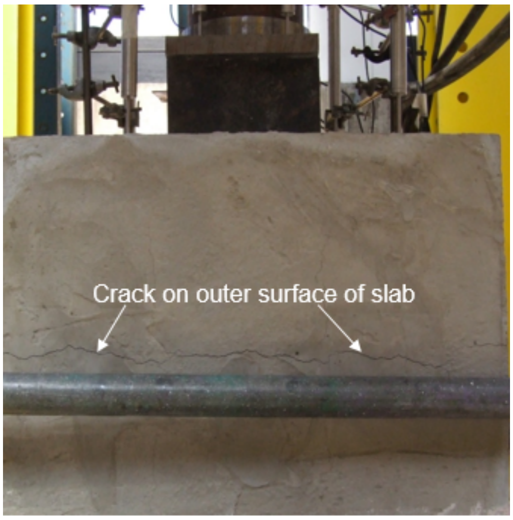

Figure 9. Longitudinal cracks on outer surface of a slab observed from test specimen M6

These cracks are visible when the metal deck was dismantled from the concrete slab after the test. The concrete damage patterns observed from different specimens are very similar irrespective to the shear connector arrangement and the concrete strength. However the concrete cone in the specimens with a single shear connector per trough was less than that in specimens with pair shear connectors per trough.

Figure 8 shows a typical failure mode, which was characterised by the combination of connector fractured, concrete cone and the cracks around the root of the connector due to compressive forces. The average width of the cone was about 260 mm in the specimens with the paired connectors of 19 mm diameter, which is approximately double the bottom width of the slab trough (110 mm) and about 25% wider than the width of the steel beam flange. The average width of the cone was about 130mm with a single shear connector connection, which is half of that of specimens with pair connectors per trough. It was observed that the average cone width was up to 360mm for the specimen with pair connectors of a diameter of 22 mm, which is 100 mm larger than that of specimens with connector diameters of 19 mm. This is possibly due to the shear resistance increase and higher compression applied to the concrete around the demountable shear connectors.

In specimen M2, although the shear connector deformed significantly but it did not fail before the concrete cone failure mode occurred. The concrete cracks propagated longitudinally across the concrete slab and caused rib shearing of the concrete slab as shown in Figure 10. The deformation of the hole in the profiled metal decking was also observed during the test. The tearing of the metal deck at connector hole was more prominent in specimens with a single shear connector compared to the paired connector specimen as shown in Figure 11.

The second mode of failure is that the shear connector fractured at the collar. For this mode of failure, the connector fully yielded and fractured at the collar of the shear connector. The connector reached its maximum yield stress while the concrete slab did not reach its maximum stress. In the paired connector specimens M1, M5 and M6; the connectors sheared off as expected due to the high concrete strength. A similar connector failure was also observed in single shear connector specimens M3 and M4.

Figure 10. Deck de-bonding in left slab and rib shear failure in right slab of specimen M2

Figure 11. Elongation of hole in metal deck with single and pair of connector specimens

Typically fracture occurred at the collar position close to the slab as shown in Figures 12 and 13. The deformation of the shear connectors observed in specimens M5 and M6 with a high concrete strength slab was much smaller than that of specimen M2 with a low concrete strength slab. Figure 14 shows excessive deformation of shear connector of specimen M2.

Figure 12.Stud shear failure observed from specimen M6

Figure 13. Fractured Stud in test specimen M 5

Figure 14. Deformed shape of shear connector in test specimen M2

3.2 Load - slip relationship

Table 3 summarises the maximum shear capacity per connector, amount of slip at maximum load and at failure; and the mode of failure observed in the push-off tests. The load versus slip behaviour of all push-off test specimens is presented in Figure 15. These load-slip curves have a clear elastic and plastic portion. In the elastic region, the load - slip curves show an almost linear relationship for all specimens but in the plastic region the slip increases and stiffness changes rapidly.

The load-slip relationship was recorded at both sides of each push-off specimen. The shear connectors on both sides behaved in a very similar manner as shown in Figure 16 for M1, M2, M4 and M5 specimens. From Figure 17, it can be seen that the initial linear behaviour of pair shear connector (M2) was almost similar to the single shear connector (M3). The shear capacity of both specimens is very similar at the slip of 6 mm.

Table 3. Summary of maximum shear resistance and failure modes

Test Specimen ID	Concrete cube strength (MPa)	Max Shear capacity (kN/stud)	Slip value at Maximum load (mm)	Slip value at failure (mm)	Mode of Failure
S1	57.5	60	5.5	10	Concrete cone failure
S2	54.5	44.5	6.4	9.2	Concrete cone failure
D1	29.4	61.5	5.2	7.8	Concrete cone failure
D2	28.5	42.2	4.3	6.5	Concrete cone failure
M1	43.4	69.9	9.2	21.2	Stud fracture
M2	40.9	68.2	7.2	18.7	Concrete cone failure
M3	36.2	80.0	16	17.9	Stud fracture
M4	30.5	79.6	26	28.2	Stud fracture
M5	55.7	76.1	6.8	10.6	Stud fracture
M6	58.1	82.6	7	9.5	Stud fracture
M7	22.7	66.3	4.1	6	Concrete cone failure
M8	19.2	63.5	5.8	6.6	Concrete cone failure

