Experimental evaluation of the structural response of Perfobond shear connectors


Abstract

This paper presents an experimental study of the structural response of the Perfobond connector with various geometry configurations. The experimental programme was performed at the Civil Engineering Department, University of Coimbra, Portugal, and consisted of eight push-out tests conducted in accordance with the Eurocode 4 [18]. The investigated variables were the number of connector holes and the presence of reinforcing steel bars at the connector holes. In addition, the possible interaction of two Perfobond connectors side by side was also investigated. Other parameters such as the connector shape, concrete resistance and slab geometry kept constant values throughout this study. The experimental results are presented and discussed, focusing on the connection resistance, ductility and failure modes. Finally, the experimental results are compared to the existing analytical model formulae to predict the shear connector load carrying capacity.

1. Introduction

Structural construction and the use of new materials are the mirror of the human capacity of technological innovation. Composite structures are one of these examples of inventive solutions, by combining steel and concrete advantages: steel provides the resistance to tension stresses, while concrete is associated to a remarkable response to compressive stresses and enhances the stability and fire capacities of the steel section. Therefore, when designing a composite structure, these advantages should be fully taken into account, by properly proportioning and designing the structural sections.

These features, allied to the high speed construction solutions that these kinds of structures may provide, justify the trend for their extensive application in real civil engineering projects, namely in buildings and bridges. One of the most popular and extensively used solutions is the composite beam: the steel girder absorbs the tension stresses and the reinforced concrete slab sustains most of the compressive stresses.

To achieve this structural behaviour in a composite beam, it is well known that shear transfer between the steel profile and the concrete deck has to be provided. This composite action is usually assured by shear connectors.

The Nelson or Stud connector is the most extensively used and known shear connector, having the advantage of being automatically welded (Fig. 1(a)). Other widely used connectors may also be referred, like the Channel connector, the C connector (Fig. 1(b)), or the Hilti connector (Fig. 1(c)) fixed with an automatic specific device.

Some other alternative shear connectors account for the contribution of the mechanical interlock formed in holes or other indentations drilled in plates that are welded to the beam flange. This is the case of the Perfobond connector, studied in the present work. Fig. 2(a). Another type of connector, recently proposed by Vianna et al. [1, 2] is the T-Perfobond connector (Fig. 2(b)), associating the Perfobond connector to the T connector. In this innovative connector the holes are drilled not into a single plate but into the T-stub web plate.

Several authors studied in the past the behaviour and application of Perfobond connectors. Reference is made to the studies of Al-Darzi et al. [3, 4], Ferreira [5], Machácek & Studnika [6], Marecek et al. [7], Medberry & Shahrooz [8], Neves & Lima [9], Oguejiofor & Hosain [10, 11], Ushijima et al. [12], Valente & Cruz [13], Verissimo et al. [14], Vianna et al. [15, 1] and Zellner [16]. From these studies it could be concluded that the structural behaviour is governed by several parameters such as the number of holes, the plate height, length and thickness, the concrete compressive strength,
2. Analytical models to predict the strength of Perfobond connectors

Oguejiofor & Hosain [10] proposed a model to evaluate the resistance of a Perfobond shear connector based on push-out test results that adds three contributions for the overall resistance: the bearing concrete resistance at the connector face, the steel reinforcement bars in the concrete slab, and the concrete cylinders in shear that are formed through the connector’s holes—Eq. (1):

\[ q_u = 4.50 h_c t_{sc} f_{ck} + 0.91 A_{tr} f_y + 3.31 n D^2 \sqrt{f_{ck}} \]  

(1)

where: \( q_u \) —Perfobond connector nominal shear strength (N); \( D \) —hole diameter in the shear connector (mm); \( n \) —number of holes in the shear connector; \( h_c \) —Perfobond connector height (mm); \( t_{sc} \) —Perfobond connector thickness (mm); \( f_{ck} \) —concrete compressive strength (MPa); \( f_y \) —yield stress of the steel reinforcement bars provided in the concrete slab (MPa); \( A_{tr} \) —total area of transversal steel reinforcement provided in the concrete slab (mm²).

