Engineering Structures 106 (2016) 15–24

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Experimental investigation of connection details for precast deck panels on concrete girders in composite deck construction

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ARTICLE INFO

Article history:
Received 4 September 2014
Revised 27 May 2015
Accepted 5 October 2015
Available online 24 October 2015

Keywords:
Connection
Precast
Deck
Studs
Rebar dowels
Pull-out
Push-off
Shear
Tension

ABSTRACT

Full-depth precast concrete bridge deck panels are connected to the girders on site by shear connectors to create a composite action and to prevent relative movement between the beam and the precast deck during all loading levels. This research study experimentally investigates different connection details that can provide a load transfer mechanism between concrete girders and precast deck panels to ensure full composite action. Three connection details were investigated: unconfined studs, confined studs and rebar dowels. Six specimens were cast and tested in direct pull-out or push-off tests. The confined studs showed the highest shear capacity at low displacement levels as well as the highest pull-out capacity. However, the three configurations showed higher capacities than expected according to the Canadian highway bridge design code.

1. Introduction

Full-depth precast concrete bridge deck panels are an alternative to cast-in-place concrete decks which can reduce the bridge closure times during deck replacements and bridge construction. Panels are prefabricated at a precast concrete plant under controlled casting and curing conditions before they are transported to the bridge site. At the site, the panels are set in place on the girders and adjusted to the correct elevation with leveling bolts. The panels are connected to the girders by shear connectors to create a composite action and to prevent relative movement between the beam and the precast deck during all loading levels. The shear connectors extend out of the girder and into shear pockets formed in the panels. The shear connectors are clustered at the shear pockets (Fig. 1) instead of being spaced uniformly along the length of the girder, as is typical with cast-in-place concrete deck construction. The objective of this research study is to experimentally study different connection details that can provide a load transfer mechanism between concrete girders and precast deck panels to ensure full composite action.

Research has been done to determine the horizontal shear resistance at the interface between the beam and the deck. Most of the previous research studies on the use of precast concrete deck panels focused on the behavior of shear connectors for steel girders [13,4,5,7]. It was reported that the number and configuration of shear studs (connectors) in addition to the spacing between the pockets affect the load carrying capacity of the connection. Some researchers have investigated the behavior of shear connectors in concrete structures under different types of loading [12,9,2,8]. However, they mainly investigated some specific precast structural systems and connections as beam to column connections in buildings. Limited amount of research has been conducted on connections of precast deck panels to concrete girders. The experiments conducted to investigate the performance of the shear connectors consists mainly of 3 types; pull-outs, push offs and full scale testing.

In the National Cooperative Highway Research Program [7], two connections were tested under direct tension load. In the tested connections, the shear connectors were 3 headed studs (31.8 mm diameter) spaced at 102 mm and embedded for 216 mm in an NU girder (web thickness 510 mm, flange width 1169 mm and gir-

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http://dx.doi.org/10.1016/j.engstruct.2015.10.002
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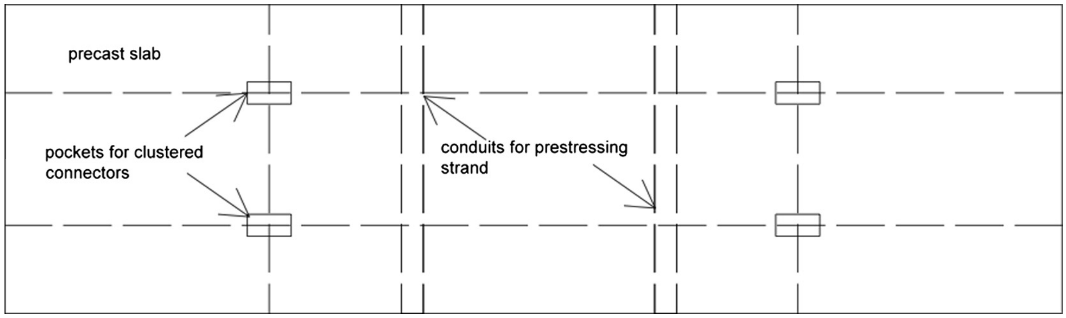