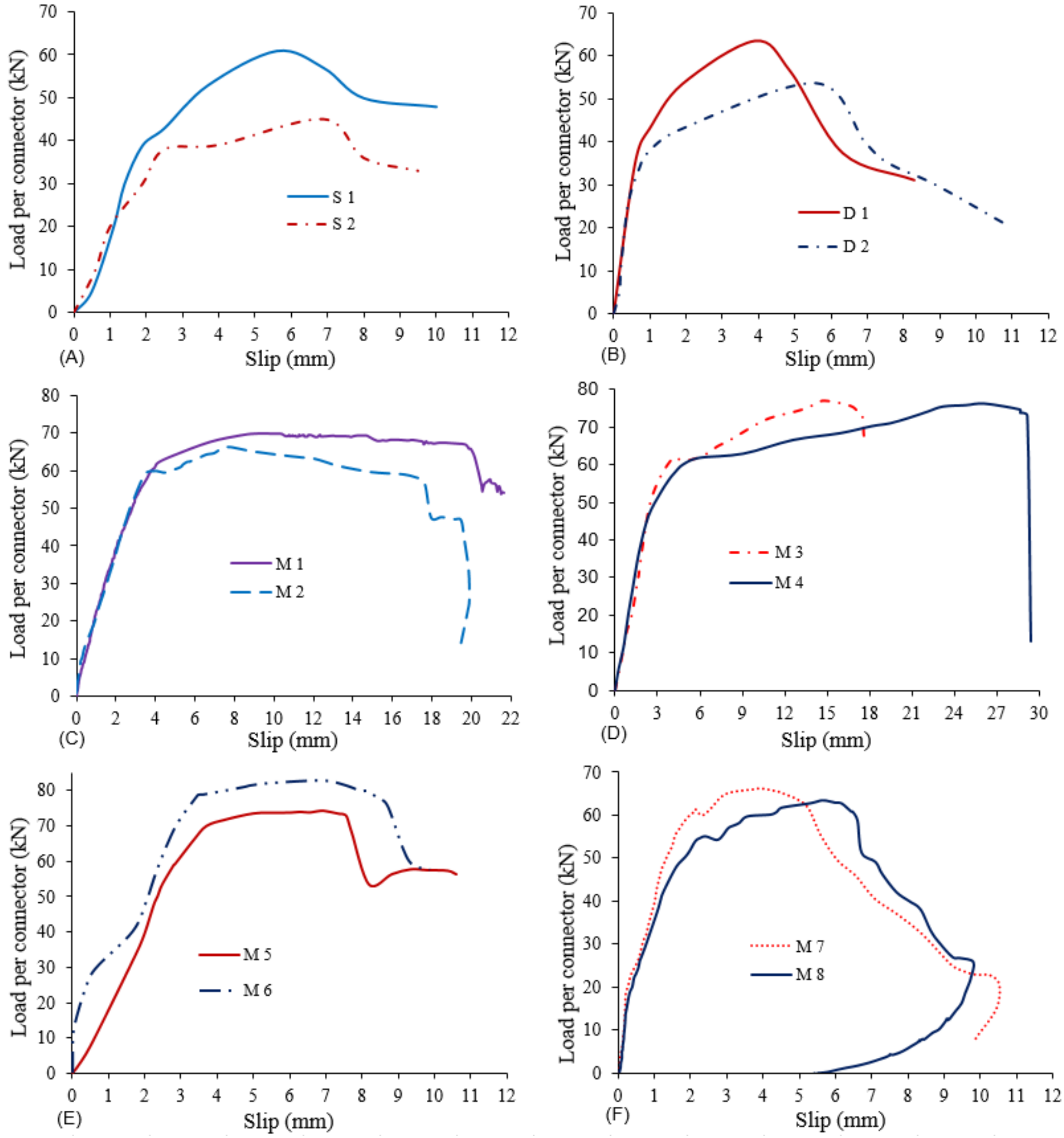

Figure 15. Load-slip curves of 12 push-off test specimens

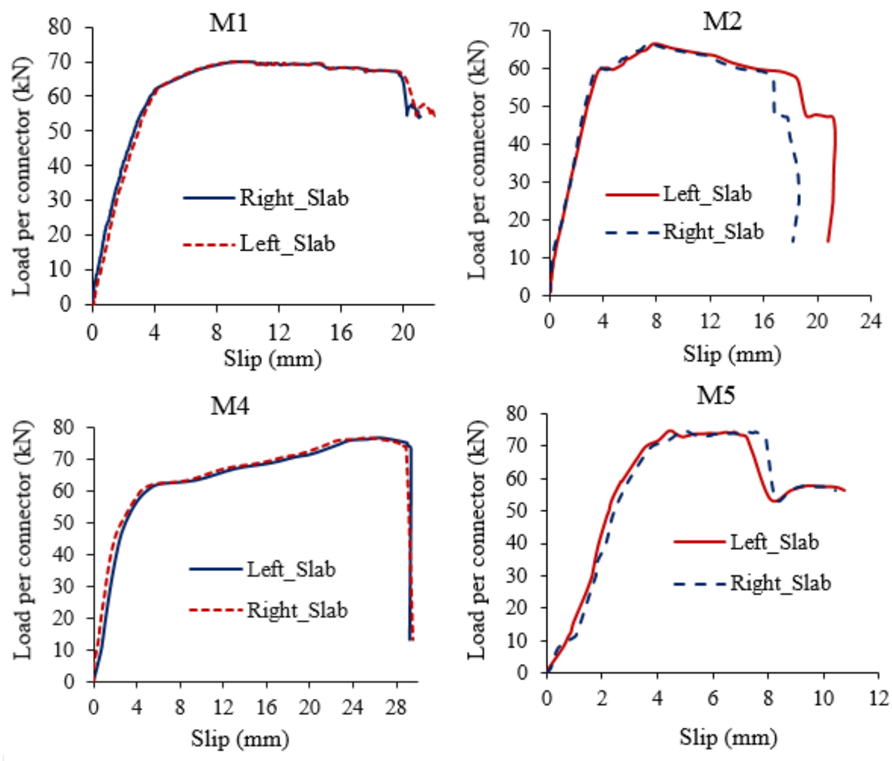

Figure 16. Comparison of typical load-slip behaviour measured in both sides of the same specimen

