



City Research Online

City St George's, University of London

Citation: Ahmed, I. M. & Tsavdaridis, K. D. (2020). Shear connection of prefabricated slabs with LWC - Part 1: Experimental and analytical studies. *Journal of Constructional Steel Research*, 169, 106016. doi: 10.1016/j.jcsr.2020.106016

This is the accepted version of the paper.

This version of the publication may differ from the final published version. To cite this item please consult the publisher's version.

Permanent repository link: <https://openaccess.city.ac.uk/id/eprint/27034/>

Link to published version: <https://doi.org/10.1016/j.jcsr.2020.106016>

Copyright and Reuse: Copyright and Moral Rights remain with the author(s) and/or copyright holders. Copies of full items can be used for personal research or study, educational, or not-for-profit purposes without prior permission or charge, unless otherwise indicated, provided that the authors, title and full bibliographic details are credited, a hyperlink and/or URL is given for the original metadata page and the content is not changed in any way. For full details of reuse please refer to [City Research Online policy](#).

1 Shear Connection of Prefabricated Slabs with LWC - Part1: Experimental 2 and Analytical Studies

3 ¹Inas Mahmood Ahmed and ^{*2}Konstantinos Daniel Tsavdaridis

4 ¹Assistant Lecturer, Northern Technical University, Mosul, Iraq

5 ²Associate Professor of Structural Engineering, School of Civil Engineering, University of Leeds,
6 Leeds, LS2 9JT, UK

7 **Corresponding author (k.tsavdaridis@leeds.ac.uk)*

8 **ABSTRACT**

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

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

28
29 *Keywords:* push-out tests; steel-concrete composite slabs; ultra-shallow flooring system;
30 shear studs; lightweight aggregate concrete; steel dowels

33 **1. INTRODUCTION**

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

46 Different flooring systems with integrated building services are beneficial for the use
47 in residential and office buildings, malls and airport structures. However, design rules for
48 such flooring systems have not been included in the European codes (Schäfer, 2015).
49 Additional guidelines for shallow flooring systems should be considered for ultimate limit
50 state, serviceability limit state, and fire design. The plastic moment resistance, the influence
51 of transverse bending moments and the characteristics in erection state for the cross section
52 classification for shallow flooring systems have been presented by (Schäfer, M., 2015). One
53 of the most recent marketed shallow flooring system is the Slim-floor beam. This type of
54 flooring system consists of a rolled or a welded steel profile which is completely or almost
55 completely integrated into the ceiling (Schäfer and Braun, 2019). The construction method of
56 this type of shallow flooring system and its development is presented by (Schäfer and Braun,
57 2019).

58 Sustainability needs have led to the development of other innovative integrated floor
59 slabs which enable wide spans and building services to be integrated through the floor height
60 (Hegger et al., 2013). These integrated floor slabs require large web openings in the structural
61 elements (Dressen et al., 2015). The influence of openings on the load-bearing capacity and
62 deformation behaviour of double-T-shaped concrete beams with prestressed tension chords
63 were investigated using six beam tests. The test parameters were the amount of vertical
64 reinforcement at the edges of the opening, the concrete strength, and the location of openings
65 in the longitudinal direction (Dressen et al., 2015). The load-carrying capacity of concrete

66 beams with openings can achieve approximately the same load-carrying capacity of the
67 concrete beams without openings using proper arrangement and dimensioning of
68 reinforcement. The load bearing of an integrated composite floor system has been
69 investigated by Gallwoszus et al. (2014). This type of floor is a novel multifunctional flooring
70 system which integrates building services and technical installations within the structural
71 element. Therefore, the presence of large openings in the structural elements' webs is
72 required. The load bearing structural element within the innovative flooring system is
73 represented by the prestressed steel-concrete hybrid beams with single flange and puzzle
74 shaped shear connectors. Twenty-one beam tests were conducted to evaluate the global load
75 bearing behaviour of the integrated composite flooring system.

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

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

92 The increasing demand for prefabricated and shallow flooring systems in the recent
93 years has led to the development of the hollow core precast floors and Cofradal floors. The
94 span and width of these flooring systems with depth below 300mm are up to 7.8m for the
95 Cofradal floor and 10.5m for the hollow core precast units (with a width of 1.2m)
96 (Bison, 2007; ArcelorMittlall, 2019). It has become obvious that the industry is looking for
97 increased spans with the lowest possible structural depth and weight of the flooring system to

98 meet architectural and functional requirements as well as to reduce the number of columns
99 and foundations leading to a lighter and more sustainable construction with reduced time and
100 costs. For that reason, different types of flooring systems have been developed with the use of
101 new lightweight materials (Yan et al., 2016a,b).