Fig. 1. Schematic showing the pockets for clustered connectors in a precast slab.

der length 610 mm). In the first set of experiments, the section was reinforced with the typical shear reinforcement. In the second set, additional reinforcement was provided at the head of each stud. They reported that, for the specimens without additional reinforcement, horizontal cracks developed at the junction between the top flange and the vertical web of the specimen at an applied load equal to the maximum tensile capacity of the web reinforcement (45% of the yield capacity of the stud group). As loading continued, the side cracks widened and close to failure cracks formed on the top surface of the specimen. At failure, the studs with concrete surrounding them pulled out of the specimen at a load that corresponded to 61% of the yield capacity of the stud group. For the second configuration, the specimen's behavior was superior to that of specimens without additional reinforcement. The first sign of cracking occurred at 75% of the yield capacity of the stud group. The failure mode did not change but the failure load was 107% of the yield capacity of the stud group.

Menkulasi and Roberts-Wollman [6] studied the horizontal shear resistance of the connection between full depth precast concrete bridge deck panels and prestressed concrete girders by performing 36 push off tests. The tested parameters were: the type of shear connector (no connector, extended stirrups, post installed reinforcing bars and insert anchors), the cross sectional area of the connector, the type of grout and the haunch height. They concluded that for clustered studs, the embedment depth must be adequate (more than 127 mm) to avoid the cone break out failure mode. The post installed reinforcing bars and insert anchors showed similar behavior to the extended stirrups indicating that they are viable shear connectors.

Trejo et al. [10] conducted similar push-off tests on three different shear connector configurations, namely cast-in-place re-bar dowels, threaded rods with a coupler and threaded rods without a coupler along with two different haunch heights. They reported that the resistance provided by the bond between the grout and the precast specimens sustained a relative displacement of approximately 0.25–1.5 mm. After that, the bond between the grout and the beam specimens broke down and the resistance dropped until the connectors were engaged at a relative displacement of approximately 2.5–4 mm. As the relative displacements increased (beyond 15 mm), the resistance increased slightly due to the strain-hardening of the connectors. Failure occurred when displacements exceeded 18 mm. They reported 4 modes of failure: grout crushing, beam anchorage/shear failure, re-bar pull-out from the deck panel, and/or shear failure of the connector.

Trejo and Kim [11] tested 8 different configurations for push-off specimens with 3 replicated specimens per configuration. The tested variables were: the connector type (re-bars or threaded rods with couplers), the type of confinement (no confinement, inside or outside the pocket, inside and outside the pocket) and the cross sectional area of the shear connector. Re-bars M13 were used as shear connectors. Two different diameters for the threaded rods (Grade 60) were used; 32 and 19 mm. Closed couplers were used to connect the shear connectors. They reported five distinct stages up to failure: initial adhesion loss, shear key action, shear key action failure, dowel action of the shear connectors at the sustained load, and final failure of the system. The hoop confinement did not significantly improve the shear performance of the shear pocket with 19 mm diameter shear connectors, but on the contrary confining the inside and the outside of the pocket with hoops in presence of the 32 mm diameter shear connector increased the peak shear resistance by almost 20%. They also reported that the shear transfer mechanisms of the small and large diameter shear connectors are likely to be different.

Further research is needed to evaluate different systems with reduced number of shear pockets leading to a more constructible and economical design. This paper reports the results of an experimental study on the performance of different shear connectors between precast concrete panels and concrete girders under static loading for new construction. The project examines the behavior of new connections in direct tension and push off specimens.