Medberry & Shahrooz [8] proposed a more general formula—Eq. (2) for the calculation of this strength, based as well on push-out test results:

\[ q_u = 0.747 b h_c \sqrt{f_{ck}} + 0.413 b f_y + 0.9 A_{tr} f_y + 1.66 n \pi \left( \frac{D}{2} \right)^2 \sqrt{f_{ck}} \]  

(2)

where: \( b \) —slab thickness (mm); \( h_c \) —slab length in front of connector (mm); \( b_f \) —steel section flange width (mm); \( L_c \) —contact length between the concrete and the flange of the steel section (mm).

Verissimo et al. [14] proposed Eq. (3) for this strength evaluation, based in Eq. (1) proposed by Oguejiofor & Hosain [10], and also supported by push-out tests:

\[ q_u = 4.04 \frac{h_c}{b} h_{sc} t_{sc} f_{ck} + 2.37 n D^2 \sqrt{f_{ck}} + 0.16 A_{cc} \sqrt{f_{ck}} + 31.85 \times 10^6 \frac{A_{tr}}{A_{cc}} \]  

(3)

where \( A_{cc} \) is the longitudinal concrete shear area per connector (mm²).

Al-Darzi et al. [3] proposed the model expressed by Eq. (4):

\[ q_u = 255.31 + 7.62 \times 10^{-4} h_{sc} t_{sc} f_{ck} - 7.59 \times 10^2 A_{tr} f_y + 2.53 \times 10^{-3} A_{cc} \sqrt{f_{ck}} \]  

(4)

where \( A_{sc} \) is the concrete area present at the connector holes.

An alternative formula was proposed by Ushijima et al. [12]—Eq. (5) to predict the individual holes contribution to the Perfobond connector resistance, \( q_{uho} \):

\[ q_{uho} = 3.38 D^2 \sqrt{\frac{t_{sc}}{D} f_{ck}} - 39 \]  

(5)

where: \( D \) —hole diameter in the shear connector (mm); \( t_{sc} \) —Perfobond connector thickness (mm); \( f_{ck} \) —concrete compressive strength in cylinder (MPa).

Finally, Marecek et al. [7] studied the case of two perforated connectors placed side by side and found that some interaction between them leads to an overall resistance that is less than the sum of the resistances of the individual connectors. In their study they proposed Eqs. (6) and (7) to predict this overall resistance \( P_{double} \) from the resistance of each individual connectors \( P_{rk} \) that are placed side by side with an axial spacing \( b \) (mm):

\[ P_{double} = k_d P_{rk} \]  

(6)

\[ k_d = 1.66 + \frac{b - 100}{14000} \leq 1.85. \]  

(7)

3. Experimental programme

3.1. Objectives

The eight push-out tests performed at the Civil Engineering Department of the University of Coimbra aimed at providing information concerning the general behaviour and suitability for practical applications of Perfobond connectors. In particular, a key issue dealt with was the contribution of the number of holes in the Perfobond plate and how this characteristic affects the connector shear resistance and ductility. Another investigated issue was the presence of transversal reinforcement passing through the Perfobond holes. Finally a test with two identical connectors, side by side, was carried out to evaluate the magnitude of the two Perfobond connectors interaction. In this configuration the axial spacing \( b \) of the two individual connectors was of 67 mm, dividing the beam flange into three equal parts.
Fig. 1. Shear connector examples.

(a) Studs. (b) C connector. (c) Hilti connector.

Fig. 2. Perfobond and T-Perfobond connectors [1].

(a) Perfobond connector. (b) T-Perfobond connector.

Fig. 3. Perfobond connector specimens overview.

(a) Connector with one hole (P1F). (b) Connector with four holes (P4F).

