Available online at www.sciencedirect.com

ScienceDirect

Procedia Structural Integrity 13 (2018) 2024–2029

Structural Integrity Procedia

www.elsevier.com/locate/procedia

ECF22 - Loading and Environmental effects on Structural Integrity

Experimental characterisation of Perfobond shear connectors through a new one-sided push-out test

Mohammed A. Al-Shuwaili [a], Alessandro Palmeri[a], Mariateresa Lombardo[a]

[a]School of Architecture, Civil and Building Engineering, Loughborough University, Loughborough, LE11 3TU, UK

Abstract

In steel-concrete composite beams, the perfobond shear connectors (PSCs) are commonly utilised as an alternative to the widely used headed studs, as the latter have limited shear capacity and susceptible to fatigue problems. The structural assessment of the PSCs is typically obtained experimentally, and mainly through a type of destructive test known as push-out test (POT). POT specimen typically consists of an I- steel section attached to two concrete slabs through the connectors under investigation, the slabs are then simultaneously tested by the application of a direct shear force to the steel section until the fracture of the specimen is reached. The shear strength of PSCs can be evaluated from the POT results. However, the weaker of the two concrete slabs tend to fail before the other side, which thus inevitably affects the results. In this paper, an efficient one-sided POT (OSPOT) is used to characterise the behaviour of the PSCs in composite steel-concrete beams. POT and OSPOT specimens are similar, but the shear force in the OSPOT is directly applied to one slab each test. As a part of this study, ten OSPOT have been carried out to investigate the behaviour and the shear resistance of the PSC. The results were compared against POT results from other researchers and the predictions offered by several shear resistance equations. It has been found that the OSPOT results are consistent with the analytical predictions offered by these expressions compared to the previous research using POT. Among the key advantages of the proposed OSPOT procedure: similar to the traditional POT, it is possible to quantify the relationship between applied loads and displacements in the shear connectors, which is the most important information for the structural design of composite steel-concrete beams; it is effectively doubled the number of results for the same research resources; the fabrication of the samples is simplified.

Peer-review under responsibility of the ECF22 organizers.

Keywords: shear connectors, push-out test, perfobond connectors, one-side push-out test, composite beams

1 Introduction

The perfobond shear connector (PSC) is a rectangular steel plate welded typically to the top flange of the steel girder. The plate has circular drilled holes which allow the concrete of the slab, more often with the rebars, to pass and forming reinforced concrete dowels which enable the slab and the girder to act compositely as one structural element. The PSCs have been used as an alternative for the widely used headed studs shear connectors due to their high resistance to shear stresses and fatigue problems (Su et al., 2016). The PSCs are employed in the construction industry such as bridges; composite joints of hybrid bridges which employ the combination of steel girders and concrete girders (Xiao et al.,2016); foundations to strengthen the steel pile connection with the pile cup (Kim et al.,2016).

The conventional POT, shown in Fig. 1(a), has been mainly used to define experimentally the structural performance of PSCs. In the POT test, the PSCs are welded to an I- steel section and then cast into the concrete slabs to secure the connection between the two slabs (blocks) and steel section. By applying the load directly to the latter, the slabs are loaded simultaneously. However, many researchers have argued about the POT testing procedure, e.g. Ernst (2006) and Valente (2007), as the weaker of the two concrete slabs usually fails before the other side despite the two slabs are, in fact, identical twins. Hence, the result represents the average of two connectors and not the individual PSC. Further, due to the POT setup, the shear connector resists the applied shear force along with a normal force inducing frictional reactions under the concrete slabs. Besides affecting the test's result, this normal force does not exist usually in composite concrete-steel beams.

In this paper a new one-side POT is used to characterize the behaviour of the PSCs which attach the concrete slab to the steel girder, wherein the aleatory uncertainties are reduced as one slab is to be tested each time and not two; the frictional reaction is eliminated as the slab is free to move in the load direction; two results can be obtained from one POT specimen.