2. Experimental program

The experimental program included the fabrication and testing of three direct shear push-off specimens and three pullout specimens (Table 1). The main variable was the connection type: Unconfined studs, confined studs and rebar dowels. The unconfined studs were tested as the typical connection used conforming to the specifications of ASTM A1044/A1044M [1]. The confined studs and rebar dowels were suggested by the Ministry of Transportation of Ontario and the partnering precast concrete plants as the feasible alternatives for the unconfined studs. To simplify the connections for the purpose of the experimental study and to ensure that the specimens could be tested to failure with the available testing equipment, each tested specimen included only a single row of studs or dowels. In contrast, a typical shear cluster in a bridge girder would have several rows of studs to achieve the required shear force. The effect of this simplification is discussed in further detail later in the paper. The results of these tests are outlined in the following sections, followed by a brief design example according the Canadian Highway Bridge Design Code which is compared with the experimental results.

3. Test specimens and setup

3.1. Push-off tests

The test specimens consisted of a bridge deck element and a girder element, which were cast separately. Fig. 2 shows the

Table 1
Connection types.

	Connection	Advantage	Number of specimens	
A	Unconfined 19 mm diameter studs	Typical connection	1 shear push-off	1 pullout with stirrup spacing 300 mm
B	Confined 19 mm diameter studs	Recommended alternatives	1 shear push-off	1 pullout with stirrup spacing 300 mm
C	15M rebar dowels		1 shear push-off	1 pullout with stirrup spacing 300 mm

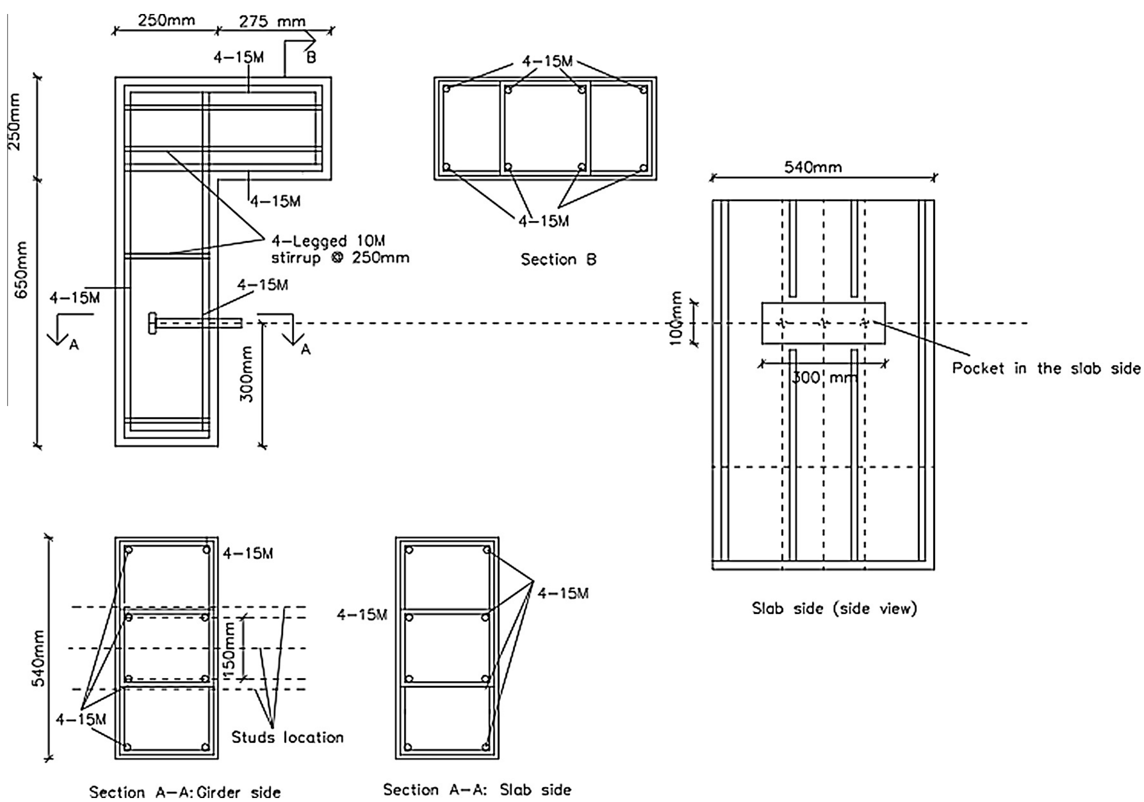


Fig. 2. Schematic of the push-off specimen.