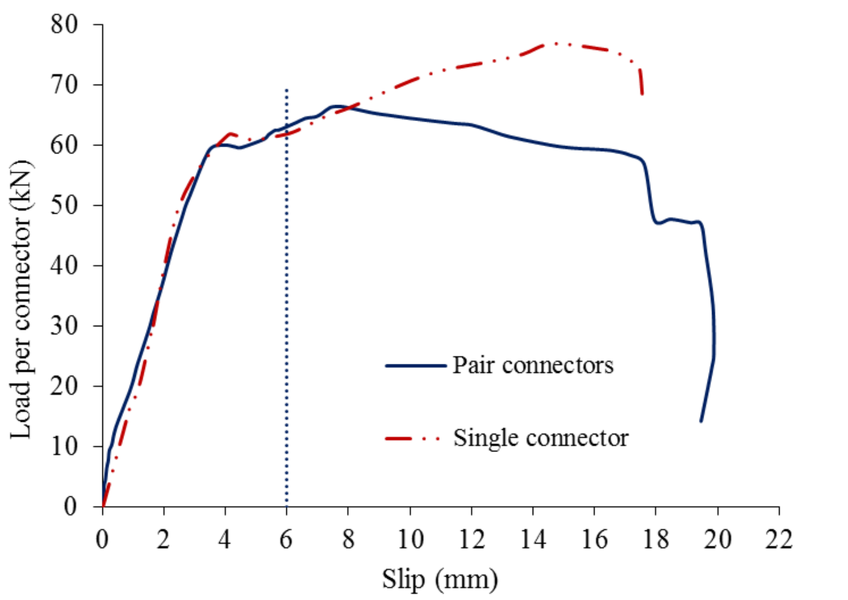

Figure 17. Comparison of specimens with pair connectors per trough and single connector per trough

4 Discussion

4.1 Shear capacity, ductility and stiffness of demountable shear connectors

The highest shear capacity was observed in the paired connector specimen M6 with 82.6 kN per stud owing to its highest concrete strength (58.1 MPa). Similar shear capacity (80.0 kN) of specimen M3 with only one shear connector per trough connecting a slab to the steel beam was also observed. There was no significant difference between the shear capacity (load per connector) of specimens M6 and M3, although the slab concrete strength of the latter was much lower. This is because the concrete resistance was fully developed for specimens with 19 mm demountable shear connector.

Regardless of slab concrete strength, the average shear resistance of 73.5 kN / connector was obtained from the specimens with pair connectors of 19 mm. This is about 5 % lower than the average shear strength of specimens with single shear connector of the same connector diameter. The overall average shear resistance per connector was 75.45kN for all six specimens M1–M6 with the modified reinforcement cage with the same connector diameter and height.

Demountable shear connectors with a shank diameter of 19mm showed very ductile behaviour in this study. The average slip of paired connector specimens, with modified reinforcement cage and connector shank diameter of 19 mm achieved in excess of 6 mm slip at maximum load. In Figure 15 (C and E), it can be seen that the slip at the maximum load for specimens with a pair of shear connectors per trough is in the range from 6 to 10 mm. This fulfils the ductile limit of 6 mm in according to Eurocode 4. The slip of specimens M5 and M6 with the higher concrete strength is less than half of those specimens (M1 and M2) with lower concrete strength. This is due to the effect of the concrete strength.

The specimens (M3 and M4) with only a single shear connector per trough showed a very ductile behaviour, see Figure 15 (D). The slip at maximum load was up to about 16 mm in specimen M3 and about 26 mm in specimen M4. The load bearing capacity increased with the increase of slip. After achieving the maximum shear resistance, the slip did not increase significantly before the connector sheared off as the connectors were fully yielded at this point.

4.2 Effect of concrete strength

Concrete strength has a significant effect to the shear resistance of demountable shear connections. As shown in Figure 18, the maximum shear resistance of specimen M6 was about 17% higher than that of specimen M1 due to the higher concrete strength of the specimen M6. The concrete strength of M6 was about 35% higher than that of M1. The strength of concrete also affected the mode of failure. The specimens (M5 and M6) with high concrete strength failed with a brittle failure mode with a slip of about 6 and 7 mm at maximum load although this fulfilled the ductility limit of 6 mm. The shear connector fractured without significant concrete crushing. The connector deformation was not as big as for the connector embedded in the lower concrete strength specimens (M1 and M2). The failure mode also changed from connector fracture to concrete failure in the paired shear connector specimens as concrete strength decreases.