102 2. NEW COMPOSITE FLOORING SYSTEM

103 A recently proposed ultra-shallow flooring system is examined in this paper also known as
104 prefabricated ultra-shallow flooring system - PUSS (Ahmed et al., 2017; Ahmed and
105 Tsavdaridis, 2017; Ahmed and Tsavdaridis, 2018; Ahmed et al., 2018). This is a
106 prefabricated steel-concrete composite flooring system which consists of two main structural
107 components: the concrete floor and the steel beams. The concrete floor is in the form of T-
108 ribbed slab sections constructed using reinforced lightweight aggregate concrete. The C-
109 channel steel edge beams encapsulate the floor slab and provide clean and straight finish
110 edges. The floor slab width is 2.0m inclusive of the width of the steel edge beams and a
111 finished depth of 230mm; **Table 1** summarises different span and depths limit for the new
112 ultra-shallow flooring system with lightweight concrete of a density of 1700kg/m³.

113 The total weight of the floor is reduced by having ribs and troughs running from one
114 side to the other side of the slab sitting on the two C-channel edge beams either side. This
115 ultra-shallow flooring system also reduces the weight and the number of erection
116 (installation) lifts by using lighter elements (lightweight concrete and thin-walled steel
117 elements) and the wider possible units. Moreover, the extent of site works is reduced by pre-
118 off site fabrication as the material cost against the fabrication and site erection costs is
119 proportional in the order of 35% and 65%, respectively (Ahmed et al., 2017; Ahmed and
120 Tsavdaridis, 2017). In addition, this new flooring system can be used with slimflor and ultra-
121 shallow floor beams, creating a shallow floor construction system, as illustrated in **Figure 1**.
122 Lytag with a low density of 700kg/m³ and Leca with a low density of 280kg/m³ were
123 employed as the lightweight aggregates to achieve a very low possible density, thus weight.
124 Lytag aggregate is recycled from the fly ash in coal-burning power plants that saved energy
125 and reduces the carbon dioxide emissions. Lytag is up to 50% lighter than normal weight
126 aggregate and is manufactured (artificial) lightweight aggregate. After heating at 1150°C in a
127 rotary kiln, the clay is expanded to about four to five times its original size and takes the
128 shape of pellets. Leca is up to 50% lighter than lightweight aggregate (Lytag) (Mazaheripour
129 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

131

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

141 This paper investigates the shear resistance and behaviour of the connection systems
 142 designed for PUSS. A series of push-out tests, consisting of 8 full-scale test specimens, was
 143 performed to examine the shear connection under direct longitudinal shear force. The test
 144 specimens were designed to represent actual configurations of the shear connection system
 145 according to the construction practice. The design principle is that the shear connection of the
 146 test specimens is subjected to the direct longitudinal shear force. Therefore, the shear-
 147 resisting capacity and load-slip behaviour of the shear connection can be obtained. The
 148 experimental apparatus was specifically designed in such a way to create the desired static
 149 loading conditions and in compliance with the specifications of Eurocode 4 (EN 1994-1-1,
 150 2004). The results of the push-out tests are analysed herein with emphasis on the failure
 151 mechanisms of the shear connection systems.

152 3. MECHANISMS OF SHEAR TRANSFER

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

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

162 An additional shear connection system is that of the horizontally lying dowels to
163 provide the tie-force for the concrete slab and the parallel flange C-channels. High yield
164 dowels of $\varnothing 20$ mm with 2m length are used to pass through the centre of slab ribs at 870mm
165 centres, as illustrated in **Figure 2(b)**. The studs are passing through the thin concrete flange
166 (at 435mm centres). The dowels with the web-welded studs are designed to simultaneously
167 resist the longitudinal shear force.

168 4. EXPERIMENTAL INVESTIGATION

169 4.1 Push-out test specimens

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

173 4.1.1 *Shear connection systems*

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

183 concrete with different compressive strength. Details of the lightweight concrete are
 184 presented in section 4.1.2.