schematic of the specimen. The reinforcement details are shown in Fig. 3. The specimen fabrication is shown in Fig. 4. Both elements were L-shaped 900 × 525 × 540 mm. The deck element had an empty pocket (300 × 100 mm) which was matched with a cluster of studs protruding from the girder element. The stud clusters consisted of either three 19 mm steel studs or 4 hooked 15M rebar dowels (the total steel cross-sectional area provided was approximately the same in each case). The shear pocket was fitted over the cluster of studs and a non-shrink grout was used to fill the pocket and the 25 mm (1-in.) haunch between the two elements.

After curing, the test specimens were placed vertically beneath a hydraulic actuator in a 500 kN capacity test frame. The vertical joint of the push-off specimens was aligned with the centreline of the actuator. LVDTs were mounted to the concrete surface in four locations (Fig. 5) to measure the relative shear displacement and horizontal crack width across the haunch. Load was applied in displacement control at a rate of 0.8 mm/min until failure occurred or the capacity of the test frame was reached. The average concrete compressive strength for the girder and deck elements at time of testing was 49.5 MPa, respectively, while the grout had an average compressive strength of 62.2 MPa.

3.2. Pull-out tests

In a typical slab-on-girder bridge, the top surface of the concrete girder is intentionally roughened to enhance load-sharing between the girder and deck with surface deformations on the order of one inch (25 mm) in depth. Shear friction theory, which is used to describe the mechanisms contributing to composite action, assumes that the relative shear displacement across a rough cracked surface will cause the crack to dilate such that the load transfer mechanism is not pure shear but a combination of shear and tension. As the crack widens, tensile stresses are developed in the reinforcement crossing the crack, providing a normal clamping force which increases the capacity of the shear plane to resist friction forces. Hence, an effective shear connection type should be able to resist significant tensile stresses prior to yielding or pulling out of the reinforcement from the concrete substrate.

The design and reinforcement details of the direct tension pull-out specimens were similar to the top half of a CPCI 1600 girder, forming a T-section with 1 row of 3 studs or 4 hooked rebar dowels with an embedment depth of 150 mm (Figs. 6 and 7). A CPCI 1600 is a standard I-girder with a depth of 1600 mm according to the Canadian Precast Prestressed Concrete Institute. Each of the studs had a diameter of 19 mm, while the hooked rebar dowels were formed from 15M rebar. A top mat of steel reinforcement was provided in the top flange of the T-section and stirrups extended into the web at a spacing of 300 mm (150 mm on either side of the shear connectors), which is a typical reinforcement layout for that section. In addition, once failure is initiated, the top flange might be experiencing tension stresses on the top surface,