2452-3216 © 2018 The Authors. Published by Elsevier B.V.
Peer-review under responsibility of the ECF22 organizers.
10.1016/j.prostr.2018.12.315

2 Experimental campaign

2.1 Push-out test vs one-sided push-out test

The OSPOT specimen is similar to the conventional POT, see Fig. 1(a,b), however, the POT setup requires that the specimen is supported by the concrete slabs during the test while the steel section moves vertically in load direction. Thus, the slabs are loaded indirectly by the shear force between the steel section and the concrete slab in both sides of the specimen and resistance by the PSCs. In the OSPOT, the steel section supports the specimen, see Fig. 1(b), and the load is applied to one of the concrete slabs each time, wherein the slab is free to move in the load direction. The load is distributed uniformly through thick plate creating a longitudinal shear stress along the steel-concrete interface. Since the applied load is eccentric in the OSPOT setup, a steel frame consisting of steel box sections is assembled to prevents the specimen from leaning forward or sliding at the base during the test.

2.2 Fabrication of samples

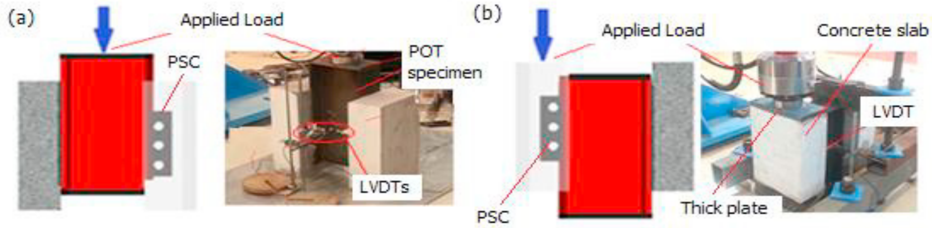

Fig. 1. (a) The conventional POT setup; (b) the OSPOT setup.

The experimental campaign involves the execution of ten OSPOTs. The parameters which were considered in this OSPOT study included the rib position and age of the specimen, beside the rebars existence, their layout and distribution. The POT specimen recommended by BS-5 with one PSC each side was adopted in these investigations due to the actuator loading capacity (about 515kN) and the size and economy of the BS-5 specimen compared to the other specimens such as EC-4. The PSC dimensions are (120mm height x150mm length x10mm thickness) and has one centred circular hole of 80mm in diameter. Each specimen consisted of a 560 mm long universal beam (254 x146 x43 UB) attached to two concrete slabs. The slab dimensions are 460mm height x300mm width x150mm thickness.

Four perfobond ribs were welded to the end of the steel section, see Fig. 2 (a,c), to make the PSCs as close as possible to the applied load. The other six PSCs were welded in such a way that the rib is located at the centre of the concrete slab, i.e. as the length of the rib is 150 mm and the height of the slab is 460mm, the rib was welded at the middle third of the slab as shown in Fig. 2(b).

For data registrations of this study, P is the un-reinforced connector for the middle PSCs see Fig. 2(b). P-ds for the reinforced samples and ds is the diameter of the transverse steel rebar. Extra information was added such as the position of the rib and the age when appropriate. For example, P-end rib is the specimen shown in Fig. 2(a).

Two steps of monotonic loading were adopted for all the tests. The vertical load was applied in a cyclical manner from zero up to 50% of the estimated failure load for 5 times at a load controlled (1 kN /sec). After that, the loading application mode was changed from loading control to displacement controlled at a rate of (0.03mm/sec) starting from zero loading until failure. The subsequent loading was imposed in such a way that failure occurs after more than 15 minutes from the start of the test to comply with both the EC-4 and BS-5 recommendations.

The relative movement in the direction of load between the steel section and the concrete slab, i.e. slip, is continuously measured, through a load cell, against the applied load to plot a load-slip curve to assess the structural performance of the shear connector in term of the connector shear resistance, ductility and elastic stiffness. Also, two LVDTs are positioned to record the separation between the steel section and the concrete slab in the OSPOT instead of one each side in the POT, see Fig. 1.

