Shear Connection of Prefabricated Slabs with LWC - Part1: Experimental and Analytical Studies

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ABSTRACT

This paper studies the shear strength and load-slip behaviours of a recently developed novel steel-concrete composite flooring system (PUSS) with two different types of shear connectors while also using lightweight concrete. PUSS consists of a T-ribbed lightweight concrete floor and C-channel steel edge beams. The proposed shear connection system is using either web-welded shear studs only, or with horizontally lying steel dowels too. This unique system further minimises its structural depth and results in ultra-shallow floors.

Eight full-scale push-out tests were conducted to investigate the connection under the direct shear force with three different concrete types (normal concrete, lightweight and ultra-lightweight concretes) and two different shear connection systems (web-welded shear studs only and horizontally lying steel dowels together with web-welded shear studs). Three types of failure were recorded from the push-out tests; shear failure with bending near the roots of the connectors, shear failure of the weld toe of shear studs, and concrete cracking. Amongst the conclusions, it was validated that the compressive strength of the concrete significantly influences the ultimate shear strength capacity loads while it is changing the failure mode of the connection. The failure mechanisms of the shear connectors were extensively studied, which led to the development of a calculation method for the shear strength and load–slip behaviours of the new connectors embedded in lightweight concrete. The analytical results are compared with those predicted by modern codes and available methods from the literature. It is concluded that the proposed formulae offer a reliable prediction.

Keywords: push-out tests; steel-concrete composite slabs; ultra-shallow flooring system; shear studs; lightweight aggregate concrete; steel dowels

1. INTRODUCTION

The use of lightweight concrete in structural applications for sustainable design of composite slabs require the revision of today's flooring systems and the development of more efficient shear connectors. Evolution of flooring systems during the past decade has resulted moving to the traditional downstand steel beams with the concrete siting on the top steel flange and forming a steel-concrete composite beam to a lighter, shallower, and often aka a 'plug' composite system where the concrete slab sits at the bottom flange of the steel beam and is confined within the two flanges (Ahmed and Tsavdaridis, 2019; Tsavdaridis, 2010; Tsavdaridis et al, 2009a,b; Tsavdaridis et al., 2013; Huo et al., 2010). Research on such shallow flooring systems expands on their vibration performance due to their thin and wide nature (Tsavdaridis and Giaralis, 2011; Kansinally and Tsavdaridis, 2015), and also on their fire performance which can change a lot due to the new system in which the steel is partially protected by the concrete (Maraveas et al., 2017a,b; Alam et al., 2018a-d).

Different flooring systems with integrated building services are beneficial for the use in residential and office buildings, malls and airport structures. However, design rules for such flooring systems have not been included in the European codes (Schäfer, 2015). Additional guidelines for shallow flooring systems should be considered for ultimate limit state, serviceability limit state, and fire design. The plastic moment resistance, the influence of transverse bending moments and the characteristics in erection state for the cross section classification for shallow flooring systems have been presented by (Schäfer, M., 2015). One of the most recent marketed shallow flooring system is the Slim-floor beam. This type of flooring system consists of a rolled or a welded steel profile which is completely or almost completely integrated into the ceiling (Schäfer and Braun, 2019). The construction method of this type of shallow flooring system and its development is presented by (Schäfer and Braun, 2019).

Sustainability needs have led to the development of other innovative integrated floor slabs which enable wide spans and building services to be integrated through the floor height (Hegger et al., 2013). These integrated floor slabs require large web openings in the structural elements (Dressen et al., 2015). The influence of openings on the load-bearing capacity and deformation behaviour of double-T-shaped concrete beams with prestressed tension chords were investigated using six beam tests. The test parameters were the amount of vertical reinforcement at the edges of the opening, the concrete strength, and the location of openings in the longitudinal direction (Dressen et al., 2015). The load-carrying capacity of concrete beams with openings can achieve approximately the same load-carrying capacity of the concrete beams without openings using proper arrangement and dimensioning of reinforcement. The load bearing of an integrated composite floor system has been investigated by Gallwoszus et al. (2014). This type of floor is a novel multifunctional flooring system which integrates building services and technical installations within the structural element. Therefore, the presence of large openings in the structural elements' webs is required. The load bearing structural element within the innovative flooring system is represented by the prestressed steel-concrete hybrid beams with single flange and puzzle shaped shear connectors. Twenty-one beam tests were conducted to evaluate the global load bearing behaviour of the integrated composite flooring system.

Another type of composite flooring system which is recently developed is using cellular beams (Frangi et al., 2011). This novel flooring system is beneficial as the integrated installation floor, adding value to the floor without extra costs. The flooring system is consisted from half-cellular beams made of hot-rolled sections. The openings in the cellular beams allow for placing the installations in all directions which providing flexibility for the user for changing the installations. The general design and details of the construction of this flooring system are presented by (Frangi et al., 2011). The load-carrying and dynamic behaviour of two floor elements with a span of 7.2m were conducted. The experimental results are compared to common calculation models for composite slabs.

InaDeck is a multifunctional composite flooring system which incorporates all building services and installations into the structural element by means of an integrated installation floor (Hegger et al., 2014). The new flooring system is consisted from prestressed composite beams with shear connectors (single flange and continuous) having large web openings for integrating building services. The physical and fire protection characteristics of the flooring system has improved due to the prestressed concrete chord at the bottom of the cross section.

The increasing demand for prefabricated and shallow flooring systems in the recent years has led to the development of the hollow core precast floors and Cofradal floors. The span and width of these flooring systems with depth below 300mm are up to 7.8m for the Cofradal floor and 10.5m for the hollow core precast units (with a width of 1.2m) (Bison, 2007; ArcelorMittlal, 2019). It has become obvious that the industry is looking for increased spans with the lowest possible structural depth and weight of the flooring system to meet architectural and functional requirements as well as to reduce the number of columns and foundations leading to a lighter and more sustainable construction with reduced time and costs. For that reason, different types of flooring systems have been developed with the use of new lightweight materials (Yan et al., 2016a,b).