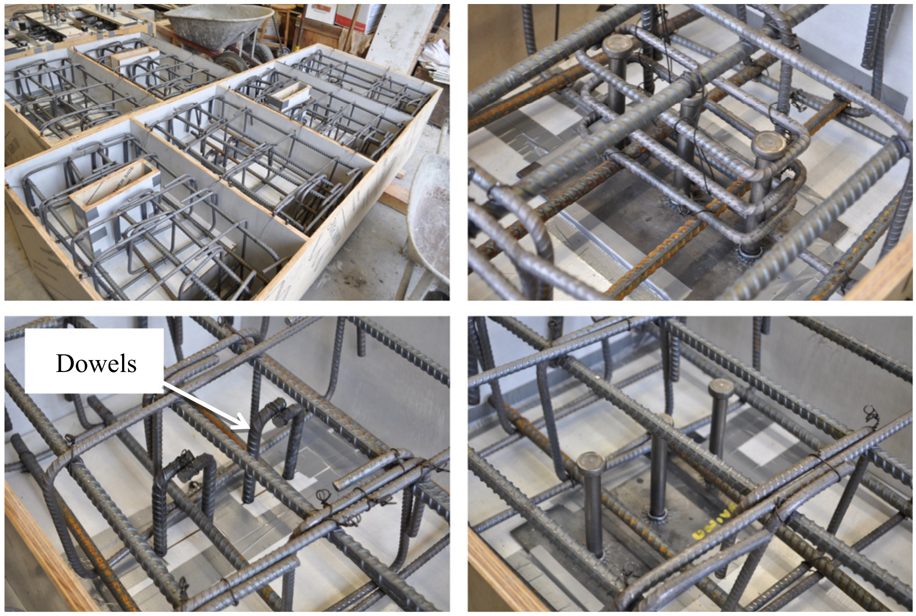
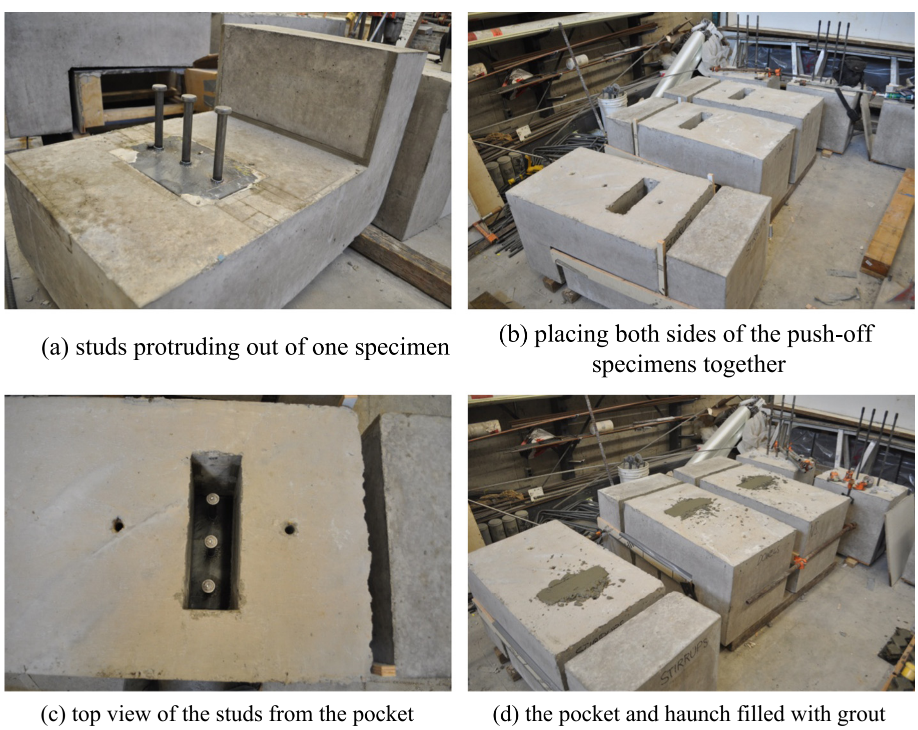


Fig. 3. Clockwise from top left: (a) reinforcing cages and formwork, (b) confined studs, (c) unconfined studs and (d) rebar dowels.



(a) studs protruding out of one specimen

(b) placing both sides of the push-off specimens together

(c) top view of the studs from the pocket

(d) the pocket and haunch filled with grout

Fig. 4. Grouting shear connectors into shear pockets.

which will be resisted by the top reinforcing mat. Each end of the test specimens consisted of solid concrete blocks which were bolted to the test frame using a set of threaded rods passing through ducts in the concrete to anchor the specimens to the testing frame during loading. Strain gauges were affixed onto each stud or dowel and a tensile load was applied to the bars simultaneously at a rate of 1 mm/min. The test setup is shown in Fig. 8. The test specimens had an average concrete compressive strength of 53.2 MPa.