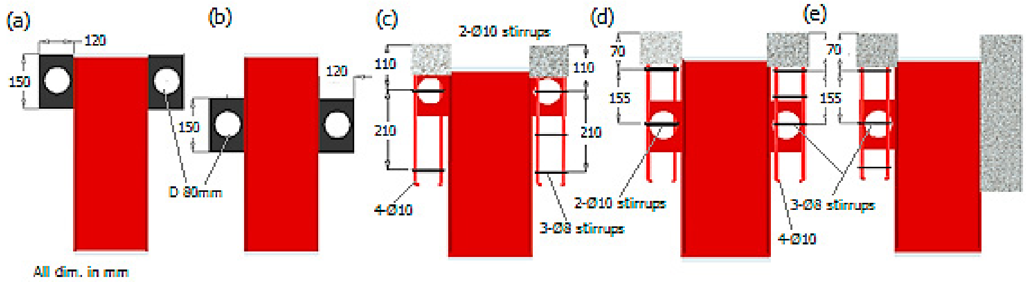

Fig. 2. The OSPOTs specimens

2.3 Specimen detailing

Five specimens were prepared, i.e. ten OSPOTs, for this testing campaign; two are un-reinforced specimens, see Fig. 2 (a,b) while the other three are reinforced. Fig. 1(c) shows specimen three in which two of the PSCs were welded to the end of the steel section similar to the first specimen shown in Fig. 1(a). In the first slab of specimen three, see Fig. 2(c), the upper Ø10 stirrup was positioned at the centre of the rib and the lower Ø10 stirrup was located at 210mm centre to centre from the upper stirrup. The second slab was reinforced by 3-Ø8 stirrups, wherein the first stirrup was located also at the centre of the hole. The other two stirrups were distributed uniformly along a 210 mm downward in the slab.

The PSCs in specimens four and five were located at the centre of the slab similar to Fig. 2(b). In the fourth specimen, shown in Fig. 2(d), the lower Ø10 stirrup, in the first slab, was positioned at the hole centre and the upper Ø10 stirrup was located at 155mm centre to centre towards the load direction. In the other side, the lower 3-Ø8 was positioned at the hole centre, and the upper stirrup is located at 110mm from the top face of concrete while the third stirrup was distributed at the middle. Therefore, specimens three and four have the same steel ratio but in different distribution as the cross-sectional area of 2-Ø10 is nearly the same area of 3-Ø8 rebars.

Specimen five, shown in Fig. 2(e), was reinforced only by 3-Ø8 stirrups in both sides. The second stirrup was positioned at the hole centre and the first and the third stirrups were located at 70mm from the upper and the lower faces of the concrete, thus the stirrups were uniformly distributed.

All the reinforced specimens were supplied by 4- Ø10 longitudinal rebars that were located inside the corners of the transverse stirrups. The concrete compressive strength for the 7day samples was 29.4 Mpa, and for 28day samples was 31.6 Mpa. A concrete cover of 25mm was ensured for all the stirrups and 30mm for the longitudinal bars. All the rebars are grade B500B conforming to BS4449-2005.

3 Results and Discussion

3.1 The un-reinforced campaign

Fig. 3(a) shows the load-slip curve of the first specimen. Starting from the early stage of loading until the maximum load there is no clear difference between the two load-slip curves of these *identical twin* slabs.

The structural behaviour of the un-reinforced concrete dowels is similar along the elastic range until the 140kN of shear resistance, at about 3 mm slip. The slip between the steel plate and the concrete block at the linear stage is normal for such un-reinforced PSC. Fig. 3(b) shows the load-slip curves to the second un-reinforced specimen. The first test was carried on at age of 7 days where the concrete compressive strength was 29.4 Mpa whereas the second tested was executed at age of 28 days. The compressive strength at the latter test was 31.6 Mpa which is nearly the same compressive strength of the 7day sample.