Table 1
Geometrical characteristics of the push-out tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Denomination</th>
<th>Concrete slab</th>
<th>Connector</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$t_c$ (mm)</td>
<td>$h_c$ (mm)</td>
</tr>
<tr>
<td>1</td>
<td>P0F</td>
<td>150</td>
<td>600</td>
</tr>
<tr>
<td>2</td>
<td>P1F</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>3</td>
<td>P2F</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>4</td>
<td>P3F</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>5</td>
<td>P4F</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>6</td>
<td>P1F AR12</td>
<td>12</td>
<td>20</td>
</tr>
<tr>
<td>7</td>
<td>P1F AR20</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>8</td>
<td>2P1F</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>
3.2. Mechanical and geometrical properties of the push-out specimens

An overview of the experimental programme is given in Table 1. The specimens were fabricated in accordance to the Eurocode 4 [18] Annex B, and their geometry may be depicted in Table 1, Figs. 3–5. The geometry of the connectors respects a 2.25 D distance between holes, based on Ogugjejofor & Hosain [10] recommendations.

In Table 1, tc represents the concrete slab thickness (mm), hc the concrete slab width (mm), l the connector length (mm), h the connector height (mm), t the connector thickness, AR the transverse reinforcement located at the connector holes (mm), D the connector holes diameter (mm) and n the number of connector holes.

All the connectors were made from S355 steel plates (355 MPa nominal tensile yield stress, according to EN10025). The adopted steel section was a HEA 200 in S275 (275 MPa nominal tensile yield stress, according to EN10025). The slab reinforcement was made of 10 mm diameter bars (φ10) in S500 (500 MPa nominal tensile yield stress, according to EN1992-1-1 [19]), as prescribed by the Eurocode 4, Annex B [18].

All specimens presented a similar concrete compressive strength, since they were all made from the same admixture and were tested after 28 days at a closely distanced times, Table 2. The concrete compressive strength, obtained from cubes at the ages of 7, 14, 28 days, and at the same age of the push-out tests is pre-
Table 2
Concrete cube compressive strength $f_{cm}$.

<table>
<thead>
<tr>
<th>Connector/test</th>
<th>Age (days)</th>
<th>$f_{cm}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>POF</td>
<td>32</td>
<td>31</td>
</tr>
<tr>
<td>P1F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P2F</td>
<td>34</td>
<td>31</td>
</tr>
<tr>
<td>P3F</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P4F</td>
<td>42</td>
<td>31</td>
</tr>
<tr>
<td>P1F12AR</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P4F</td>
<td>42</td>
<td>31</td>
</tr>
<tr>
<td>P1F12AR</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Thickness of all connectors 15 mm

Fig. 5. Geometrical characteristics of the tested connectors (dimensions in mm).

Table 2 contains the test results. The tested concrete stress corresponds to a nominal C25/30 class according to Eurocode 2 [19].

3.3. Experimental protocol

The experimental tests were loaded in two stages according to Eurocode 4, paragraph B.2.4 [18]. At the first stage the load was applied in 25 cycles of loading/unloading ranging from 5% up to 40% of the expected failure load. At this stage the procedure was controlled by an applied load at a rate of 5 kN/s. At the subsequent stage and up to the specimens failure, the control parameter was the relative displacement between the steel and the concrete, reaching at least a point where the descending load after the peak load was equal to 80% of the peak load.

The connector slip capacity $\delta_u$ can be evaluated from the measured value at the characteristic load level ($P_{Rk}$), as specified in Eurocode 4 [18]. The characteristic load is taken as the least collapse load, divided by the number of connectors, and reduced by 10%. The characteristic slip is $\delta_{uk}$ and should be taken as 90% of the connector slip capacity i.e. $0.9\delta_u$.

3.4. Test layout and instrumentation

In Fig. 6 the test layout is illustrated, showing the loading device: a 5000 kN hydraulic testing machine. Neoprene sheets were placed at the base of the specimen to absorb any imperfections present at the concrete bottom face and to reduce friction, as recommended by Iwasaki et al. [20].

The concrete slab and steel profile relative displacement and the uplift were measured by load–displacement transducers (LVDTs), Fig. 7. The LVDT’s readings were able to provide the force–slip and the force–uplift curves for both connectors.