In the case of a single connector per trough specimens (M3 and M4), the increase of concrete strength did not have an evident effect on the shear capacity of demountable shear connections as shown in Figure 14 (D). The maximum shear resistance is very similar and both failed with connector shearing fracture although the difference in concrete slab strength was about 15%. The stiffness increased about 23% in test specimen M7 as compared to test specimen M8 as the concrete strength increased about 26%. Similar behavior was observed in test M1 when compared to M5. Stiffness increased about 25% as concrete strength increased about 28%. These behaviors are shown in Figure 19. This shows that the stiffness increases with the increase of concrete strength. Moreover, when the load reached about 70% of the maximum load capacity (P) of a shear connector, the stiffness of the shear connectors starts to decrease.

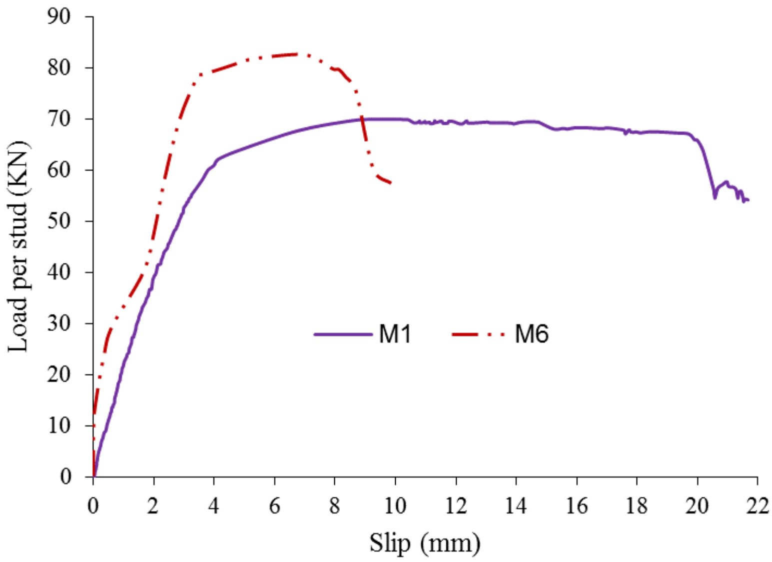

Figure 18.Comparsion of high concrete strength and low concrete strength

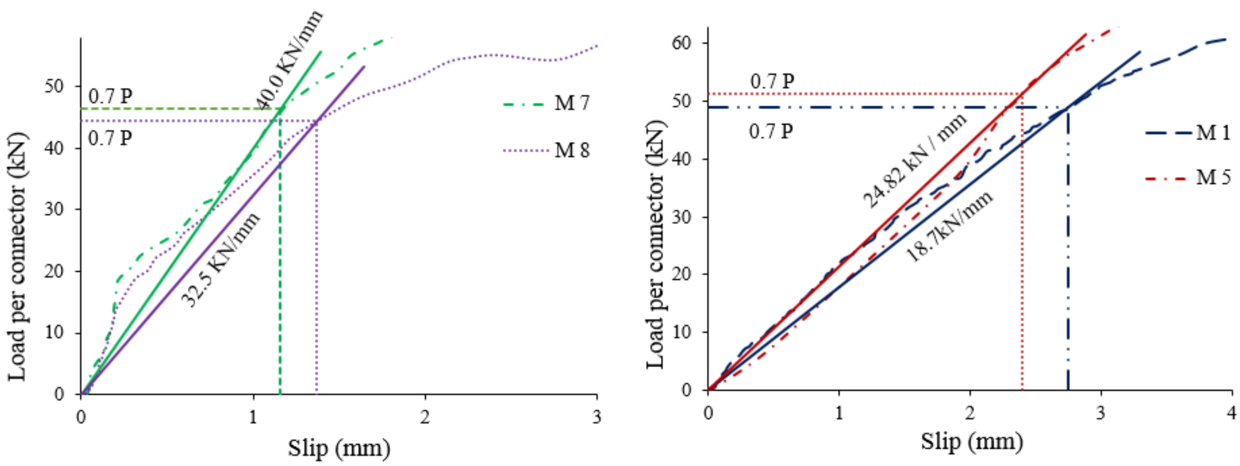

Figure 19. Stiffness of shear connectors

4.3 Effect of number of connectors

Figure 20 compares the effect of number of connectors per trough. It was found that the reduction in shear strength with paired connectors was about 13% although the concrete strength was lower in single shear connector specimens. This is possible because the shear strength of connectors can't be fully developed with two connectors per trough. Previous research by Qureshi *et al.* [16, 17] showed that the spacing between connectors have direct effect to the shear strength of the connectors. Figure 20 shows that the connection with two shear connectors per trough has slightly better ductility while the single shear connector specimen had higher shear resistance per connector as suggested by Qureshi *et al.* [16, 17]