Although the two specimens have a minor difference in the compressive strength, the maximum shear resistance is considerably different, about 50kN, which represents a 1/3 increase in the shear resistance of the 28day sample, see Fig. 3(b). This shows that the age of testing is a significant factor, especially for the early age samples. Researchers, especially in the large testing campaigns, cast all the samples at the same time but the testing of these samples could happen at several stages, therefore, the effect of testing age might be important to consider. However, this effect might be less for the samples after 28 days as the water moisture content of the concrete is less. According to the results of this test, it is not recommended to test the un-reinforced POT samples before four weeks.

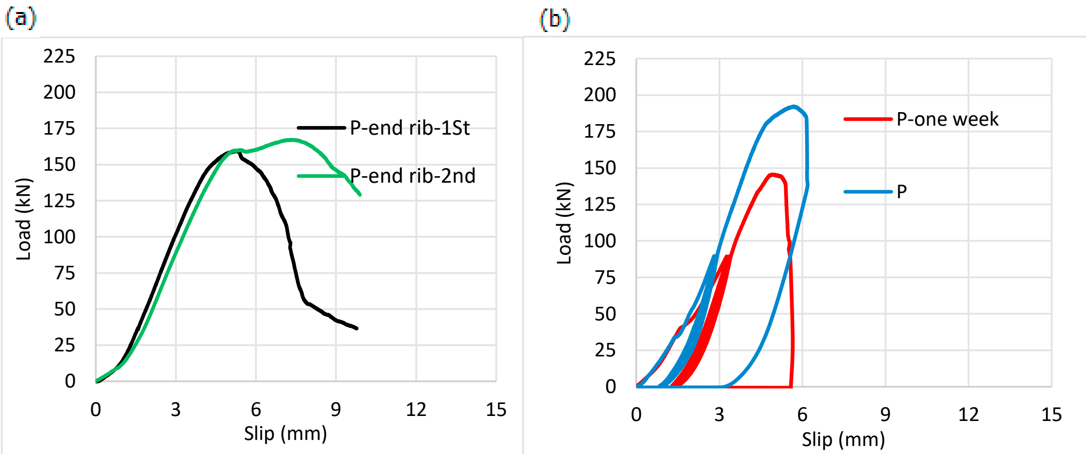

Fig. 3. The load-slip curves of the un-reinforced specimens.

3.2 The reinforced campaign

Fig. 4(a) shows the load-slip curves for the twins, but not identical, from specimen three shown in Fig. 2(c). The elastic behaviour of both samples is identical from the start until nearly the maximum load. When the relative slip has exceeded 10 mm, the connector reached its maximum shear capacity. It is worth mentioning that between 4 and 10 mm of slip the behaviour of both ribs converted to non-linear behaviour, i.e. elasto-plastic stage, in which the load had increased by 55 kN from the end of the elastic range.

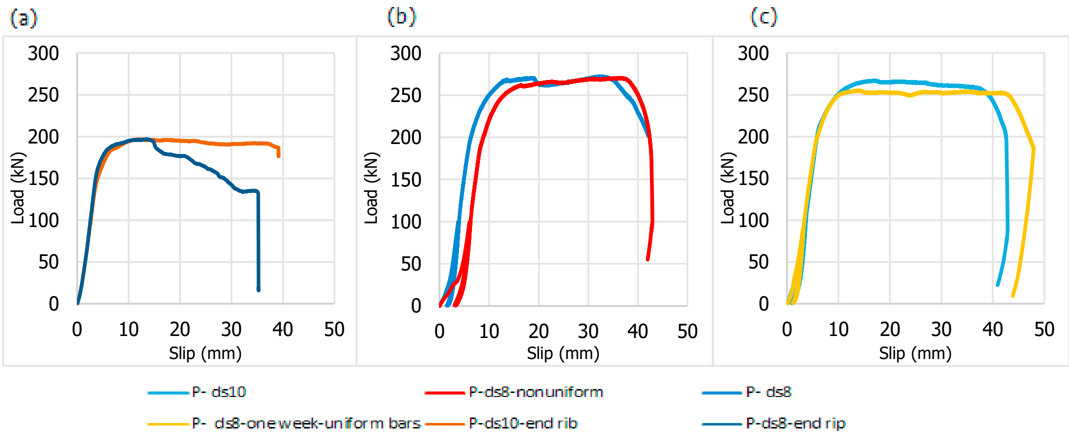

Fig. 4. Load-slip of the reinforced specimens.