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

192 **Table 2:** Push-out test group details

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

193
 194 All test specimens comprised of two parallel flange C-channel steel sections as edge beams
 195 and the concrete slab flush with the steel flanges, as shown in **Figure 3(b)**. The studs and
 196 dowels were welded to the web of the channels. The reinforced concrete ribbed slab is
 197 connecting the parallel steel edge beams and sitting on their bottom flanges. In practice, it is
 198 common to use steel wire mesh or rebar reinforcement in the concrete slab, thus minimum

199 reinforcement was provided for the ribbed slab as well. However, heavy reinforcement might
200 create undesirable confinement in the vicinity of the shear connection and may restrain the
201 transverse separation of the shear connection in the push-out tests, therefore to jeopardise the
202 accuracy of the results while overestimating the capacity. Consequently, no reinforcement
203 has been provided in the area of shear connection systems for the experiments in order to
204 examine the system solely subjected to direct longitudinal shear force and to minimise the
205 number of variables which affect the push-out test [11].

206 The total width of the concrete slab was 2000mm for all test specimens of the test
207 groups (T1 and T2) aiming to represent the effective width of the full concrete slab of the test
208 specimen. Wider slabs are not suggested, as they will not fit horizontally in tracks for the
209 transportation; equally it is not suggested to be positioned inclined as the shear connection
210 system may be damaged during transportation. In case, narrower slabs are designed, it is
211 expected that the failure will be more uniform, with a better interaction of developing locally
212 around the shear studs and ends of dowels, resulting to higher shear capacity. Thus, the tested
213 system will yield the most underestimated results. The depth of the infill part of the slabs was
214 217.5mm. The depth of the ribbed slabs is 75mm, with ribs of 85mm at 870mm centres in
215 addition to the finishes of 40mm (within the depth of the ribbed slab). The overall depth of
216 the slabs including the finishes is 200mm, as depicted in **Figure 4** and **Figure 5**.

217 *4.1.2 Materials properties*

218 In the present study, three types of materials were used: (1) normal weight concrete (NWC),
219 (2) light weight concrete (LWC), and (3) ultra-light weight concrete (ULWC). **Figure 6**
220 illustrates different types of aggregates used for different types of concrete. The concrete
221 mixture proportions were presented in **Table 3**. NWC was manufactured from the coarse
222 aggregate (gravel), natural sand, Portland cement, and water. Coarse aggregates with a
223 maximum size of 10mm and natural sand were used as fine aggregates, respectively.
224 Densities for gravel and natural sand were 1600 and 1800kg/m³, respectively. The density of
225 the normal concrete was 2325kg/m³ with a compressive strength of 30MPa at 28 days.

226 LWC consisted of recycled lytag aggregates; coarse aggregates of size 8mm, fine
227 aggregates of size 4mm, cement, and water with a density of 1700kg/m³ with a compressive
228 strength of 30MPa at 28 days.

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

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

240 **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

241 W/C water to cement ratio, FA fine aggregate, CA coarse aggregate, NA natural aggregate, NS natural sand, RA
 242 recycled aggregate, EC expanded clay.

243 ^a NG: natural aggregate with dry density of 1600kg/m³

244 ^b NS: natural sand with dry density of 1800 kg/m³

245 ^c RA: recycled aggregate (coarse Lytag) with bulk density of 700 kg/m³

246 ^d RA: recycled aggregate (fine Lytag) with bulk density of 1000 kg/m³

247 ^e EC: expanded clay (coarse Leca) with bulk density of 280 kg/m³

248 ^f EC: expanded clay (fine Leca) with bulk density of 620 kg/m³

249

250

251

252 **Table 4:** Comparison of concrete strength between normal concrete and lightweight concrete
 253 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

254

Connection type	d	f _y (MPa)	f _u (MPa)	E _s (GPa)
-----------------	---	----------------------	----------------------	----------------------

255 **Table**

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

5:

256 Mechanical properties of steel section and steel connectors

257 4.2 Details of test specimens

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

261 Test specimens of the group T2 were designed to include the dowels and the web-
262 welded shear studs. The reinforced concrete ribbed slab was designed according to Eurocode
263 2 (EN 1992-1-1, 2004) and the steel-concrete composite flooring system was designed
264 according to Eurocode 4 (EN 1994-1-1, 2004). The diameter of the dowels was 20mm and
265 welded to the edge beams, tying the slab and edge beams together while passing through the
266 centre of the slab ribs. The dowels are also useful during the casting process, while holding
267 the two edge beams in place. With these, the fabrication of this composite flooring system
268 can also be easily done on the site if necessary. The 2 dowels were positioned at 870mm
269 centres, as shown in **Figure 5**. The shear studs were positioned at 435mm centres passing
270 through the thin concrete slab only (not the ribs). The dowels and studs shear connections
271 were designed to act simultaneously to resist the longitudinal shear force.

