# 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

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/m$^3$.

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/m$^3$ and Leca with a low density of 280kg/m$^3$ 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).

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**Table 1:** Span limits for the ultra shallow flooring system

| Floor Type | Concrete Type | Concrete density kg/m³ | Maximum Span (m) | Overall Floor Depth (mm) | Total Floor Weight (kN/m²) | Live Load (kN/m²) | 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 |

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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.

## 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

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(NWC) | T1-NWC-1 |
| | Web-welded studs | Normal weight Concrete(NWC) | T1-NWC-2 |
| | Web-welded studs | Lightweight Concrete (LWC) | T1-LWC |
| | Web-welded studs | Ultra Lightweight Concrete (ULWC) | T1-ULWC |
| Group T2 | Dowels and Web-welded studs | Normal weight Concrete(NWC) | T2-NWC |
| | Dowels and Web-welded studs | Lightweight Concrete (LWC) | T2-LWC |
| | Dowels and Web-welded studs | Lightweight Concrete (LWC) | T2-LWC |
| | Dowels and Web-welded studs | Ultra Lightweight Concrete (ULWC) | T2-ULWC |

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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/m$^3$, respectively. The density of
225　the normal concrete was 2325kg/m$^3$ 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/m$^3$ 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.

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) |
|---|---|---|---|---|

255  **Table**

| | | | | | |
|---|---|---|---|---|---|
| Web-welded shear stud | 6.6 | 452.1 | 530.2 | 200 | 5: |
| Dowels | 19.83 | 322.5 | 455.5 | 200 | |
| Steel section | - | 406 | 570 | 200 | |

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 tesing. 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**.

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

13

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, $\delta_u$ (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 |

| | | | 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 |
| T1-ULWC | 20.0 | 1.36 | 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, $\delta_u$ (mm) | Stiffness, K, (kN/mm) | Ductility classification |
|---|---|---|---|---|---|---|---|
| | | | Right top dowel | 121.9 | 12.18 | 10.0 | Pass |
| | | | Right stud | 121.9 | 11.55 | 10.58 | Pass |
| T2-NWC | 37.3 | 2.45 | 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 |
| | | | Right top dowel | 101.65 | 22.10 | 4.59 | Pass |
| | | | Right stud | 101.65 | 21.50 | 4.72 | Pass |
| T2-LWC-1 | 34.6 | 2.11 | 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 |
| | | | Right top dowel | 103.51 | 22.20 | 4.66 | Pass |
| | | | Right stud | 103.51 | 21.00 | 4.92 | Pass |
| T2-LWC-2 | 36.8 | 2.12 | 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 |
| | | | Right top dowel | 73.83 | 31.90 | 2.31 | Pass |
| | | | Right stud | 73.83 | 30.70 | 2.40 | Pass |
| T2-ULWC | 20.0 | 1.38 | Right bottom dowel | 73.83 | 30.90 | 2.38 | Pass |

15

| | | | 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, $\delta$ 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 $\delta$ 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:

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

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

17

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:

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

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.

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

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

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

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

18

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:

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

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

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

487       For specimens with horizontally lying dowels together with web-welded stud shear

488    connection system:

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

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

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

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 <u>Existing design formulae for headed shear studs</u>

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:

$$499 \qquad P_s = \min\left(\frac{0.8 f_u \pi d^2/4}{\gamma_v}, \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_c}}{\gamma_v}\right) \qquad (9)$$

500   Where $f_u$ is the specified ultimate strength of the stud ($\leq$ 500 MPa), d is the diameter

501   of the stud, $\gamma_v$ is the partial factor (1.25), $f_{ck}$ is the concrete cylinder compressive strength, $E_c$

502   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

503   $h_s$ is the overall height of the stud.

504   In Annex C of Eurocode 4 (EN 1994-2, 2005), the shear strength of the horizontal

505   lying shear stud connector, which is responsible for the splitting in the direction of slab

506   thickness, is specified by:

$$507 \qquad P_s = \frac{1.4 k_v (f_{ck} d a_r')^{0.4} (a/s)^{0.3}}{\gamma_v} \qquad (10)$$

508   Where $a_r'$ is the effective edge distance = $a_r$ - $c_v$-$\varnothing_s/2 \geq$ 50 mm; $k_v$ = 1 for shear

509   connection in an edge position, $k_v$ = 1.14 for shear connection in a middle position; $\gamma_v$ is a

510   partial factor taken as (1.25), $f_{ck}$ is the characteristic cylinder strength of the concrete at the

511   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

512   the overall height of the headed stud with $h/d \geq 4$; a is the horizontal spacing of studs with

513   $110 \leq a \leq 440$ mm; s is the spacing of stirrups with both $a/2 \leq s \leq a$, and $s/a_r' \leq 3$; $\varnothing_s$ is the

514   diameter of the stirrups with $\varnothing_s \geq 8$ mm, $\varnothing_\ell$ is the diameter of the longitudinal reinforcement

515   with $\varnothing_\ell \geq 10$ mm, and $C_v$ is the vertical concrete cover.

516   In ANSI/AISC 360-10 (2010), the nominal shear strength of the headed studs

517   embedded in concrete is specified by:

$$518 \qquad P_s = 0.5 A_s \sqrt{f_{ck} E_c} \leq 0.75 f_u A_s \qquad (11)$$

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

520   calculated as:

$$521 \qquad P_s = \varnothing 0.5 A_s \sqrt{f_{ck} E_c} \leq 0.75 f_u A_s \qquad (12)$$

522   Where $\varnothing$ is the resistance factor for the shear connectors (=0.85).

523   Chinn (1965) proposed a formula for estimating the shear strength of headed shear

524   studs embedded in LWC. The shear strength of the headed shear studs is given as:

$$525 \qquad P_s = 39.22 d^{1.766} \qquad (13)$$

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   $$P_s = 1.106 A_s f_c^{`0.3} E_c^{0.44} \tag{14}$$

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

534   Where:

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

536   $\chi_x = \dfrac{e_x}{4.5 h_{p0}}$

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

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

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 <u>Proposed formulae for connection system with web-welded studs and dowels</u>
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:

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

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 $500N/mm^2$, 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 Ø0.5), 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
$$P_{sd} = \min\left(1.873(f_{ck}\, d\, a_r)^{0.835}\, , 0.8 f_u A_s\right)$$

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).

(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).

## 9. ACKNOWLEDGEMENTS

## 10. REFERENCES

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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