After that, the P-ds10-end rib continues to sustain the maximum load for about 23 mm of extra slip at nearly the same rate, i.e. horizontal plastic plateau. Whereas, the P-ds8- end rib maximum resistance deteriorates gradually by more than 40 kN along 20mm of extra slip until the transverse rebar inside the hole ruptured at 35 mm of slippage. In this side of the specimen, the transverse Ø8 stirrup sheared off at the part which is side the rib. At this stage, the relative slip was about 35mm.

Specimen 4 shown in Fig. 2(d) was design to study the effect of the rib position and the effect of the rebars re-distribution. A minor variation can be noticed in the maximum shear resistance of slabs with similar steel ratio for both specimens, i.e. three and four. However, a significant effect for the perfobonf rib position is clearly indicated in Fig. 4(b). The difference in the final shear resistance between the third and the forth specimen is about 70 kN which represent about 40% increase in shear resistance of specimen four. The fifth specimen, shown in Fig. 2(e), was design to investigate two parameters: the influence of the transverse reinforcement positioning and the testing age on the overall connector's shear resistance.

From the tests results of P-ds8 and P-s8- 7days, it seems that the age of testing has minor effect if the samples are reinforced. Only about 15 kN the shear resistance of the 28day sample is higher than the 7day sample but with less slip about 5mm in comparison. Both samples have shown almost an identical load-slip behaviour during the elastic and the elasto-plastic behaviour, see Fig. 4(c).

The P-ds8 curve from the last test is in an excellent consistency with the P-ds10 curve from specimen four and the only difference between the two P-ds8 curves is the permanent slip at the start of the test otherwise the two P-ds8 curves are almost identical which means that the re-distribution of the stirrups across the slab did not affect the shear resistance of PSCs. Remarkably, both mid PSCs with Ø8 rebars were sheared off while the rib with Ø10 rebar was failed by tension-shear failure which might confirm the effect of the rebars on the rib deformation. In the third specimen, the ribs were plastically deformed without any rib shearing off. It is worth mentioning that, in all the tests, the repeated load has no influence on the PSCs structural behaviour, as seen in Fig. 4(b,c).

3.3 A comparison study

The four numerical expressions, shown in Table 1, have been used widely among the researchers to estimate the shear resistance PSCs.

Table 1. The numerical expressions to estimate the PSC shear resistance

$Q_{rib} = 4.50\, h_{rib} t_{rib} f_{ck} + 0.91\, A_{tr}\, f_y + 3.31 n\, D^2 \sqrt{f_{ck}}$	(1) (Oguejiofor et al., 1997)
$Q_{rib} = 0.747\, bh \sqrt{f_{ck}} + 0.413\, b_f\, L_c + 0.9\, A_{tr} f_y + 1.3\, n\, D^2 \sqrt{f_{ck}}$	(2) (Medberry et al., 2002)
$Q_{rib} = 4.04\, \frac{h_{rib}}{b}\, h_{rib} t_{rib} f_{ck} + 2.37\, nD^2 \sqrt{f_{ck}} + 0.16\, A_{cc} \sqrt{f_{ck}} + 31.85 \times 10^6 \left({A_{tr}}/{A_{cc}} \right)$	(3) (Veríssimo et al., 2006)
$Q_{rib} = 3.14\, h_{rib}\, t_{rib}\, f_{ck} + 1.21\, A_{tr} f_y + 2.98\, n\, D^2 \sqrt{f_{ck}}$	(4) (Ahn et al., 2010)