4. Experimental results

4.1. General results and force–slip curves

Table 3 summarises the test results. The first column identifies the test, the second column shows the test maximum load (peak load) $P_{test}$, the third column presents the test characteristic load $P_{Rk}$, while the fourth column contains a normalized value of $P_{Rk}$, designated as $P_{Rk,norm}$, that corrects the age variability of the concrete compressive strength. However, as indicated in Table 2, the age variability of the concrete strength in the present case is very small. The last two columns present, respectively, the connector slip capacity, $\delta_u$ and characteristic slip, $\delta_{uk}$.

An important conclusion to be drawn is the adequate ductility of all connectors, since they present, in most cases, a slip capacity larger than 6 mm, the criterion stated in Eurocode 4 [18] for considering a ductile connector behaviour. An exception to this conclusion is the test 2PF1 where two connectors were tested, side by side, that showed a much smaller slip capacity, and therefore an insufficient ductility.

The typical behaviour for the set of tests of single Perfobond connectors without reinforcement is depicted in Fig. 8. One main feature of these tests is that the onset of cracking is identified in the force–slip curve by a drop in the connector stiffness at the corresponding load level. In this figure, each test curve corresponds to the average of the two measured curves, each of them corresponding to connector slabs A and B.

The typical behaviour of the connections for the “uplift” displacement is illustrated in Fig. 11, being substantially smaller than the steel to concrete slip.

4.2. Influence of the number of holes in the connector

The force–slip curves for the connectors with a number of holes varying from zero to four are plotted in Fig. 8. From this figure it may be observed that all the tests present an adequate ductility according to the Eurocode 4 [18] criterion, and that an increase of the number of holes enhances the connector load carrying capacity. These facts were already highlighted in Section 4.1 when referring to Table 3, where the relevant results are listed. The gain in resistance may be observed in Fig. 9 where the results for the different tests and their trend are plotted. Also, the relative gain comparing to the bare plate connector is highlighted in Fig. 10. The relative gain when an additional hole is added to a connector shows that one more hole in the plate leads to an average resistance increase of about 5%.

As far as the stiffness is concerned, Fig. 8 shows that all connectors present approximately the same stiffness up to their maxi-
Fig. 7. Instrumentation layout: LVDTs arrangement.

Table 3  
Perfobond connector push-out test results.

<table>
<thead>
<tr>
<th>Connector/test</th>
<th>( P_{\text{ex}} ) (kN)</th>
<th>( P_{\delta} ) (kN)</th>
<th>( P_{\text{rel, norm}} ) (kN)</th>
<th>( \delta_{u} ) (mm)</th>
<th>( \delta_{uk} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P0F A</td>
<td>283.51</td>
<td>255.16</td>
<td>252.99</td>
<td>15.34</td>
<td>13.81</td>
</tr>
<tr>
<td>P0F B</td>
<td></td>
<td></td>
<td></td>
<td>15.32</td>
<td>13.79</td>
</tr>
<tr>
<td>P1F A</td>
<td>309.44</td>
<td>278.49</td>
<td>276.13</td>
<td>9.50</td>
<td>8.55</td>
</tr>
<tr>
<td>P1F B</td>
<td></td>
<td></td>
<td></td>
<td>10.15</td>
<td>9.14</td>
</tr>
<tr>
<td>P2F A</td>
<td>317.52</td>
<td>285.77</td>
<td>284.30</td>
<td>17.80</td>
<td>16.02</td>
</tr>
<tr>
<td>P2F B</td>
<td></td>
<td></td>
<td></td>
<td>17.44</td>
<td>15.70</td>
</tr>
<tr>
<td>P3F A</td>
<td>331.35</td>
<td>298.22</td>
<td>296.69</td>
<td>10.76</td>
<td>9.68</td>
</tr>
<tr>
<td>P3F B</td>
<td></td>
<td></td>
<td></td>
<td>10.52</td>
<td>9.47</td>
</tr>
<tr>
<td>P4F A</td>
<td>354.03</td>
<td>318.627</td>
<td>317.40</td>
<td>10.00</td>
<td>9.00</td>
</tr>
<tr>
<td>P4F B</td>
<td></td>
<td></td>
<td></td>
<td>9.44</td>
<td>8.50</td>
</tr>
<tr>
<td>P1F-AR 12 A</td>
<td>365.93</td>
<td>329.34</td>
<td>328.07</td>
<td>10.28</td>
<td>9.25</td>
</tr>
<tr>
<td>P1F-AR 12 B</td>
<td></td>
<td></td>
<td></td>
<td>10.42</td>
<td>9.38</td>
</tr>
<tr>
<td>P1F-AR 20 A</td>
<td>395.68</td>
<td>356.11</td>
<td>355.02</td>
<td>11.10</td>
<td>10.30</td>
</tr>
<tr>
<td>P1F-AR 20 B</td>
<td></td>
<td></td>
<td></td>
<td>11.10</td>
<td>9.99</td>
</tr>
<tr>
<td>2 P1F A</td>
<td>546.84</td>
<td>492.16</td>
<td>490.65</td>
<td>2.96</td>
<td>2.66</td>
</tr>
<tr>
<td>2 P1F B</td>
<td></td>
<td></td>
<td></td>
<td>2.02</td>
<td>1.82</td>
</tr>
</tbody>
</table>