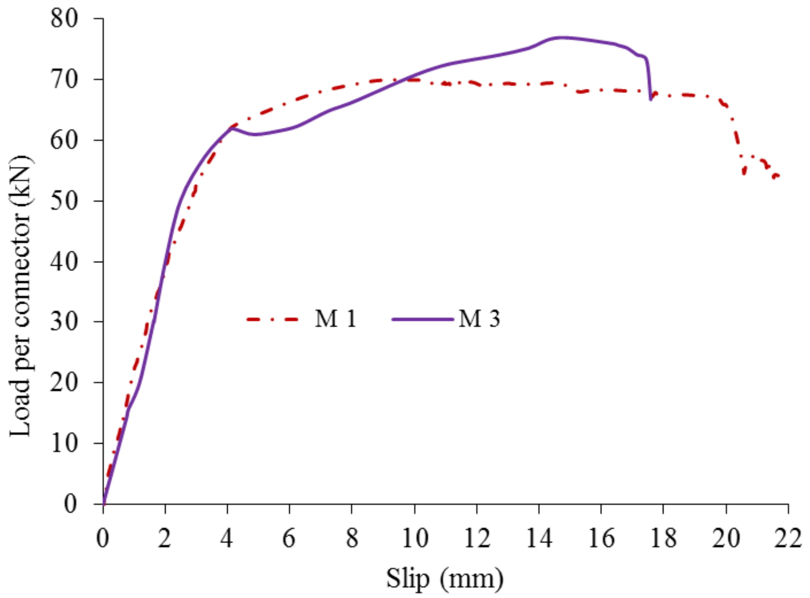

Figure 20. Comparison of specimens with single connector per trough and pair connectors per trough

4.4 Effect of Reinforcement

The experimental results show that the specimens (M1 to M8) with the modified reinforcement had higher capacity than those specimens with single and double layers of reinforcement. The increases in capacity was about 75% of the single layer specimen S1 and

S2 and was about 65% of the double layer specimen D1 and D2. However, the high percentage difference was due the effect of premature failure of the concrete slabs.

4.5 Comparison of Gr 8.8 bolt and demountable headed stud shear connector

When comparing the load - slip behaviour of the M20 Gr 8.8 bolt shear connectors with the demountable shear connectors with a collar diameter of 20mm, as shown in Figure 15 (F). It can be observed that both specimens behaved in a very similar way and failed with concrete cone failure. Due to the failure mode, the strength of the connectors did not make any effect on the shear capacity.

4.6 Comparison with results from other researchers

The shear capacity of a demountable shear connector is slightly higher than the capacity of a similar welded shear connector [18] as shown in Figure 21. The ductility of the demountable shear connector is found to be better than that of the welded shear connector as shown in Figure 21, but the initial stiffness is much lower. The difference was possibly caused by the clearance hole in the steel beam for the demountable shear connector and the initial slip. However the stiffness might be increased by reducing the clearance in the hole or pre-tensioned the connector. The increase in stiffness observed in test specimens M7 and M8 to M1 and M5 is due to the increases in connectors' diameter. From Figure 19, it can be seen that the stiffness of M7 and M8 (40 and 32.5 kN/mm) increased by about 74% and 31% as compared to the stiffness of M1 and M5 (18.7 and 24.82 kN/mm).

The low initial stiffness of the connectors could be due to the torque applied to these demountable shear connectors was not enough to develop the friction forces between the interface of steel beam and metal decking concrete slab. The shear connection was not able to transfer the initial shear forces at interface through friction. The demountable shear connectors started resisting the applied load in bearing. The other factors could be the oversized hole in steel flange and metal decking slab reduce the stiffness at low load levels. But the observed load slip behaviour of the connectors was very stable initially as well as after reaching at the yielding point in specimens (M1-M6).

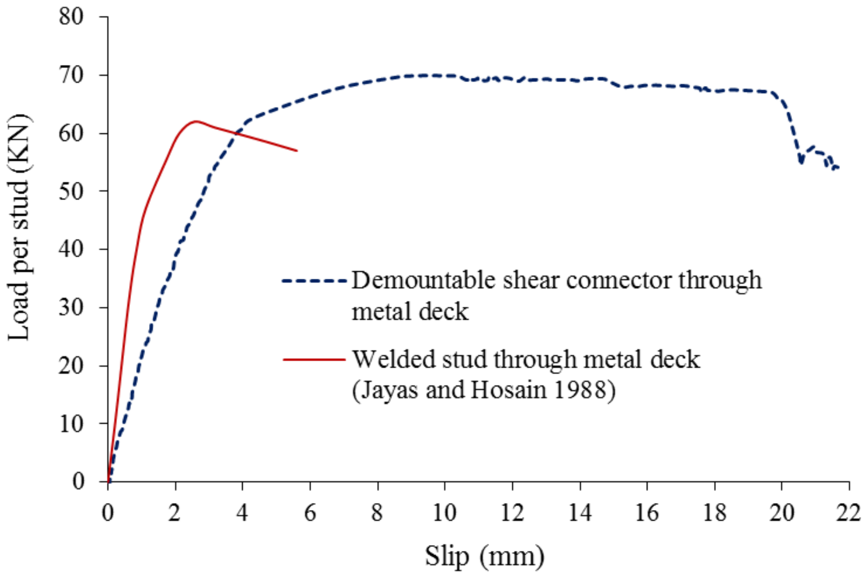

Figure 21. Comparison of demountable shear connector with welded shear connector

Figure 22 compares the load-slip relationship of a demountable shear connector in metal decking composite slab (current research) with a similar shear connector in a solid concrete slab [4]. It is found that the shear resistance capacity, ductility and stiffness behaviour of a demountable shear connector embedded in a concrete solid slab and embedded in a metal decking slab are very similar except that the ultimate strength of demountable connectors in a solid slab is about 12% higher than that in a metal deck slab. This is because of higher concrete confinement around the shear connectors as solid slabs do not have troughs like a metal deck slab.