272 4.3 Setup and testing procedures

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

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

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

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

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

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

304 **5. TEST RESULTS**

305 5.1 Failure mechanisms

306 Tested specimens were further examined to understand the failure mechanisms of the two
307 shear connection systems. The failure profiles of the web-welded shear stud connection
308 system are depicted in **Figures 8, 9, 10, and 11**. The studs were sheared off from one side
309 (either right or left side of the specimen) in the direction of the longitudinal shear force while
310 bending near the root of the stud; however, the studs on the opposite side were bent without
311 shearing off. This was due to the distribution of stresses over the slab width during the test,
312 which results in stress concentration on one side of the specimen. The bending length of the
313 shear studs with NWC was around 40mm and larger than the one of the shear studs with
314 LWC and ULWC which was around 10mm. This is related to the higher compressive
315 strength of NWC which imposes higher stress on the shear studs and increases their bended

316 length. The concrete in the vicinity of the studs was crushed in the shear direction. The
317 concrete and web-welded shear stud connection system's failure patterns were similar to the
318 concrete and the horizontally lying shear connection system failure patterns tested by
319 Kuhlmann and Breuninger (2002), as shown in **Figure 12**.

320
321 The web-welded shear stud connection demonstrated splitting of the concrete slab in
322 the push-out tests. Annex C of Eurocode 4 (EN 1994-2, 2005) provides specifications for the
323 design of lying studs. It is thus recommended that the design of the web-welded shear stud
324 connection system should, in practice, conform to Annex C.

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

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

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

343 5.2 Load-slip and Load-separation behaviours

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

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

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

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

381 It was clearly demonstrated by all four specimens of test group T2 that an
382 interlocking mechanism occurs between the concrete and the shear connectors at ultimate

383 load levels. This mechanism indicates that the failure resistance (or longitudinal shear
 384 strength) of the dowels contributed to holding the whole system from failure. This confirms
 385 with the observation that the failure of the dowels occurred after the failure of the shear studs,
 386 near the end of the test. In contrast, this mechanism did not occur in the specimens of test
 387 group T1 and the contribution of web-welded studs only to holding the whole system from
 388 failure was reasonably small.

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

392

393

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							

394

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

395 5.3 Effect of connection system type

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

403 Nevertheless, the maximum shear capacity of the shear connection system is also
404 influenced by the yield strength of the steel shear connectors and the mechanical properties of
405 the concrete used. When the diameter of the shear connector is large (> 12mm), the
406 maximum shear capacity of the shear connection system depends on the strength of the
407 concrete materials. However, if the diameter of the shear connection system is small
408 (< 10mm), the failure is controlled by shank shearing and not influenced much by the type
409 and strength of concrete (Yan et al., 2014).

410 5.4 Effect of concrete type

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

414 It is evident that the maximum applied load increased by 15% when NWC was used
415 in comparison with the LWC of similar compression strength (see **Tables 6** and **7**).
416 Subsequently, the maximum applied load increased by 14% when LWC was used in
417 comparison with the ULWC of similar compression strength. Modern design codes, such as
418 Eurocode 4(EN 1994-1-1, 2004) and AISC (1994), include the compressive strength and
419 secant modulus properties of concretes to predict the shear strength of the connection. The
420 formulae in Eurocode 4 (EN 1994-1-1, 2004) can be used with a concrete of density not less
421 than 1750kg/m³, thus it deals with LCW, but not ULWC.

422 6. LOAD-SLIP BEHAVIOUR OF SHEAR CONNECTION

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

428 6.1 Load-slip models for headed shear stud connector

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

$$431 \quad \frac{P}{P_u} = (1 - e^{-18\delta})^{0.4} \quad (1)$$

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

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

$$437 \quad \frac{P}{P_u} = (1 - e^{0.55\delta})^{0.3} \quad (2)$$

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

$$442 \quad \frac{P}{P_u} = \frac{\delta}{0.5 + 0.97\delta} \quad (3)$$

441
443 Where δ is the slip in mm.