Fig. 8. Load–displacement curves for different number of connector holes.

From these results it may be concluded that the contribution of the concrete cylinders formed through the holes is not dominant for the connector strength and stiffness when compared to other contributions, like the bearing of the plate or the slab reinforcement. Although, it could also be concluded that at least one hole is essential to guarantee an adequate behaviour.

4.3. Effect of two connectors placed side by side

When a pair of Perfobond connectors with one hole are placed side by side with an axial spacing of \( b = 67 \) mm, the resulting connection (test 2P1F) exhibits the behaviour shown in Fig. 12. Also in this figure a direct comparison is made to the test P1F,
Fig. 10. Influence of the number of connector holes over their associate load carrying capacity (relative values).

Fig. 11. Load–uplift curves for various numbers of connector holes.

Fig. 12. P1F vs. 2P1F load–displacement curves.

Fig. 13. P1F vs. P1F AR12 and P1F AR20 load/displacement curves.

Fig. 14. Uplift displacement for unreinforced and reinforced shear connectors.

with only one connector of the same geometry. For each case the characteristic load $P_{Rk}$, resulting from the application of Eurocode 4 criterion is also illustrated by a horizontal line.

Referring to Table 3, the peak load of the connection 2P1F is of 546.84 kN, that is 1.77 times the peak load of the connection P1F. A similar relation is observed for the characteristic load $P_{Rk,norm}$ of the connections (490.65 kN for 2P1F and 276.13 kN for P1F). It may be concluded that some interaction between the two adjacent connectors is preventing this configuration from reaching twice the resistance of the single connector configuration. Although more tests are needed to confirm this assertive, the connector front contribution and the holes contribution to the global connector load carrying capacity is compromised by the close arrangement of the pair of Periflode connectors.

This conclusion is in agreement with the conclusions of the study performed by Marecek et al. [7], and referred in paragraph 2. Application of Eqs. (6) and (7) leads to a factor $k_d = 1.66$, and is for this particular situation a safe assessment of the interaction between connectors, with an error of about 6%.

4.4. Effect of steel reinforcing bars within the holes

Fig. 13 compares the force–slip curves of the connector with one hole without reinforcement (P1F) and with reinforcing bars within the holes as detailed in Fig. 4(c) (12 mm diameter; P1F AR12 and 20 mm diameter; P1F AR20). Again, the characteristic load $P_{Rk}$ from the application of Eurocode 4 criterion is also illustrated by a horizontal line.

It may be concluded that adding reinforcement enhances the connector capacity (Table 4 shows a resistance gain of 19% and 29% respectively for 12 mm and 20 mm bars) and its associated ductility and at the same time reduces the uplift displacement at the cracked stage (Fig. 14).

4.5. Failure mode analysis

The failure mode is similar for all the tests without reinforcement passing through the holes—Figs. 15 and 16. For these tests, the failure mode is characterized in the concrete slab lower side by the formation of a vertical crack along the concrete slab moving...
forward and growing thicker as the loading increases up to their maximum. Subsequently to the initiation of cracking there is a concrete crushing at the loaded edge of the connector. Roughly at the same time there is further crack opening at the inside surface of the concrete. This cracking is located at a region of the connector moving upward at 45° angle. The 45° cracking is less visible, or is even imperceptible, for the tests with hole reinforcement, (P1F AR12) and (P1F AR20).