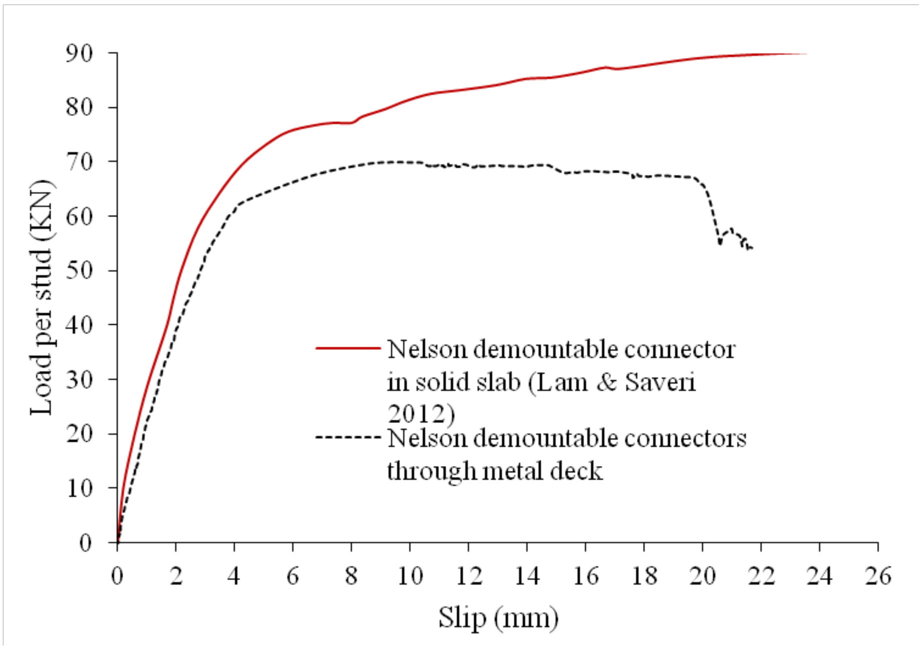

Figure 22. Comparison of demountable shear connector in solid slab and metal decking slab

Figure 23 shows a comparison of current research and the push-off experiment results conducted by Pavlovic *et al.* [3] using Gr8.8 bolts with embedded nuts in solid slabs. It is found that the Gr8.8 bolted stud with embedded nuts in a solid slab has very stiff behaviour and the maximum resistance is approximately 25% higher than that of the demountable shear connector in a metal deck slab. This is due to the stiffness of the high strength bolt and embedded nuts in a solid slab. But the slip responding to the maximum load is about 4.5 mm and is very small compared to the slip behaviour of a demountable shear connector in a metal deck slab.