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

$$447 \quad \frac{P}{P_u} = \frac{2.24(\delta - 0.058)}{1 + 0.98(\delta - 0.058)} \quad \text{for NWC,} \quad (4a)$$

$$448 \quad \frac{P}{P_u} = \frac{4.44(\delta - 0.031)}{1 + 4.24(\delta - 0.031)} \quad \text{for HPC,} \quad (4b)$$

449 Where δ is the slip in mm.

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

$$452 \quad \frac{P}{P_u} = \alpha \sqrt{1 - e^{-\beta\delta/\alpha}} + \gamma\delta \quad (5)$$

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

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

459 6.2 Load-slip models for the two proposed shear connection systems

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

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

467 Based on the measured values and shape of the experimental push-out test curves, the
468 constitutive laws of Xue et al. (2008), Ollgaard et al. (1971), and Gattesco and Giuriani
469 (1996) were adopted for the theoretical analysis of the two proposed shear connection
470 systems.

$$471 \quad \frac{P}{P_u} = \frac{A\delta}{0.5 + B\delta} \quad (6a)$$

$$472 \quad \frac{P}{P_u} = (1 - e^{A\delta})^B \quad (6b)$$

$$473 \quad \frac{P}{P_u} = A\sqrt{1 - e^{-B\delta/A}} + C\delta \quad (6c)$$

474 Where A , B , and C are the coefficients.

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

483 For specimens with web-welded stud shear connection system:

484
$$\frac{P}{P_u} = \frac{4.02\delta}{1 + 4.16\delta} , \text{for NWC} \quad (7a)$$

485
$$\frac{P}{P_u} = \frac{0.98\delta}{1 + 0.96\delta} , \text{for LWC} \quad (7b)$$

486
$$\frac{P}{P_u} = \frac{1.92\delta}{1 + 1.77\delta} , \text{for ULWC} \quad (7c)$$

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

489
$$\frac{P}{P_u} = \frac{1.81\delta}{1 + 1.95\delta} , \text{for NWC} \quad (8a)$$

490
$$\frac{P}{P_u} = \frac{1.09\delta}{1 + 1.25\delta} , \text{for LWC} \quad (8b)$$

491
$$\frac{P}{P_u} = \frac{0.23\delta}{1 + 0.21\delta} , \text{for ULWC} \quad (8c)$$

492 **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

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

495 7.1 Existing design formulae for headed shear studs

496 Design codes are available to determine the shear capacity (P_{Rd}) of the headed shear stud
497 connectors. In Eurocode 4 (EN 1994-1-1, 2004), the shear strength of the headed shear studs
498 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 (≤ 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 $3 \leq h_s/d \leq 4$ or $\alpha = 1.0$ for $h_s/d \geq 4$, 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:

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

Where a_r' is the effective edge distance = $a_r - c_v - \emptyset_s/2 \geq 50$ 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/mm²; 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 $s/a_r' \leq 3$; \emptyset_s is the diameter of the stirrups with $\emptyset_s \geq 8$ mm, \emptyset_ℓ is the diameter of the longitudinal reinforcement with $\emptyset_\ell \geq 10$ 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:

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

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

$$P_s = \emptyset 0.5A_s \sqrt{f_{ck} E_c} \leq 0.75 f_u A_s \quad (12)$$

Where \emptyset 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:

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

526 Where d is the stud diameter.

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

$$529 \quad P_s = 1.106A_s f_c^{0.3} E_c^{0.44} \quad (14)$$

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

$$533 \quad P_{po} = \frac{1}{\eta} \cdot \chi_x \cdot (1 + \rho_{D,i}) \cdot 41 \cdot \sqrt{f_{ck}} \cdot h_{p0}^{1.5} \quad (15)$$

534 Where:

$$535 \quad \eta = 0.4 - 0.001 \cdot f_{ck}$$

$$536 \quad \chi_x = \frac{e_x}{4.5h_{p0}}$$

$$537 \quad \rho_{D,i} = \frac{A_{sf} E_s}{A_{D,i} E_c} = \frac{(A_b + A_t) E_s}{h_c - e_x - E_c}$$

$$538 \quad h_{p0} = \min(c_t + 0.07 \cdot e_x; c_b + 0.13 \cdot e_x)$$

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

545 7.2 Proposed formulae for connection system with web-welded studs and dowels

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

549 The shear strength (P_{sd}) from the web-welded shear studs and the one from the
550 horizontal lying dowels together with the web-welded shear studs was considered as an

551 independent variable. The f_c , d , and a_r were considered as dependent variables with respect to
552 the shear strength of the connection system.