2. NEW COMPOSITE FLOORING SYSTEM

A recently proposed ultra-shallow flooring system is examined in this paper also known as prefabricated ultra-shallow flooring system - PUSS (Ahmed et al., 2017; Ahmed and Tsavdaridis, 2017; Ahmed and Tsavdaridis, 2018; Ahmed et al., 2018). This is a prefabricated steel-concrete composite flooring system which consists of two main structural components: the concrete floor and the steel beams. The concrete floor is in the form of T-ribbed slab sections constructed using reinforced lightweight aggregate concrete. The C-channel steel edge beams encapsulate the floor slab and provide clean and straight finish edges. The floor slab width is 2.0m inclusive of the width of the steel edge beams and a finished depth of 230mm; **Table 1** summarises different span and depths limit for the new ultra-shallow flooring system with lightweight concrete of a density of 1700kg/m^3.

The total weight of the floor is reduced by having ribs and troughs running from one side to the other side of the slab sitting on the two C-channel edge beams either side. This ultra-shallow flooring system also reduces the weight and the number of erection (installation) lifts by using lighter elements (lightweight concrete and thin-walled steel elements) and the wider possible units. Moreover, the extent of site works is reduced by pre-off site fabrication as the material cost against the fabrication and site erection costs is proportional in the order of 35% and 65%, respectively (Ahmed et al., 2017; Ahmed and Tsavdaridis, 2017). In addition, this new flooring system can be used with slimflor and ultra-shallow floor beams, creating a shallow floor construction system, as illustrated in **Figure 1**. Lytag with a low density of 700kg/m^3 and Leca with a low density of 280kg/m^3 were employed as the lightweight aggregates to achieve a very low possible density, thus weight. Lytag aggregate is recycled from the fly ash in coal-burning power plants that saved energy and reduces the carbon dioxide emissions. Lytag is up to 50% lighter than normal weight aggregate and is manufactured (artificial) lightweight aggregate. After heating at 1150°C in a rotary kiln, the clay is expanded to about four to five times its original size and takes the shape of pellets. Leca is up to 50% lighter than lightweight aggregate (Lytag) (Mazaheripour et al., 2011).

Table 1: Span limits for the ultra shallow flooring system

Floor Type	Concrete Type	Concrete density kg/m^3	Maximum Span (m)	Overall Floor Depth (mm)	Total Floor Weight (kN/m^2)	Live Load (kN/m^2)	Unit Width (mm)
Ultra shallow flooring system	Lightweight concrete	1700	8.0	230	2.67	2.5	2000
			8.0	260	2.71	3.5	2000
			9.5	300	2.81	5.0	2000
			10.0	300	2.81	3.5	2000

Ultra-shallow flooring system exercises the sustainability approach in the selection of its components using sustainable materials such as lightweight concrete (Doel, 2007) and thin-webbed steel members. An explicit Life Cycle Assessment (LCA) for this flooring system was developed and compared with other lightweight composite flooring systems such as Cofradal slab and hollow core precast slab (Ahmed and Tsavdaridis, 2018). From the study, it was found that this ultra-shallow flooring system reduces the embodied energy and embodied carbon by about 28.89% and 37.67%, respectively when compared with the Cofradal slab, and 19.47% and 33.05%, respectively when compared with the hollow core precast slab.

This paper investigates the shear resistance and behaviour of the connection systems designed for PUSS. A series of push-out tests, consisting of 8 full-scale test specimens, was performed to examine the shear connection under direct longitudinal shear force. The test specimens were designed to represent actual configurations of the shear connection system according to the construction practice. The design principle is that the shear connection of the test specimens is subjected to the direct longitudinal shear force. Therefore, the shear-resisting capacity and load-slip behaviour of the shear connection can be obtained. The experimental apparatus was specifically designed in such a way to create the desired static loading conditions and in compliance with the specifications of Eurocode 4 (EN 1994-1-1, 2004). The results of the push-out tests are analysed herein with emphasis on the failure mechanisms of the shear connection systems.

3. MECHANISMS OF SHEAR TRANSFER

The most commonly used shear connectors in bridge and building applications are the headed shear studs which are usually welded vertically to the steel flanges. In ultra-shallow flooring systems, headed shear studs are usually welded horizontally to the steel webs – no need for extra concrete depth to create the shear bond, otherwise horizontal dowels are employed to assist and/or replace the headed shear studs.

In PUSS, headed shear studs are welded on the inner side of the webs of the parallel C-channels, as shown in **Figure 2(a)**. The shear studs of the examined specimens were positioned at 435mm centres to resist the longitudinal shear force. The diameter of the studs was 19mm and the height was 95mm.

An additional shear connection system is that of the horizontally lying dowels to provide the tie-force for the concrete slab and the parallel flange C-channels. High yield dowels of Ø20mm with 2m length are used to pass through the centre of slab ribs at 870mm centres, as illustrated in **Figure 2(b)**. The studs are passing through the thin concrete flange (at 435mm centres). The dowels with the web-welded studs are designed to simultaneously resist the longitudinal shear force.

4. EXPERIMENTAL INVESTIGATION

4.1 Push-out test specimens

A total of 8 full-scale test specimens were conducted to investigate the performance of the shear mechanisms and grouped in two categories: (a) the web-welded shear studs, and (b) the combination of horizontally lying dowels and web-welded shear studs.

4.1.1 Shear connection systems

The design principle is that the shear connection of the test specimens is subjected to direct longitudinal shear force. Each test group consists of four different test specimens investigating a particular type of shear connection. The details of the test groups and the corresponding shear connection system are summarised in **Table 2**. In order to investigate the factors that influence the shear-resisting properties of the shear connection system, the test specimens of test groups (T1 and T2) were designed to have one type of variables; the concrete strength. Three types of concretes were used to cast the slabs, i.e., normal concrete, lightweight concrete (using Lytag aggregates), and ultra lightweight concrete (using Leca aggregates). The tensile strength of normal concrete was higher than that of the lightweight concrete with different compressive strength. Details of the lightweight concrete are presented in section 4.1.2.

The availability and cost of lightweight aggregate material was a limiting factor. At least one specimen with normal weight concrete, one with lightweight concrete, and one with ultra lightweight concrete, for each group, was planned. More specimens were prepared and tested to confirm the results when it was needed. For instance, one of the specimens from the Group T1 (with normal weight concrete) failed from one side rather than both sides as the load was slightly moved towards that side during testing. As a result, this specimen was repeated with normal weight concrete.