In this comparison study, shown in Fig. 5(a), previous tests conducted by Costa-Neves et al. (2013) and Zheng et al. (2016), see Fig. 5(b,c); were selected for the following reasons: Costa-Neves et al. (2013) tested un-reinforced and reinforced PSCs which is similar to this testing campaign ; Costa-Neves et al. (2013) used the POT setup and the specimen was the EC-4 sample with two-holes rib while this research employed one-hole rib embedded in BS-5 sample which is the same as Zheng et al. (2016) who tested a rib with one circular hole of 60mm but cast into(460x460x400) mm concrete slabs; Costa-Neves et al. (2013) and Zheng et al. (2016) also compared their tests results to the equations shown in Table 1, which is useful to compare the results of this paper with them also.

In these equations, Q_{rib} is the estimated shear resistance of the PSC; f_c' is the concrete compressive strength; D is the diameter of the hole; n is the number of the holes; h_{rib}, t_{rib} are the height of connector and its thickness; A_{tr} is the cross-sectional area of transverse rebars; f_y is the yield strength of reinforcement. A_{cc} is the longitudinal slab area minus the connector area; b and h are the thickness of the concrete slab and the distance from the end of the rib to the bottom of the slab respectively.

The average results of a set of three POTs conducted by Zheng et al. (2016) was 425 kN while the average of the predictions, is 568 kN which makes the percentage of the difference between the test result and the mean of the predictions is 25%. Costa-Neves et al. (2013) test result of a PSC with two holes of 30mm in diameter embedded in un-reinforced concrete slab was 280 kN. The average of the estimations is 258 kN and the percentage of difference between the test value and the average of estimation is 8.5%. The second test was for the same above sample but with transverse reinforcement embedded in the slab. The test result was 398 kN and the mean of the four predictions was 332 kN. The percentage of difference between the test value and the average of predictions s in this case is 17%, as shown in Fig. 5. In this study, the difference in results of OSPOT tests and the average of predications of the four equations are for P-ds10, P- ds8 and P is about 5% which is less than the average of the other researchers, see Fig. 5.

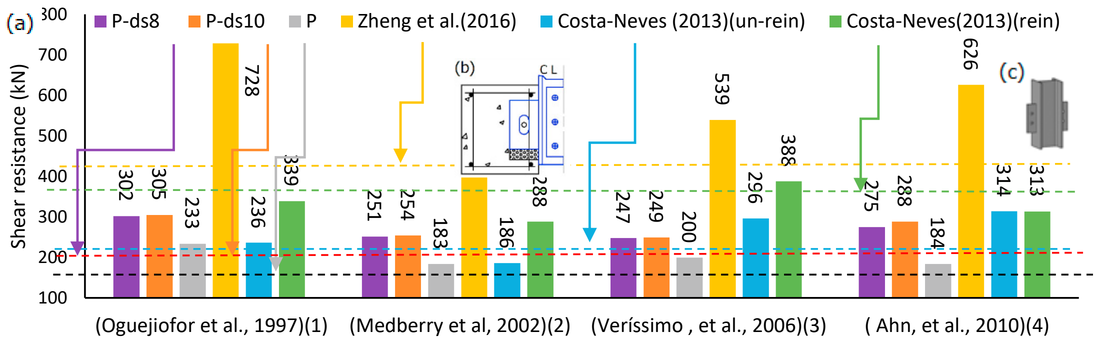

Fig. 5. The estimated shear resistance and the test results

4 Conclusion

The results of the OSPOT identical twins were consistent, the difference is less than 5%, and this difference in results might be related the concrete mechanical properties, the vertical casting of the three layers of concrete for each slab and its vibration. The other results which are obtained from the OSPOTs are consistent also and have shown the effect of several design parameters. Further, according to Lorenc et al. (2010)(p.4) ' If the strength is reached due to shear of the steel connector, then the information about its load-bearing capacity and the ductility is given', all the reinforced rib have shown this behaviour. Since all the reinforced PSCs have failed by the shearing of the connector. Also, the results of the comparison study have shown a better consistent between the OSPTs results and the estimations offered by several numerical expression which were originally derived from the regression analysis of the POT results. Hence, the OSPOT setup might consider a more economic option to investigate the structural behaviour of the PSCs as two results can be obtained from one POT specimen which reduces the cost and time significantly.