5. Comparison of the experimental results to the analytical failure load models

In this section the test results are compared to the ultimate load values predicted by the previously presented analytical models to enable a discussion of the models accuracy and applicability.

As a general remark, all models state that an increase of the number of holes corresponds to an increase of the connection’s strength. This is in line with the experimental findings presented in Figs. 9 and 10.

The model proposed by Oguejiofor & Hosain [10] uses Eq. (1) to evaluate the resistance of a Perfobond shear connector adding three contributions: the first is the bearing concrete resistance at the connector face; the second comes from the steel reinforcing bars present in the concrete slab, computed from Eq. (1) considering that the amount of reinforcement \( A_r \), is any reinforcement passing in the connectors holes plus four transverse bars from the slab that are in front of the connector (see Fig. 4); and the third comes from the concrete cylinders in shear that are formed through the connector’s holes. In Fig. 17 the experimental results expressed as the normalized characteristic resistance \( P_{k,norm} \), are compared to the analytical results from Eq. (1). It may be concluded that this model leads to an unsafe prediction of the characteristic load, with an error ranging from 17% to 20% for connectors without reinforcement at the connector holes. For the other tests with reinforcement at the connector holes, the error is equal to 14% for 12 mm rebars and to 32% for 20 mm rebars (also on the unsafe side). It may therefore be concluded that this model overestimates the resistance of the studied connector’s configurations, due most likely to an overestimation of the reinforcement contribution, thus giving wrong results for higher reinforcement ratios.

These results are in line with the conclusions from Vianna et al. [2], where the Oguejiofor & Hosain’s model used to compute the resistance of Perfobond connectors without reinforcement in the holes was reported to overestimate this feature up to 16%.

The Medberry & Shahrooz [8] model results in the application of Eq. (2) and takes into account roughly the same contributions of the first model plus the steel to concrete bond. Again, the amount of reinforcement \( A_r \) was computed as in the previous model. The application of this model leads to the comparison established in Fig. 18, from which it may be observed that the analytical results overestimate the experimental characteristic load. In fact, for the connectors without rebars the error is always on the unsafe side and varies from 32% (without holes) to 13% (four holes). Also, this error decreases with the increasing number of connector holes, suggesting that the term reflecting the contribution from the holes should most likely be modified. Compared to the previous model, Eq. (2) produces in this case a slightly better approximation for the connectors of three or more holes, and a worse approximation for a smaller number of holes. For the connectors with rebars, the estimation is also unsafe, with an error of 17% for 12 mm rebars and of 35% for 20 mm rebars, having for this case roughly the same accuracy as the previous model. Again, these findings are in line with the conclusions from Vianna et al. [2], where the Medberry & Shahrooz’s model was reported to overestimate in up to 16% the resistance of Perfobond connectors without reinforcement in the holes.

The comparison between experimental results and the Verísimo et al. [14] analytical results obtained from Eq. (3) is also shown in Fig. 19, again with \( A_r \) computed as before. This model leads to a very reasonable estimation of the actual load, slightly overestimating the resistance for connectors without hole rebars with an error of up to 11%. For the connectors with rebars in the holes the model accurately predicts the resistance for 12 mm rebars and leads to an 11% resistance overestimation for 20 mm rebars. It may therefore be concluded that this model leads generally to a reasonable prediction of the connectors resistance. The study performed by Vianna et al. [2] reported a similar accuracy for this model (8% average strength overestimation).
The model proposed by Al-Darzi et al. [3], expressed by Eq. (4), is depicted in Fig. 20, and overestimates the resistance for the connectors without rebars, with a maximum error of 12%. For the connectors with hole rebars this model is on the safe side, with an error equal to 11% for 12 mm rebars and equal to 18% for 20 mm rebar. Again, the steel reinforcement $A_r$ was computed as in the previous models.