Table 2: Push-out test group details

Test Group	Shear connection	Concrete type	Specimen No.
Group T1	Web-welded studs	Normal weight Concrete (NWC)	T1-NWC-1
Group T1	Web-welded studs	Normal weight Concrete (NWC)	T1-NWC-2
Group T1	Web-welded studs	Lightweight Concrete (LWC)	T1-LWC
Group T1	Web-welded studs	Ultra Lightweight Concrete (ULWC)	T1-ULWC
Group T2	Dowels and Web-welded studs	Normal weight Concrete (NWC)	T2-NWC
Group T2	Dowels and Web-welded studs	Lightweight Concrete (LWC)	T2-LWC
Group T2	Dowels and Web-welded studs	Lightweight Concrete (LWC)	T2-LWC
Group T2	Dowels and Web-welded studs	Ultra Lightweight Concrete (ULWC)	T2-ULWC

All test specimens comprised of two parallel flange C-channel steel sections as edge beams and the concrete slab flush with the steel flanges, as shown in **Figure 3(b)**. The studs and dowels were welded to the web of the channels. The reinforced concrete ribbed slab is connecting the parallel steel edge beams and sitting on their bottom flanges. In practice, it is common to use steel wire mesh or rebar reinforcement in the concrete slab, thus minimum reinforcement was provided for the ribbed slab as well. However, heavy reinforcement might create undesirable confinement in the vicinity of the shear connection and may restrain the transverse separation of the shear connection in the push-out tests, therefore to jeopardise the accuracy of the results while overestimating the capacity. Consequently, no reinforcement has been provided in the area of shear connection systems for the experiments in order to examine the system solely subjected to direct longitudinal shear force and to minimise the number of variables which affect the push-out test [11].

The total width of the concrete slab was 2000mm for all test specimens of the test groups (T1 and T2) aiming to represent the effective width of the full concrete slab of the test specimen. Wider slabs are not suggested, as they will not fit horizontally in tracks for the transportation; equally it is not suggested to be positioned inclined as the shear connection system may be damaged during transportation. In case, narrower slabs are designed, it is expected that the failure will be more uniform, with a better interaction of developing locally around the shear studs and ends of dowels, resulting to higher shear capacity. Thus, the tested system will yield the most underestimated results. The depth of the infill part of the slabs was 217.5mm. The depth of the ribbed slabs is 75mm, with ribs of 85mm at 870mm centres in addition to the finishes of 40mm (within the depth of the ribbed slab). The overall depth of the slabs including the finishes is 200mm, as depicted in **Figure 4** and **Figure 5**.

4.1.2 Materials properties

In the present study, three types of materials were used: (1) normal weight concrete (NWC), (2) light weight concrete (LWC), and (3) ultra-light weight concrete (ULWC). **Figure 6** illustrates different types of aggregates used for different types of concrete. The concrete mixture proportions were presented in **Table 3**. NWC was manufactured from the coarse aggregate (gravel), natural sand, Portland cement, and water. Coarse aggregates with a maximum size of 10mm and natural sand were used as fine aggregates, respectively. Densities for gravel and natural sand were 1600 and 1800kg/m^3, respectively. The density of the normal concrete was 2325kg/m^3 with a compressive strength of 30MPa at 28 days.

LWC consisted of recycled lytag aggregates; coarse aggregates of size 8mm, fine aggregates of size 4mm, cement, and water with a density of 1700kg/m^3 with a compressive strength of 30MPa at 28 days.

The ULWC was produced by expanded clay coarse aggregates of size 8mm, expanded clay fine aggregates of size 4mm cement, and water with a 28-day compressive strength of about 16MPa.

All materials (steel and concrete) properties were determined through standard tests. The tensile strength of the steel beam sections, shear stud connectors, and dowel shear connectors used to fabricate the push-out test specimens were obtained from coupon tests according to the ISO 6892-1 (2009). The concrete material properties were obtained from compression and splitting tensile tests that carried out on cylinder specimens in accordance with the BS 1881-116 (1983). Compressive and splitting tensile concrete properties are presented in **Table 4**. The mechanical properties of the steel section and steel shear connectors are summarised in **Table 5**.

Table 3: Concrete mixture proportions

Concrete type	W/C ratio	Cement (kg/m^3)	FA (kg/m^3)	CA (kg/m^3)	CA type	FA type	Density (kg/m^3)
NWC	0.75	300	810	990	NA	NS	2325
LWC	0.79	250	625	520	RA	RA	1700
ULWC	0.98	450	324.5	229	EC	EC	1300

W/C water to cement ratio, FA fine aggregate, CA coarse aggregate, NA natural aggregate, NS natural sand, RA recycled aggregate, EC expanded clay.
[a] NG: natural aggregate with dry density of 1600kg/m^3
[b] NS: natural sand with dry density of 1800 kg/m^3
[c] RA: recycled aggregate (coarse Lytag) with bulk density of 700 kg/m^3
[d] RA: recycled aggregate (fine Lytag) with bulk density of 1000 kg/m^3
[e] EC: expanded clay (coarse Leca) with bulk density of 280 kg/m^3
[f] EC: expanded clay (fine Leca) with bulk density of 620 kg/m^3

Table 4: Comparison of concrete strength between normal concrete and lightweight concrete at age of 28 days.

Concrete type	Compressive strength, (MPa)	Tensile strength, (MPa)	E_c (GPa)
Normal weight concrete	30.0	2.31	31.18
Lightweight concrete	30.0	1.99	18.73
Ultra Lightweight concrete	16.0	1.25	9.56

Connection type	d	f_y (MPa)	f_u (MPa)	E_s (GPa)

Web-welded shear stud	6.6	452.1	530.2	200
Dowels	19.83	322.5	455.5	200
Steel section	-	406	570	200

Table 5: Mechanical properties of steel section and steel connectors

4.2 Details of test specimens

Test specimens of group T1 were designed with 6 headed shear studs welded symmetrically on the inner side of the web of each edge beam, as depicted in **Figure 4**. The diameter of the studs was 19mm and the height was 95mm.