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

$$555 \quad P_{sd} = 1.873(f_{ck} d a_r)^{0.835} \leq 0.8f_u A_s \quad (16)$$

556 Where P_{sd} is the shear resistance of shear stud or dowel, f_{ck} is the cylinder compressive
557 strength of concrete, d is the diameter of stud or dowel, and a_r is the distance from first stud
558 or dowel to the top of concrete, f_u is the ultimate tensile strength of the material of the stud or
559 dowel which should not be greater than 500N/mm², and A_s is the cross-sectional area of the
560 shear the stud or dowel.

561 7.3 Shear strength verification against test results

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

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

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

578 **8. CONCLUDING REMARKS**

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

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

$$602 \quad P_{sd} = \min(1.873(f_{ck} d a_r)^{0.835}, 0.8f_u A_s)$$

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

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

624 9. ACKNOWLEDGEMENTS

625 The authors would like to thank the academic staff and technicians of the Heavy Structures
626 Laboratory at the University of Leeds, and express their gratitude to Higher Committee For
627 Education Development in Iraq (HCED) for funding this PhD study at University of Leeds.
628 Gratitude is also extended to SC4 for the supply of the steel sections, Lytag and Leca for
629 supplying the lightweight aggregates.

630

631 10. REFERENCES

632

633 Tsavdaridis, K.D., D’Mello, C. and Hawes, M. 2009a. Experimental Study of Ultra
634 Shallow Floor Beams (USFB) with Perforated Steel Sections. The 11th Nordic Steel
635 Construction Conference 2009 (NSCC 2009). 2-4 September 2009, Malmö, Sweden,
636 Reference no: 128, pp. 312-319.

637 Tsavdaridis, K.D., D’Mello, C. and Huo, B.Y. 2009b. Computational Study Modelling
638 the Experimental Work Conducted on the Shear Capacity of Perforated Concrete-Steel
639 Ultra Shallow Floor Beams (USFB). The 16th National Concrete Conference. 21-23
640 October 2009, Paphos, Cyprus, Reference no: 201101, pp. 159.

641 AASHTO, 2004. LRFD bridge design specifications, 3rd edn. AASHTO, Washington,
642 DC.

643 Ahmed, I. and Tsavdaridis K.D. 2017. A New Breed of Sustainable Ultra-lightweight and
644 Ultra-Shallow Steel-Concrete Composite Flooring System: LCA. In: Proceedings.
645 Postgraduate Research Conference, Faculty of Engineering, 30-31 Mar 2017, Leeds, UK.
646 The University of Leeds.

647 Ahmed, I., Tsavdaridis, K. and Neysari, F. 2017. A New breed of sustainable ultra-
648 lightweight and ultra-shallow steel-concrete composite flooring system: life cycle
649 assessment of materials. CESARE’17 International Conference of Coordinating
650 Engineering for Sustainability and Resilience, Dead Sea, Jordan 3-8 May 2017.

651 Ahmed, I., Tsavdaridis, K.D. and Neysari, F. 2018. Push-Out Shear Tests for a Novel
652 Pre-fabricated Shallow Steel-Concrete Composite Flooring System. The 12th

653 International Conference on Advances in Steel-Concrete Composite Structures (ASCCS
654 2018). 27-29 June 2018, Valencia, Spain.

655 Ahmed, I.M. and Tsavdaridis, K.D. 2018. Life Cycle Assessment (LCA) and Cost (LCC)
656 Studies of Lightweight Composite Flooring Systems. *Journal of Building Engineering*,
657 20, pp.624-633.

658 Ahmed, I.M. and Tsavdaridis, K.D. 2019. The Evolution of Composite Flooring Systems:
659 and Applications, Testing, Modelling and Eurocode Design Approaches. *Journal of*
660 *Constructional Steel Research*. 155, pp. 286-300.

661 AISC. 1994. "Load and resistance factor design specification for structural steel
662 buildings", American Institute of Steel Construction, Inc., Chicago, September.

663 Alam, N., Nadjai, A., Maraveas, C. and Tsavdaridis, K.D. 2018. Response of Asymmetric
664 Slim Floor Beams in Parametric Fires. *European Symposium on Fire Safety Science*
665 (ESFSS 2018). 12-14 September 2018, Nancy, France.

666 Alam, N., Nadjai, A., Maraveas, C., Tsavdaridis, K.D. and Ali, F. 2018. Response of
667 Asymmetric Slim Floor Beams in Parametric Fires. *Journal of Physics: Conference*
668 *Series*. (1107) 032009.