The model proposed by Ushijima et al. [12]—Eq. (5), does not predict the overall connector resistance, but only the individual hole contribution. Therefore in Fig. 21 a comparison is made between the value of 53.7 kN given by Eq. (4) for the expected gain of resistance from each additional hole, and the effective increase of resistance, obtained experimentally when the number of holes was incremented. It may be concluded that the Ushijima et al. [12] model largely overestimates the contribution from the holes, with an error of up to 6.5 times the experimental value.

6. Conclusions

This paper presented and discussed a set of results from eight push-out tests conducted at the Civil Engineering Department of University of Coimbra, Portugal. The main investigated variables were the number of holes drilled in the Perforbond connector, and the amount of reinforcement in the form of rebars passing through the connectors holes. Additionally, a configuration using a pair of Perforbond connectors, placed side by side, was also studied. In all tests the concrete resistance was kept constant to eliminate the influence of this parameter from the investigated test programme.

A special attention was given to the strength and ductility of the studied shear connectors. In fact, most of the tests have shown that the geometry used for the connectors meet the Eurocode 4 [18] ductility requirement of a 6 mm minimum slip capacity. The exception to this rule was the two connectors P1F side by side (test 2P1F) configuration. On the other hand, this configuration presents a large increase of the connector load carrying capacity. This high capacity is one of the key features of this type of connectors, since one single connector may replace a considerable number of the traditional studs. As an example, at least four 19 mm stud connectors (with a resistance of 74 kN each according to Eurocode 4 [18]) may be replaced by a single one hole Perforbond connector with a 12 mm rebar passing through this hole (configuration of test P1FAR12).

It was shown that that adding more holes to the connector increases the resistance, and that for each added hole the mean gain of resistance was around 5%. It was also concluded that the contribution of the concrete cylinders formed through the holes is not dominant for the strength when compared to other contributions, like the bearing of the plate or the slab reinforcement. Although, it is reasonable to observe that at least one hole is essential to guarantee an adequate behaviour and to prevent an undesirable uplift. This uplift was shown to remain controlled and of the same magnitude for all the perforated connectors. As far as the stiffness is concerned, it was shown that all the isolated perforated connectors without rebars present approximately the same stiffness up to their maximum load.

Enhanced resistance and ductility may be achieved by passing reinforcing bars in the holes, and providing this reinforcement led to a resistance increase of about 20% and 30% for 12 mm and 20 mm rebar, respectively. These rebars were also shown effective to further reduce the uplift displacement.

When two identical connectors are used side by side, some interaction between them prevents this configuration from reaching twice the resistance of the single connector configuration. Furthermore, the twin connector detailing has a much smaller ductility than the single connector connection, not enough for the connection to be classified as ductile according to the previously referred criterion.
The shear connector failure mode was associated to the concrete crushing by the bearing plate, since the connectors were conservatively designed to stay in the elastic domain for the predicted ultimate test loads. As a consequence of this failure mode an extensive cracking of the specimens could be observed in all tests at advanced load stages. It was also shown that providing reinforcement at the connector holes inhibits some of the concrete cracking, and changes the cracking pattern during the test.

When comparing the test results to the available analytical models, and referring to Table 5, it was concluded that the models proposed by Oguejiofor & Hosain [10], Medberry & Shahroz [8] and Veríssimo et al. [14] lead to predictions on the unsafe side with an error ranging from 2% to 32%, for tests without hole reinforcement. These results are in agreement with the conclusions published by Vianna et al. [2]. When reinforcement is added, the first two models still lead to unsafe predictions and the third model leads to a quite accurate prediction of the failure load for 12 mm rebars. However, the three models lead to unsafe predictions when the rebar diameter increases to 20 mm.

The model proposed by Al-Darzi et al. [3] provides a safe and reasonable estimation of the reinforced connector’s strength, but is on the unsafe side for unreinforced connectors with a maximum error of 12%.

Finally, the model proposed by Ushijima et al. [12] to predict the contribution of the individual holes on the overall connector resistance largely overestimates the observed values, with an error of up to 6.5 times the experimental value.

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