Test specimens of the group T2 were designed to include the dowels and the web-welded shear studs. The reinforced concrete ribbed slab was designed according to Eurocode 2 (EN 1992-1-1, 2004) and the steel-concrete composite flooring system was designed according to Eurocode 4 (EN 1994-1-1, 2004). The diameter of the dowels was 20mm and welded to the edge beams, tying the slab and edge beams together while passing through the centre of the slab ribs. The dowels are also useful during the casting process, while holding the two edge beams in place. With these, the fabrication of this composite flooring system can also be easily done on the site if necessary. The 2 dowels were positioned at 870mm centres, as shown in **Figure 5**. The shear studs were positioned at 435mm centres passing through the thin concrete slab only (not the ribs). The dowels and studs shear connections were designed to act simultaneously to resist the longitudinal shear force.

4.3 Setup and testing procedures

The steel sections of all test specimens were covered with de-bonding grease before casting with concrete. The use of de-bonding grease was to prevent the development of the bond between the steel and concrete for this investigation. All push-out test specimens were cast in the Heavy Structures Laboratory at the University of Leeds.

The test specimens were cast horizontally for the ease of casting and replicating the fabrication in the shop. The concrete mix was designed with less flow for normal and lightweight concrete, and all test specimens were uniformly compacted to avoid any voids or segregation of the aggregates from the cement paste. Examination of tested specimens showed that segregation of aggregates did not occur. The concrete strength specimens, cubes, and cylinders were prepared using the same batch of concrete for the push-out test specimens.

All specimens, cubes, and cylinders of each push-out tests were cured under the same conditions; covered with wet sacked and plastic sheets.

A test rig of 1000kN capacity was used for the push-out tests. Static monotonic loads were applied to the test specimens by one identical hydraulic jack of 1000kN capacity. A spreader steel beam 254x254x73 UC was used to distribute the load uniformly from the hydraulic jack to the specimen. Digital dial gauges were used to measure the slip and separations of the shear connection systems. 6 digital dial gauges were positioned on both sides of the slabs measuring the slips in the vertical direction, as shown in **Figure 7**. Two digital dial gauges were positioned on both sides of the slab measuring the separations in the horizontal direction.

A data logger machine connected to a computer recording all the readings from different load levels. All the push-out test specimens were loaded until failure. The failure patterns were captured using a digital camera.

The push-out tests were carried out according to Eurocode 4 (EN 1994-1-1, 2004). Test specimens were settled onto a layer of plaster (gypsum) to create an even contact surface between the specimens and the reaction platform. The push-out tests were load-controlled with the monotonic loading applied to the steel section; hence, the incremental shear force was applied to the shear connectors, rather than the concrete slab as in typical push-out tests, aiming to avoid damaging the thin and wide concrete slab in tesing. The specimens were tested until the destructive failure of the shear connection. The duration of all push-out tests was approximately 2 hours with a load rate of 0.5kN/sec.

5. TEST RESULTS

5.1 Failure mechanisms

Tested specimens were further examined to understand the failure mechanisms of the two shear connection systems. The failure profiles of the web-welded shear stud connection system are depicted in **Figures 8, 9, 10,** and **11**. The studs were sheared off from one side (either right or left side of the specimen) in the direction of the longitudinal shear force while bending near the root of the stud; however, the studs on the opposite side were bent without shearing off. This was due to the distribution of stresses over the slab width during the test, which results in stress concentration on one side of the specimen. The bending length of the shear studs with NWC was around 40mm and larger than the one of the shear studs with LWC and ULWC which was around 10mm. This is related to the higher compressive strength of NWC which imposes higher stress on the shear studs and increases their bended length. The concrete in the vicinity of the studs was crushed in the shear direction. The concrete and web-welded shear stud connection system's failure patterns were similar to the concrete and the horizontally lying shear connection system failure patterns tested by Kuhlmann and Breuninger (2002), as shown in **Figure 12**.

The web-welded shear stud connection demonstrated splitting of the concrete slab in the push-out tests. Annex C of Eurocode 4 (EN 1994-2, 2005) provides specifications for the design of lying studs. It is thus recommended that the design of the web-welded shear stud connection system should, in practice, conform to Annex C.

The concrete cracking profile of the specimens with web-welded shear studs initiated from the top studs' position, where the position of the ribs in both sides extend towards the shear studs in the middle of the specimen, and then the cracks appear near the bottom studs as shown in **Figures 8, 9, 10,** and **11**.

The failure profile of the horizontally lying steel dowels together with the web-welded shear studs are shown in **Figures 13, 14, 15,** and **16**. The dowels and studs were sheared off from one side (either right or left side of the specimen) with bending shown near their roots, nevertheless the dowels and studs on the other side were bent without shearing off. The bending length of the steel dowels with NWC was around to 80mm and larger than the one of the steel dowels with LWC and ULWC which was around 40mm. The concrete in the vicinity of the studs was crushed in the shear direction. The shear failure mechanism of this connection system was similar to the failure mechanism shown in standard push-out tests of the headed shear studs [EN 1994-1-1, 2004].

The steel dowels together with the web-welded studs shear connection system demonstrated the splitting of the concrete slab in the push-out tests. The concrete cracks of the specimens with this shear connection system were similar to the concrete cracks shown by the specimens with web-welded shear studs connection, as depicted in **Figures 13, 14, 15,** and **16**.

5.2 Load-slip and Load-separation behaviours

Relative slip between the steel beams and concrete slab was recorded during the tests. **Figure 17** demonstrates the typical load–slip curves of the push-out tests. The load–slip curves selected for the discussion according to: (1) the type of shear connection and (2) the type of concrete (i.e., NWC, LWC, and ULWC). A comparison of the load–slip behaviours and failure modes was then established.

The majority of the load–slip curves illustrate that the specimens exhibited substantial inelastic deformation before failure. All shear connectors, which failed by bending and shearing off failure, describe a ductile load–slip performance where the slip at maximum load was more than 6mm, which is the minimum requirement according to Eurocode 4 (EN1994-1-1, 2004) for ductile shear connection. Specimens failed by bending of the connectors near the roots without shearing off, the load–slip behaviour was ductile with a slip at maximum load of more than 6mm.