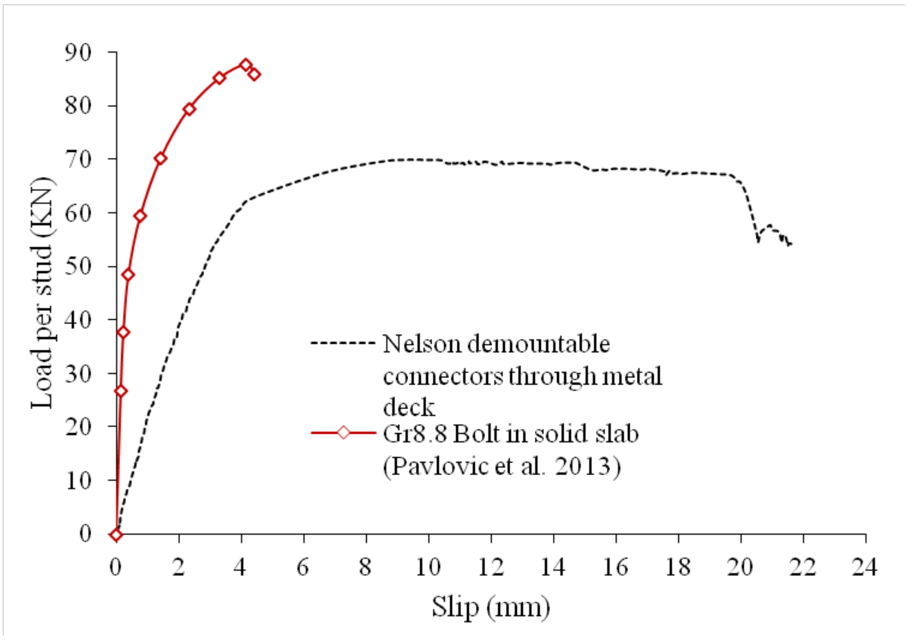

Figure 23. Comparison of demountable shear connector and Gr 8.8 bolt connector

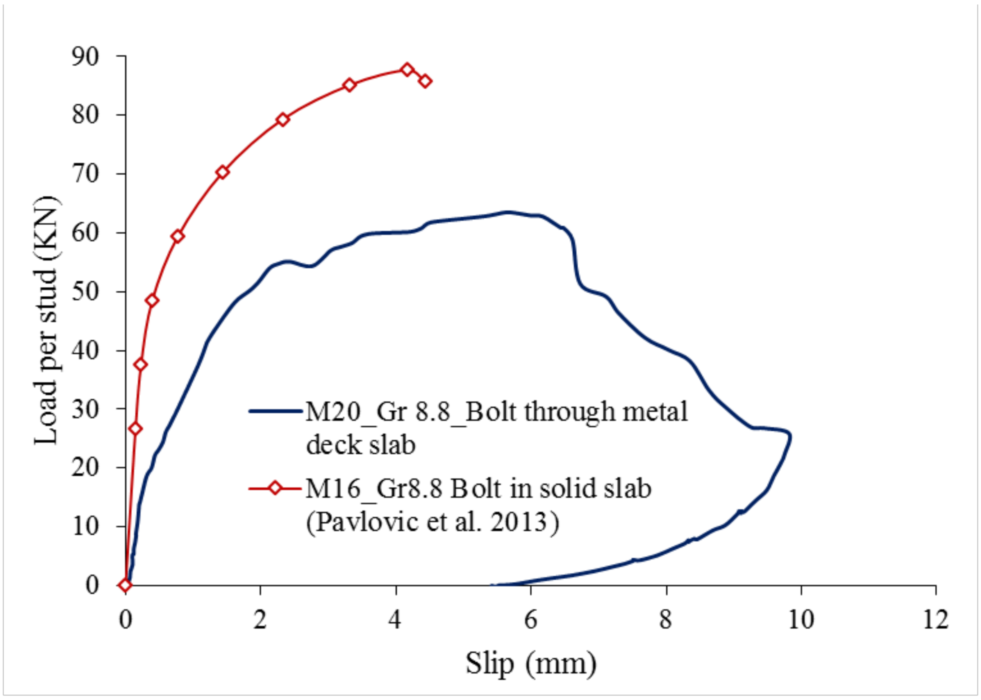

Figure 24. Comparison of M20 Gr 8.8 bolt connector in metal deck slab and M16 Gr 8.8 bolt connector in solid slab

Figure 24 presents a comparison of load-slip of Gr 8.8 bolts in a metal deck slab (current study) and M16 Gr8.8 bolt in a solid slab [3]. Solid slabs with Gr 8.8 bolts behave in a stiffer manner than the demountable stud in a metal deck slab and with higher shear capacity although the bolt diameter was smaller. This is because the specimens with a metal deck had lower concrete strength and the failure mode was due to concrete cone failure.

5 Design rules for demountable shear connectors

Currently there is no specific assessment method available for demountable shear connectors. The methods available for headed shear connectors in Eurocode 4, Eurocode 3 [19], AISC360-10 [20] and ACI 318-08 [21] are summarised in Table 4 and are used to predict the shear capacity of demountable shear connections.

Table 4. A review of design codes

Codes	Expression	
EC4	$P_{rd,C} = 0.29\alpha d^2\sqrt{f_{ck}E_{cm}} \,/\, \gamma_v$	(1)
	$P_{rd,S} = 0.8f_u\frac{\pi d^2}{4} \,/\, \gamma_v$	(2)
	$K_t = \frac{0.7}{\sqrt{n_r}}\frac{b_o}{h_p}\left[\frac{h_{sc}}{h_p} - 1\right] \leq 0.85 \text{ for } n_r = 1 \text{ and } 0.7 \text{ for } n_r = 2$	(3)
EC3	$F_{v,Rd} = \alpha_v f_{ub} A$	(4)
AISC 360-10	$Q_n = 0.5A_{sa}\sqrt{f_c^{/}E_c} \leq R_g R_p A_{sa} f_u$	(5)
ACI 318-08	$P_{Rd,S} = \varphi A_{se} f_{ut}$	(6)
	$P_{Rd,C} = k_{cp}\, k\sqrt{f_c^{/}}\ (h_{ef})^{1.5}$	(7)

With the Eurocode prediction, a combination of Eurocode 4 and 3 are used to predict the shear capacity of the demountable shear connectors in this study. The use of these equations is illustrated in Figure 25 according to different failure modes as shown in Figures 7, 8, 10, 12 and 13.

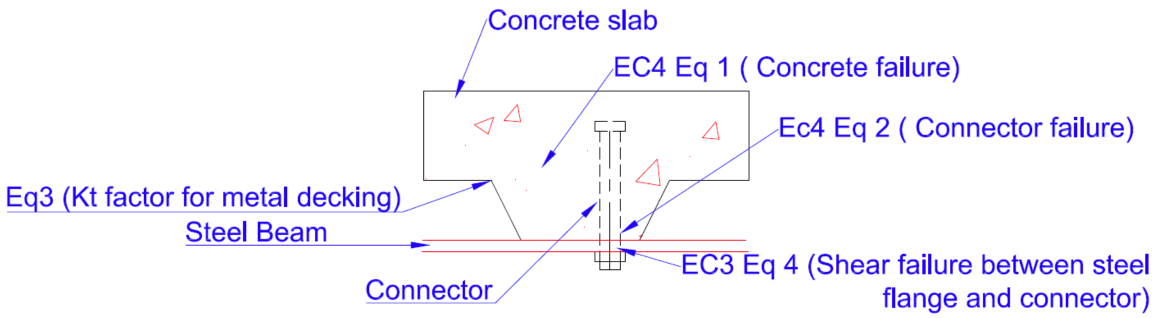

Figure 25. Application of EC4 and EC3 on different failure modes

5.1 Comparison of the coded methods

Table 5 summaries the comparison of the experimental results and predictions using the method provided in Eurocodes 4 and 3, AISC 360-10 and ACI 318-08. It can be seen that Eurocodes slightly underestimated the shear resistance of demountable shear connectors regardless of shear connector failure and concrete failure in specimens with a modified reinforcement. The AISC code overestimated the shear resistance of demountable shear connectors in both concrete and connector shear failure, except in the case of specimens with high concrete strength, which is about 5% lower than the experimental value of the shear resistance of the demountable connector. The ACI 318-08 method was found to be conservative for high strength concrete and for a single shear connector per trough specimens and slightly overestimated the capacity for lower concrete strength specimens with two connectors per trough.