The structural analysis of the load-slip curve from the OSPOT shows the elastic part of load-slip curve, i.e. the concrete dowel shear resistance, is coinciding with the same part of the reinforced PSC and they are identical if the other design parameters are kept constant. This confirms that the concrete dowel sustain the applied load first until the fracture of the concrete after that the rebar inside the hole starts to resist the applied load. Further, by comparing the results of the one-week samples with their counterparts of 28day samples, the increase in the overall shear resistance in the 28day sample is due to the increase in the concrete dowel shear resistance.

Acknowledgements

The first author thanks the University of Kufa, for financial support of his PhD study at Loughborough University, and to Roform Product Limited for their cooperation in the fabrication of the OSPOT specimens.

References

Ahn, J.-H., Lee, C.-G., Won, J.-H. & Kim, S.-H., 2010. Shear resistance of the perfobond-rib shear connector depending on concrete strength and rib arrangement. *Journal of Constructional Steel Research,* Volume 66, pp. 1295-1307.

BS 5400-5, 2005. *Part 5: Code of practice for the design of composite bridge,* London: British Standards Institution.

Costa-Neves, L., Figueiredo, J., da S. Vellasco, P. & da Cruz Vianna, J., 2013. Perforated shear connectors on composite girders under monotonic loading: An experimental approach. *Engineering Structures,* Volume 56, pp. 721-737.

EN 1994-2 : Eurocode 4, 2005. *Design of composite steel and concrete structures – Part 2,* Brussels: Standarisation Uropeain Committee for Standardization..

Ernst , S., 2006. *Factors Affecting the Behaviour of the Shear Connection of Steel-Concrete Composite Beams (PhD Thesis).* s.l.:University of Western Sydney .

Kim, Y.-O., Kang, J.-Y., Koo, H.-B. & Kim, D.-J., 2016. Pull-Out Resistance Capacity of a New Perfobond Shear Connector for Steel Pile Cap Strengthening. *Advances in Materials Science and Engineering,* Hindawi Publishing Corporation(Volume2016), pp. 1-12.

Lorenc, W., Kubica, E. & Kozuch, M., 2010. Testing procedures in evaluation of resistance of innovative shear connection with composite dowels. *Archives of Civil and Mechanical Engineering,* 10(3).

Medberry, S. B. & Shahrooz, B. M., 2002. Perfobond Shear Connector for Composite Construction. *Engineering Journal,* 39(1), pp. 2-12.

Oguejiofor , E. & Hosain , M., 1997. Numerical analysis of push-out specimens with perfobond rib connectors. Comput. *Canadian J. Civil Eng.,* Volume 62, pp. 617-624.

Su, Q., Yang, G. & Bradford, M., 2016. Bearing Capacity of Perfobond Rib Shear Connectors in composite girder bridges. *Journal of Bridge Engineering,* 21(4).

Valente, M. I., 2007. *Experimental Studies on Shear Connection Systems in Steel and Lightweight Concrete Composite Bridges ,* University of Minho, Portugal : PhD Thesis.

Veríssimo , G. S. et al., 2006. *Design and experimental analysis of a new shear connector for steel and concrete composite structures.* s.l., 3rd International Conference on Bridge Maintenance, Safety and Management, pp. 1313-1322.

Xiao, L., Li, X. & John Ma, Z., 2016. Behavior of perforated shear connectors in steel–concrete composite joints of hybrid bridges. *Journal of Bridge Engineering,* 22(4), p. 04016135.

Zheng, S., Liu, Y., Yoda, T. & Lin, W., 2016. Parametric study on shear capacity of circular-hole and long-hole perfobond shear connector. *Journal of Constructional Steel Research,* Volume 117, pp. 64-80.