Figure 18 illustrates the two load–slip behaviours per specimen of a flooring system with the two modes of failure of the shear connectors. At the initial stage of loading, the relationship between load and slip was linear. The load–slip curve exhibited nonlinear behaviour up to the maximum load. Shear studs or dowels were sheared off from one side (either right or left side of the specimen) while bending near the root of the connectors was found; however, the studs on the opposite side were bent without shearing off, as shown in **Figure 18**. The reason for obtaining these differences is that the quality of the concrete cannot be entirely guaranteed as it is produced in a laboratory environment and by technicians, instead of large mixers. It is also worth to note that the quality of lightweight concrete can vary a lot during compaction affecting the concrete strength. Thus, it is preferable to be compacted in the shop and using approved methods. In our case, as it was aforementioned, we did not use reinforcement along the steel members and in the vicinity of the shear connectors, to avoid further compaction and confinement issues. The specimens with lightweight concrete exhibited a noticeable brittle behaviour compared with the specimens with normal concrete. This is related to the normal concrete properties and its higher compressive strength.

The use of web-welded shear studs resulted in slips between 2mm and 30mm in the push-out tests. The steel dowels together with the web-welded shear studs resulted in slips between 13mm and 29mm. Large separations of the steel and concrete were observed for the specimens with web-welded shear studs; somewhere between 3mm and 24mm, which indicates the weak tie-resistance of the web-welded shear studs. On the other hand, small separations of the steel and concrete were observed for specimens with steel dowels and web-welded shear studs; somewhere between 3mm and 9mm, which indicates the strong tie-resistance of the steel dowels. All specimens demonstrate that the separation started at a load level where the sudden slip increased (**Figure 17**).

It was clearly demonstrated by all four specimens of test group T2 that an interlocking mechanism occurs between the concrete and the shear connectors at ultimate load levels. This mechanism indicates that the failure resistance (or longitudinal shear strength) of the dowels contributed to holding the whole system from failure. This confirms with the observation that the failure of the dowels occurred after the failure of the shear studs, near the end of the test. In contrast, this mechanism did not occur in the specimens of test group T1 and the contribution of web-welded studs only to holding the whole system from failure was reasonably small.

Load-separation curves represent the tie-resisting behaviour of the shear connection to the longitudinal shear force and are shown in **Figures 17** and **19**. The results of the push-out tests are summarised in **Tables 6** and **7**.

Table 6: Results of the push-out test group (T1)

Specimen No.	f_{cu}[a] (MPa)	f_{ct}[b] (MPa)	Shear Connections	Ultimate shear capacity, P_u, (kN)	Slip capacity, δ_u (mm)	Stiffness, K, (kN/mm)	Ductility classification
T1-NWC-1[c]	31.60	2.26	Right top stud	187.17	2.37	78.97	Fail
			Right middle stud	187.17	2.06	90.85	Fail
			Right bottom stud	187.17	2.06	90.85	Fail
			Left top stud	187.17	13.59	13.77	Pass
			Left middle stud	187.17	13.09	14.29	Pass
			Left bottom stud	187.17	12.33	15.18	Pass
T1-NWC-2	38.52	2.88	Right top stud	103.97	21.60	5.34	Pass
			Right middle stud	103.97	21.30	4.88	Pass
			Right bottom stud	103.97	23.20	4.48	Pass
			Left top stud	103.97	6.58	15.80	Fail
			Left middle stud	103.97	6.58	15.80	Fail
			Left bottom stud	103.97	6.63	15.68	Fail
T1-LWC	32.20	1.61	Right top stud	86.70	16.28	5.32	Pass
			Right middle stud	86.70	15.45	5.61	Pass
			Right bottom stud	86.70	15.63	5.54	Pass
			Left top stud	86.70	30.07	2.88	Pass
			Left middle stud	86.70	30.07	2.88	Pass
			Left bottom stud	86.70	21.82	3.97	Pass

T1-ULWC	20.0	1.36	Right top stud	57.02	20.63	2.76	Pass
			Right middle stud	57.02	20.29	2.81	Pass
			Right bottom stud	57.02	20.12	2.83	Pass
			Left top stud	57.02	12.41	4.59	Pass
			Left middle stud	57.02	11.85	4.81	Pass
			Left bottom stud	57.02	11.73	4.86	Pass

a Mean cube compressive strength.
b Mean cylinder tensile splitting strength.
c The specimen, T1-NC-1 was failed from one side rather than two sides, therefore the ultimate load is taken by three shear connections only rather than six shear connections, and the ultimate shear capacity is per shear connection of the three shear connections

Table 7: Results of the push-out test group (T2)

Specimen No.	f_{cu}[a] (MPa)	f_{ct}[b] (MPa)	Shear Connections	Ultimate shear capacity, P_u, (kN)	Slip capacity, δ_u (mm)	Stiffness, K, (kN/mm)	Ductility classification
T2-NWC	37.3	2.45	Right top dowel	121.9	12.18	10.0	Pass
			Right stud	121.9	11.55	10.58	Pass
			Right bottom dowel	121.9	12.09	10.08	Pass
			Left top dowel	121.9	13.64	8.93	Pass
			Left stud	121.9	12.83	9.50	Pass
			Left bottom dowel	121.9	13.64	8.93	Pass
T2-LWC-1	34.6	2.11	Right top dowel	101.65	22.10	4.59	Pass
			Right stud	101.65	21.50	4.72	Pass
			Right bottom dowel	101.65	21.10	4.81	Pass
			Left top dowel	101.65	31.10	3.63	Pass
			Left stud	101.65	30.10	3.37	Pass
			Left bottom dowel	101.65	30.10	3.37	Pass
T2-LWC-2	36.8	2.12	Right top dowel	103.51	22.20	4.66	Pass
			Right stud	103.51	21.00	4.92	Pass
			Right bottom dowel	103.51	22.20	4.66	Pass
			Left top dowel	103.51	31.20	3.31	Pass
			Left stud	103.51	30.10	3.43	Pass
			Left bottom dowel	103.51	30.90	3.34	Pass
T2-ULWC	20.0	1.38	Right top dowel	73.83	31.90	2.31	Pass
			Right stud	73.83	30.70	2.40	Pass
			Right bottom dowel	73.83	30.90	2.38	Pass

			Left top dowel	73.83	29.00	2.54	Pass
			Left stud	73.83	27.30	2.70	Pass
			Left top dowel	73.83	28.00	2.63	Pass

5.3 Effect of connection system type

Figure 20 shows the effect of the connector type on the maximum applied load. It could be observed that changing the type of the shear connection from web-welded shear studs to the combination of horizontal lying dowels with web-welded shear studs leads to a higher capacity. This is related to the larger diameter of the dowel with a larger cross-sectional area and thus a larger bearing area of the concrete as it passes from one side to the other side of the flooring system tying it all together, which in turn increases the maximum shear capacity of the connection system.