According to the comparison and analysis, the Eurocodes were found to be more accurate about the failure mode prediction than the AISC 360-10 and ACI 318-08. Currently, no standard provides any design guidance for this form of demountable shear connectors and a combination of Eurocode 3 and 4 is recommended by the authors to be used to predict the shear resistance of demountable shear connectors with a reasonable accuracy.

In AISC 360-10, the reduction factor is directly proportional to the product of Rg and Rp. The value of Rg = 0.85 for paired connectors is provided in the code which is used for the demountable shear connectors. It was observed in experimental results that the shear capacity of paired connectors specimen is about 87% of the single connector specimens. The experimental results show that the shear capacity of specimens with single connector per trough is about $0.7A_sF_u$ here A_s is the area of a shear connector and F_u is the ultimate strength of the shear connectors. Where as $0.67A_sF_u$ for high concrete strength with a pair of shear connectors specimen (M5).

Table 5. Comparison of test results with different code's predictions

ID	Experiment		Combine EC4+3					AISC				ACI			
	P_{Test} (kN/Stud)		P_{Rd} (kN/Stud)					Q_n (kN/Stud)				P_{Rd}			
	Max. Load	Failure	$P_{rd,C}$	$P_{rd,S}$	$F_{v,Rd}$	P_{Rd}	P_{Rd} /P_{Test}	$Q_{n\ C}$	$Q_{n\ S}$	Q_n	Q_n/P_{Test}	$P_{Rd,C}$	$P_{Rd,S}$	P_{Rd}	P_{Rd}/P_{Test}
S1	60	Concrete	94.9	90.8	**68.1**	68.1	1.13	178.0	**72.3**	72.3	1.20	427.9	**73.8**	73.8	1.22
S2	44.5	Concrete	91.8	90.8	**68.1**	68.1	1.5	171.0	**72.3**	72.3	1.62	416.6	**73.8**	73.8	1.65
D1	61.5	Concrete	**62.6**	90.8	68.1	62.6	1.01	107.6	**72.3**	72.3	1.17	306.0	**73.8**	73.8	1.19
D2	42.2	Concrete	**54.6**	90.8	68.1	54.6	1.29	105.1	**72.3**	72.3	1.71	273.5	**73.8**	73.8	1.74
M1	69.9	Stud	79.6	90.8	**68.1**	68.1	0.97	144.2	**72.3**	72.3	1.03	371.7	**73.8**	73.8	1.05
M2	68.2	Stud	76.8	90.8	**68.1**	68.1	0.99	137.9	**72.3**	72.3	1.07	360.9	**73.8**	73.8	1.09
M3	80	Stud	77.8	90.8	**68.1**	68.1	0.85	125.8	**85.1**	85.1	1.06	311.6	**73.8**	73.8	0.92
M4	79.6	Stud	86.4	90.8	**68.1**	68.1	0.85	110.6	**85.1**	85.1	1.06	339.5	**73.8**	73.8	0.92
M5	76.1	Stud	93.0	90.8	**68.1**	68.1	0.89	173.8	**72.3**	72.3	0.95	421.1	**73.8**	73.8	0.96
M6	82.6	Stud	95.6	90.8	**68.1**	68.1	0.82	179.4	**72.3**	72.3	0.87	430.1	**73.8**	73.8	0.89
M7	66.3	Concrete	**59.3**	150.8	113.1	59.3	0.89	**118.8**	120.2	118.8	1.79	268.8	**122.5**	122.5	1.84
M8	63.5	Concrete	**53.6**	209.5	117.6	53.6	0.85	**86.6**	392.2	86.6	1.36	**247.2**	399.9	247.2	3.89
W1*	61	Concrete	**58.9**	90.8	68.1	58.9	0.97	99.8	**72.3**	72.3	1.18	291.1	**73.8**	73.8	1.2
Average							0.97				1.2				1.4
COV							0.19				0.23				0.56

W= welded stud, * Ref [18]

6 Conclusions

Twelve push-off tests have been conducted to investigate the shear strength, ductility and stiffness of the demountable shear connections in profiled metal deck composite slabs. The following conclusions may be drawn:

(1) The demountable shear connections have high ductility and similar shear capacity and behaviour compared with their equivalent welded shear connectors although the initial stiffness is slightly lower.

(2) The shear connector arrangement affects the shear connection's behaviour. Connection with a single connector per trough allows the development of full shear resistance of the connector, but the specimen with two connectors per trough provides better ductility.

(3) Concrete strength affects the behaviour of the demountable shear connectors. It appears that the ultimate shear resistance increases with the increase of concrete strength, however the connector's ductility decreases.

(4) Similar to the welded shear connectors, demountable shear connectors have two main failure modes: connector fracture and concrete crushing.

(5) Experimental results showed that a combination of Eurocode 3 and 4 could be used to predict the shear capacity of demountable shear connectors accurately. The AISC and ACI codes may be used to assess the shear capacity of demountable shear connectors for connector failure mode. The reduction factor Rg in AISC 360-10 is only appropriate for pair demountable shear connector specimens.

(6) The use of the modified reinforcement has improved the splitting resistance of a concrete slab and overcomes the possibility of premature failure of the concrete slabs. Therefore it is recommended to use for all push-off tests.

(7) The demountable headed shear connectors have a good potential to be used as an environmental friendly alternative to the welded headed studs in profiled metal deck composite slabs, which will allow the steel beam to be reused after dismantling.

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