669 Alam, N., Nadjai, A., Maraveas, C., Tsavdaridis, K.D. and Kahanji, C. 2018. Effect of
670 Air-gap on Response of Fabricated Slim Floor Beams in Fire. *Journal of Structural Fire*
671 *Engineering*.

672 Alam, N., Nadjai, A., Maraveas, C., Tsavdaridis, K.D. and Ali, F. 2018. Effect of Air-gap
673 on Performance of Fabricated Slim Floor Beams in Fire. *Ninth International Conference*
674 *on Advances in Steel Structures (ICASS 2018)*. 5-7 December 2018, Hong-Kong, China

675 An, L. and Cederwall, K., 1996. Push-out tests on studs in high strength and normal
676 strength concrete. *Journal of Constructional Steel Research*, 36(1), pp.15-29.

677 ANSI/AISC 2010. AISC 360-10, specification for structural steel buildings. American
678 National Standards Institute, Chicago.

679 ArcelorMittal ArcelorMittal Construction Benelux: Arval COFRADAL200®, A. C. B. A.
680 Cofradal.

681 Aribert, J.M. 1990. Dimensionnement de poutres mixtes en connection partielle. In *Mixed*
682 *structures including new materials*, IABSE symposium, Brussels, September, pp. 215-
683 220.

684 BS 1881-116:1983. Testing concrete. Method for determination of compressive strength
685 of concrete cubes.

686 Chinn, J. 1965. Pushout tests on lightweight composite slabs. *AISC Engineering Journal*,
687 2(4), pp.129-134.

688 Classen, M. and Hegger, J., 2017. Shear-slip behaviour and ductility of composite dowel
689 connectors with pry-out failure. *Engineering Structures*, 150, pp.428-437.

690 Classen, M. and Hegger, J., 2018. Shear tests on composite dowel rib connectors in
691 cracked concrete. *ACI Structural Journal*, 115(3), pp.661-671.

692 Classen, M., & Dressen, T. (2015). Experimental investigations on prestressed concrete
693 beams with openings. *ACI Structural Journal*, 112(2), 221.

694 Classen, M., Gallwoszus, J., & Hegger, J. (2014). Load-bearing behavior of an integrated
695 composite floor system. *Bauingenieur*, 89, 91-101.

696 Doel, A. 2007. Lightweight aggregates for use in concrete. *Concrete*, 41(7), pp.36-37.

697 EN 1992-1-1: Eurocode 2: Design of concrete structures: Part 1-1: General rules and rules
698 for buildings. European Committee for Standardization: 2004.

699 EN 1994-1-1: Eurocode 4: Design of composite steel and concrete structures, part 1-1:
700 general rules and rules for buildings. European Committee for Standardization: 2004.

701 EN 1994-2: Eurocode 4. Design of composite steel and concrete structures, part 2:
702 general rules and rules for bridges. European Committee for Standardization; 2005.

703 Frangi, A., Knobloch, M., Raveglia, E., Fontana, M., Mensinger, M., & Schwindl, K.
704 (2011). Composite slab with integrated installation floor using cellular beams.
705 In *Composite Construction in Steel and Concrete VI* (pp. 112-124).

706 Gattesco, N. and Giuriani, E. 1996. Experimental study on stud shear connectors
707 subjected to cyclic loading. *Journal of Constructional Steel Research*, 38(1), pp.1-21.

708 Hegger, J., Classen, M., Gallwoszus, J., Schaumann, P., Weisheim, W., Sothmann, J., &
709 Hargus, S. (2014). Multifunctional composite slab system with integrated building
710 services – Studies on the load bearing and fire behavior, thermal efficiency and
711 sustainability of a novel composite floor system. *Stahlbau*, 83(7), 452-460.

712 ISO 6892-1:2009. Metallic materials–tensile testing–Part 1: Method of test at room
713 temperature.

714 Johnson, R.P., Greenwood, R.D. and Van Dalen, K., 1969. Stud shear-connectors in
715 hogging moment regions of composite beams. *The Structural Engineer*, 47(9), pp.345-
716 350.

717 Johnson, R.P., Molenstra, N. 1991. Partial shear connection in composite beams for
718 buildings. *Proceedings of the Institution of Civil Engineers*, 91(4), pp.679-704.

719 Kansinally, R. and Tsavdaridis, K.D. 2015. Vibration Response of USFB Composite
720 Floors. The 13th Nordic Steel Construction Conference (NSCC 2015). 23-25
721 September, 2015, Tampere, Finland.