Nevertheless, the maximum shear capacity of the shear connection system is also influenced by the yield strength of the steel shear connectors and the mechanical properties of the concrete used. When the diameter of the shear connector is large (> 12mm), the maximum shear capacity of the shear connection system depends on the strength of the concrete materials. However, if the diameter of the shear connection system is small (< 10mm), the failure is controlled by shank shearing and not influenced much by the type and strength of concrete (Yan et al., 2014).

5.4 Effect of concrete type

Figure 21 shows the effect of the concrete type on the maximum shear capacity of both shear connection systems. The shear capacity of the connection system is defined as the ratio of the maximum applied load to the number of the shear connectors per specimen.

It is evident that the maximum applied load increased by 15% when NWC was used in comparison with the LWC of similar compression strength (see **Tables 6** and **7**). Subsequently, the maximum applied load increased by 14% when LWC was used in comparison with the ULWC of similar compression strength. Modern design codes, such as Eurocode 4(EN 1994-1-1, 2004) and AISC (1994), include the compressive strength and secant modulus properties of concretes to predict the shear strength of the connection. The formulae in Eurocode 4 (EN 1994-1-1, 2004) can be used with a concrete of density not less than 1750kg/m^3, thus it deals with LCW, but not ULWC.

6. LOAD–SLIP BEHAVIOUR OF SHEAR CONNECTION

To analyse this proposed ultra-shallow flooring system for load–slip response and ultimate shear capacity, it is essential to represent the load–slip (P–s) behaviour of the shear connection systems. This section proposes a suitable load–slip model for web-welded studs, and horizontally lying dowels together with web-welded studs, which is established from the regression analysis of the load–slip curves of the push-out tests.

6.1 Load-slip models for headed shear stud connector

Ollgaard et al. (1971) suggested an expression to represent the load–slip relationship based on curved fitting with the data from the push-out test as shown below:

$$\frac{P}{P_u} = \left(1 - e^{-18\delta}\right)^{0.4} \qquad (1)$$

Where P is the applied shear force, P_u is the shear resistance of the shear connector, δ is the slip in inches due to applied load P.

However, a modification has been made by Lorenc and Kubica (2006) on Eq. 1 using an experimental calibration with the data from the push-out test to achieve different coefficients:

$$\frac{P}{P_u} = \left(1 - e^{0.55\delta}\right)^{0.3} \qquad (2)$$

Xue et al. (2008) introduced a formula to predict the load–slip relationship using 30 push-out tests using headed shear stud connectors and the analysis of other researchers' expressions. The formula is as follows:

$$\frac{P}{P_u} = \frac{\delta}{0.5 + 0.97\delta} \qquad (3)$$

Where δ is the slip in mm.

An and Cederwall (1996) proposed two expressions based on a nonlinear regression analysis of the test results to predict the load–slip behaviour of the headed shear stud connectors in NWC and high-performance concrete (HPC) under cyclic loading, as follows:

$$\frac{P}{P_u} = \frac{2.24(\delta - 0.058)}{1 + 0.98(\delta - 0.058)} \quad \text{for NWC,} \qquad (4a)$$

$$\frac{P}{P_u} = \frac{4.44(\delta - 0.031)}{1 + 4.24(\delta - 0.031)} \quad \text{for HPC,} \qquad (4b)$$

Where δ is the slip in mm.

Gattesco and Giuriani (1996) proposed an alternate empirical model for the load-slip behaviour, the model is as follows:

$$\frac{P}{P_u} = \alpha\sqrt{1 - e^{-\beta\delta/\alpha}} + \gamma\delta \qquad (5)$$

Where a, b, and c are empirical parameters with the values of 0.97, 1.3, and 0.0045 mm^{-1}, respectively, obtained from curve fitting with the test data. Eq. 5 is a modified model to the models suggested by Aribert (1990) and by Johnson and Molenstra (1991).

The following section extends the existing models, which are established for headed shear stud connectors, to predict the load–slip behaviour of web-welded studs, and horizontally lying dowels together with web-welded studs.

6.2 Load-slip models for the two proposed shear connection systems

The experimental non-dimensionalised load (P/P_u) and slip (δ) curves of specimens in groups T1 and T2 with the two shear connection systems and with different concrete types are shown in **Figure 22**.

It is noticed that the generalised load–slip curves are very similar for specimens with similar concrete type and similar shear connection system. Therefore, it is proposed that the load–slip models should be identified based on the specimens with (i) different concrete types and (ii) different shear connections.

Based on the measured values and shape of the experimental push-out test curves, the constitutive laws of Xue et al. (2008), Ollgaard et al. (1971), and Gattesco and Giuriani (1996) were adopted for the theoretical analysis of the two proposed shear connection systems.

$$\frac{P}{P_u} = \frac{A\delta}{0.5 + B\delta} \qquad (6a)$$

$$\frac{P}{P_u} = (1 - e^{A\delta})^{\,B} \qquad (6b)$$

$$\frac{P}{P_u} = A\sqrt{1 - e^{-B\delta/A}} + C\delta \qquad (6c)$$

Where A, B, and C are the coefficients.

The nonlinear regression analysis of the push-out test results was carried out to obtain the coefficients in Eq. 6. Different values of A, B, and C were suggested for NWC, LWC, and ULWC and summarised in **Table 8**. The comparisons between generalised load–slip curves from Eqs. 6a–6c and test results are also shown in **Figure 22**. It is noticed that the suggested models for representing the load–slip behaviours agree well with the experimental load–slip curves, especially for the specimens with LWC. Equation 6a is the simplest among the three equations and therefore it is recommended for the use in predicting the load–slip response of both shear connection systems using different concrete materials as follows:

For specimens with web-welded stud shear connection system:

$$\frac{P}{P_u} = \frac{4.02\delta}{1 + 4.16\delta} \quad \text{, for NWC} \qquad (7a)$$

$$\frac{P}{P_u} = \frac{0.98\delta}{1 + 0.96\delta} \quad \text{, for LWC} \qquad (7b)$$

$$\frac{P}{P_u} = \frac{1.92\delta}{1 + 1.77\delta} \quad \text{, for ULWC} \qquad (7c)$$

For specimens with horizontally lying dowels together with web-welded stud shear connection system:

$$\frac{P}{P_u} = \frac{1.81\delta}{1 + 1.95\delta} \quad \text{, for NWC} \qquad (8a)$$

$$\frac{P}{P_u} = \frac{1.09\delta}{1 + 1.25\delta} \quad \text{, for LWC} \qquad (8b)$$

$$\frac{P}{P_u} = \frac{0.23\delta}{1 + 0.21\delta} \quad \text{, for ULWC} \qquad (8c)$$

Table 8: Coefficients for proposed design formula

Shear connection type	Concrete type	A	B	C
Equation 6a				
Web-welded	NWC	4.02	4.16	-

stud	LWC	0.98	0.96	-
	ULWC	1.92	1.77	-
Horizontally lying dowels together with web-welded stud	NWC	1.81	1.95	-
	LWC	1.09	1.25	-
	ULWC	0.23	0.21	-
Equation 6b				
Web-welded stud	NWC	-0.5	0.35	-
	LWC	-0.2	0.35	-
	ULWC	-0.3	0.4	-
Horizontally lying dowels together with web-welded stud	NWC	-0.2	0.35	-
	LWC	-0.1	0.35	-
	ULWC	-0.05	0.35	-
Equation 6c				
Web-welded stud	NWC	0.9	0.75	0.0095
	LWC	0.85	0.45	0.0075
	ULWC	0.9	0.5	0.006
Horizontally lying dowels together with web-welded stud	NWC	0.85	0.35	0.01
	LWC	0.75	0.3	0.009
	ULWC	0.75	0.35	0.0075

7. SHEAR STRENGTH OF CONNECTION SYSTEM WITH WEB-WELDED STUDS AND DOWELS

7.1 Existing design formulae for headed shear studs

Design codes are available to determine the shear capacity (P_{Rd}) of the headed shear stud connectors. In Eurocode 4 (EN 1994-1-1, 2004), the shear strength of the headed shear studs is given as:

$$P_s = \min\left(\frac{0.8f_u \pi d^2/4}{\gamma_v}, \frac{0.29\alpha d^2\sqrt{f_{ck}E_c}}{\gamma_v}\right) \quad (9)$$

Where f_u is the specified ultimate strength of the stud ($\leq$ 500 MPa), d is the diameter of the stud, γ_v is the partial factor (1.25), f_{ck} is the concrete cylinder compressive strength, E_c is the elastic modulus of concrete, $\alpha = 0.2(h_s/d + 1)$ for $3 \leq hs/d \leq 4$ or $\alpha = 1.0$ for $hs/d \geq 4$, and h_s is the overall height of the stud.

In Annex C of Eurocode 4 (EN 1994-2, 2005), the shear strength of the horizontal lying shear stud connector, which is responsible for the splitting in the direction of slab thickness, is specified by:

$$P_s = \frac{1.4k_v(f_{ck}d\grave{a}_r)^{0.4}(a/s)^{0.3}}{\gamma_v} \quad (10)$$

Where $\grave{a}_r$ is the effective edge distance = $a_r - c_v - \text{Ø}_s/2 \geq 50$ mm; $k_v = 1$ for shear connection in an edge position, $k_v = 1.14$ for shear connection in a middle position; γ_v is a partial factor taken as (1.25), f_{ck} is the characteristic cylinder strength of the concrete at the age considered, in N/mm^2; d is the diameter of the shank of the stud with $19 \leq d \leq 25$mm; h is the overall height of the headed stud with $h/d \geq 4$; a is the horizontal spacing of studs with $110 \leq a \leq 440$ mm; s is the spacing of stirrups with both $a/2 \leq s \leq a$, and $s/\grave{a}_r \leq 3$; Ø_s is the diameter of the stirrups with $\text{Ø}_s \geq 8$ mm, Ø_ℓ is the diameter of the longitudinal reinforcement with $\text{Ø}_\ell \geq 10$ mm, and C_v is the vertical concrete cover.

In ANSI/AISC 360-10 (2010), the nominal shear strength of the headed studs embedded in concrete is specified by:

$$P_s = 0.5A_s\sqrt{f_{ck}\,E_c} \leq 0.75f_uA_s \quad (11)$$

In AASHTO (2004), the shear strength of headed shear studs embedded in concrete is calculated as:

$$P_s = \text{Ø}0.5A_s\sqrt{f_{ck}\,E_c} \leq 0.75\,f_uA_s \quad (12)$$

Where Ø is the resistance factor for the shear connectors (=0.85).

Chinn (1965) proposed a formula for estimating the shear strength of headed shear studs embedded in LWC. The shear strength of the headed shear studs is given as:

$$P_s = 39.22d^{1.766} \quad (13)$$

Where d is the stud diameter.