722 Kuhlmann, U. and Breuninger, U. 2002. Behaviour of horizontally lying studs with
723 longitudinal shear force. In *Composite Construction in Steel and Concrete IV*, pp. 438-
724 449.

725 Lorenc, W. and Kubica, E. 2006. Behavior of composite beams prestressed with external
726 tendons: Experimental study. *Journal of constructional steel research*, 62(12), pp.1353-
727 1366.

728 Maraveas, C., Fasoulakis, Z. and Tsavdaridis, K.D. 2017a. Fire Resistance of Axially
729 Restrained and Partially Unprotected Ultra Shallow Floor Beams (USFB) and
730 DELTABEAM Composite Beams. *Applications of Fire Engineering - Proceedings of the*
731 *International Conference of Applications of Structural Fire Engineering Conference*
732 *(ASFE'17)*. 7-8 September, 2017, Manchester, UK.

733 Maraveas, C., Tsavdaridis, K.D. and Nadjai, A. 2017b. Fire Resistance of Partially
734 Unprotected Ultra Shallow Floor Beams (USFB): A Numerical Investigation. *Fire*
735 *Technology*. 53(2), pp. 609-627.

736 Mazaheripour, H., Ghanbarpour, S., Mirmoradi, S.H. and Hosseinpour, I. 2011. The
737 effect of polypropylene fibres on the properties of fresh and hardened lightweight self-
738 compacting concrete. *Construction and Building Materials*, 25(1), pp.351-358.

739 MINITAB Release 18, 2017. Minitab Reference Manual. Minitab, State college,
740 Philadelphia.

741 Olgaard, J., Slutter R. and Fisher, J. 1971. Shear strength of stud connections in
742 lightweight and normal weight concrete, *AISC Engineering Journal*, pp. 55-64.

743 Precast Concrete Flooring, 2017. BISON Precast Ltd., August 2007.

744 Schaefer, M. (2015). Determination of moment resistance for composite shallow-floor
745 beams-Additional design rules for slim-floor beams. *STAHLBAU*, 84(4), 231-238.

746 Tsavdaridis, K.D. 2010. Structural Performance of Perforated Steel Beams with Novel
747 Web Openings and with Partial Concrete Encasement. Doctoral Thesis, City University
748 London, 2010.

749 Tsavdaridis, K.D. and Giaralis, A. 2011. Derivation of Dynamic Properties of Steel
750 Perforated Ultra Shallow Floor Beams (USFB) via Finite Element Modal Analysis and
751 Experimental Verification. The 7th Greek National Steel Structures Conference. 28
752 September – 1 October 2011, Volos, Greece, Reference no: 053, Vol. 2, pp. 321-329.

753 Tsavdaridis, K.D., D’Mello, C. and Huo, B.Y. 2013. Experimental and Computational
754 Study of Vertical Shear Behaviour of Partially Encased Perforated Steel
755 Beams. *Engineering Structures*. 56, pp. 805-822.

756 Xue, W., Ding, M., Wang, H. and Luo, Z. 2008. Static behavior and theoretical model of
757 stud shear connectors. *Journal of Bridge Engineering*, 13(6), pp. 623-634.

758 Yan, J.B., Liew, J.R., Sohel, K.M.A. and Zhang, M.H. 2014. Push-out tests on J-hook
759 connectors in steel–concrete–steel sandwich structure. *Materials and Structures*, 47(10),
760 pp. 1693-1714.

761 Yan, J.B., Liu, X.M., Liew, J.R., Qian, X. and Zhang, M.H. 2016a. Steel–concrete–steel
762 sandwich system in Arctic offshore structure: Materials, experiments, and design.
763 *Materials & Design*, 91, pp. 111-121.

764 Yan, J.B., Wang, J.Y., Liew, J.R., Qian, X. and Zhang, W. 2016b. Reinforced ultra-
765 lightweight cement composite flat slabs: Experiments and analysis. *Materials & Design*,
766 95, pp.148-158.

767 Yu Huo, B., D’Mello, C. and Tsavdaridis, K.D. 2010. Experimental and Analytical Study
768 of Push-out Shear Tests in Ultra Shallow Floor Beams. The 34th International Association
769 for Bridge and Structural Engineering Symposium (IABSE 2010). 22-24 September 2010,
770 Venice, Italy, IABSE Proceedings, pp. 31-38.