Ollgaard et al. (1971) also developed a formula for calculating the ultimate shear strength of the stud (P_s) as follows:

$$P_s = 1.106 A_s f_c^{\prime\,0.3} E_c^{\,0.44} \qquad (14)$$

Classen & Hegger (2017) have proposed more accurate models using realistic parameters like stiffness and ductility for calculating the shear strength of composite dowel connectors.

$$P_{po} = \frac{1}{\eta} . \chi_x . \left(1 + \rho_{D,i}\right) . 41 . \sqrt{f_{ck}} . h_{p0}^{1.5} \qquad (15)$$

Where:

$$\eta = 0.4 - 0.001 . f_{ck}$$

$$\chi_x = \frac{e_x}{4.5 h_{p0}}$$

$$\rho_{D,i} = \frac{A_{sf} E_s}{A_{D,i} E_c} = \frac{(A_{b+} A_t) E_s}{h_c - e_x - E_c}$$

$$h_{po} = \min\left(c_t + 0.07 . e_x ; c_b + 0.13 . e_x\right)$$

To this end, Eqs. 9-12 presented earlier were developed for headed shear stud connectors embedded in NWC. The latter two studies have been conducted on establishing the shear strength of headed shear studs embedded in LWC, but there is no design guide available for the design of the horizontal lying dowels. Therefore, the design of the two proposed shear connection systems and with the use of ULWC require further calibration with test data as described in the next section.

7.2 Proposed formulae for connection system with web-welded studs and dowels

A preliminary equation suggested based on the nonlinear regression analysis using the statistic software MINITAB (2017). Further development will be carry out based on finite element parametric studies.

The shear strength (P_{sd}) from the web-welded shear studs and the one from the horizontal lying dowels together with the web-welded shear studs was considered as an independent variable. The f_c, d, and a_r were considered as dependent variables with respect to the shear strength of the connection system.

For specimens in group T1 and T2, shear strength is assumed as an exponential function of the above parameters:

$$P_{sd} = 1.873(f_{ck}\, d\, a_r)^{0.835} \leq 0.8 f_u A_s \qquad (16)$$

Where P_{sd} is the shear resistance of shear stud or dowel, f_{ck} is the cylinder compressive strength of concrete, d is the diameter of stud or dowel, and a_r is the distance from first stud or dowel to the top of concrete, f_u is the ultimate tensile strength of the material of the stud or dowel which should not be greater than 500N/mm^2, and A_s is the cross-sectional area of the shear the stud or dowel.

7.3 Shear strength verification against test results

The shear resistances of the two proposed connection systems as predicted by various formulae are compared with the test results and shown in **Table 9**.

From the results shown in **Table 9** and **Figure 23**, the proposed equation (Eq. 16) demonstrates a good fit. Ollgaard et al. (1971) gives the least reliable predictions which overestimate the test results by about 36%. The formula given in AASHTO (2004) is almost identical to design formulae given by ANSI/AISC 360-10 (2010) except the value of the reduction factor (ANSI adopted 0.5 instead of Ø0.5), see Eqs. 11 and 12. Hence, the AASHTO (2004) gives lower predictions than the ones by ANSI/AISC 360-10 (2010). Eurocode 4 (EN 1994-1-1, 2004) (Eq. 9) provides the second most conservative predictions compared to Eq. 14.

It is worth noting that the exiting formulae given in modern codes are derived the connection systems proposed in this paper – i.e., web-welded studs and horizontal lying dowels, neither for the use of ULWC. Therefore, considering both accuracy and reliability, the proposed formula Eq. 16 offers a reasonable prediction and is recommended to be used in the design of PUSS with both proposed shear connection systems. More data is required to validate the proposed formula; a parametric finite element study is further suggested.

8. CONCLUDING REMARKS

The maximum shear strength and load–slip behaviours of two proposed connection systems using NWC, LWC, and ULWC were investigated through full-scale 8 push-out tests of a new prefabricated ultra-shallow flooring system design, the so-called PUSS. On the basis of the test results and analyses presented herein, the following conclusions were made.

(1) Three types of failure were noticed from the push-out tests: (a) shear failure with bending near the roots of the connectors, (b) shear failure of the weld toe of shear studs, and (c) concrete cracking. Brittle weld failure must be avoided by ensuring quality of the welding during the fixing of the shear connectors.
(2) The concrete strength, f_{ck}, influences the failure modes. The shear resistance of each connection system was increased with the increase of the concrete strength.
(3) Larger diameter of horizontally lying steel dowels (up to 20mm in the current study) increases the shear interaction area in addition to the concrete bearing area, thus enhances the shear resistance.
(4) The horizontally lying steel dowels together with the web-welded shear studs connection system increases the shear resistance and the slip capacity of the shear connection.
(5) The shear resistance of any connection system is governed by both the tensile strength of the connectors and the concrete bearing strength. The compressive strength of the concrete significantly influences the ultimate shear strength capacity loads (higher when NWC and lower when ULWC) while it is changing the failure mode of the connection. After a regression analysis of the push-out test results, an empirical formula has been proposed; it is suggested to revise it after conducting parametric finite element studies.

$$\mathrm{P_{sd}} = \min\left(1.873(\mathrm{f_{ck}}\, d\, a_r)^{0.835}\ ,0.8\mathrm{f}_u \mathrm{A}_s\right)$$

(6) The connection system with the web-welded shear studs demonstrated a ductile failure mode of the entire slab system under direct longitudinal shear force, with slip capacities ranging between 2mm and 30mm for different concrete strengths.
(7) The connection system with the horizontal lying steel dowels together with the web-welded shear studs demonstrated a more ductile failure mode of the entire slab system under direct longitudinal shear force in comparison with the system having studs only, with slip capacities ranging between 13mm and 29mm for different concrete strengths.
(8) An interlocking mechanism was found at ultimate loads between the concrete and the shear connectors of the specimens in group T2. This mechanism demonstrates strong tie-resistance of the steel dowels, as very little separation in the transverse direction was observed when compared with the large separation of the specimens in group T1 (shear studs only).

(9) It is worth to mention that the combined horizontal and vertical shear has an important effect on the behaviour of the new PUSS. The probability of concrete cracks occur in the layer of the stud connectors should be taken into consideration. The aspect has not been investigated for conventional steel and concrete composite structures, where concrete is only used in the compression zone. However, for these shallow flooring systems, where concrete is also used in the tension zone, this may have critical. It has been demonstrated from several researchers, that concrete cracking may have a significant influence on the connector behaviour (e.g., Johnson, R. P., Greenwood, R. D., & Van Dalen, K., 1969; Classen, M., & Hegger, J., 2018).

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