4. Test results and discussion

4.1. Push-off tests

The test results for the push-off tests are shown in Figs. 9 and 10. The relative shear displacement between the deck and girder elements were measured using two LVDTs mounted on the concrete surface of the deck element. Similarly, the horizontal crack width was measured near the top and bottom of the grouted

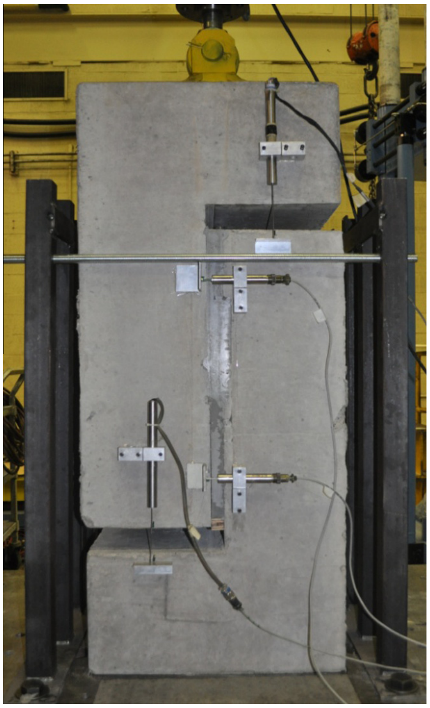


Fig. 5. Direct shear push-off test setup.

haunch. For each test, the first vertical crack appeared at the interface between the grout and the concrete at a load ranging from 140 kN to 180 kN. This resulted in a loss of adhesion and a small drop in stiffness as shown in Fig. 9. The load continued to increase until cohesion between each interface was lost at loads of approximately 230 kN, 240 kN and 300 kN, corresponding to the rebar dowels, unconfined studs and confined studs, respectively. At this stage, a horizontal crack appeared at the level of the shear connectors and the cracks widened further (Fig. 11). As all of the force was mainly carried by the steel bridging the crack, the stiffness reduced considerably indicating loss of composite action. Although the relative displacement continued to increase, little change in load was observed for both specimens reinforced with studs, which eventually ruptured following a sharp drop in load at a relative displacement of approximately 20 mm. The rebar dowels, on the other hand, continued to take additional load until the test was stopped at a load of 500 kN (capacity of the testing frame) and a relative displacement of approximately 45 mm. After cracking, some small rotation of the deck element was observed as the horizontal crack became wider at the bottom of the test specimen than at the top. The maximum crack width measured by the bottom horizontal LVDT is shown in Fig. 10. It is worth mentioning that the failure stages and the displacements reported here are consistent with the findings of Trejo and Kim [11] for connections between precast concrete deck panels and steel girders.

For each test, no damage was observed at the exterior surface of the grouted pocket. Confining the concrete around the shear pocket and the embedded studs prevented local crushing of the concrete on the deck side (see Fig. 12) and increased the concrete contribution to the overall load carrying capacity and stiffness. Although the failure mode occurred by rupture of the studs in both cases, the concrete around the studs displayed noticeably more crushing when no confinement was provided, indicating that the concrete had failed and was no longer contributing to load resistance resulting in a lower load capacity. This can also be shown in Fig. 9. For the confined studs, the load reached 300 kN and remained almost constant as the relative shear displacement increased to 10 mm. In this phase, the forces were mainly carried by the stud/steel bridging. However, the confinement effect was clear, where the load increased to 350 kN before rupture of the studs occurred. As for the specimen with rebar dowels, the small rebar diameter resulted in a relatively flexible dowel which was able to displace considerably without rupturing.

4.2. Pull-out tests

Test results for the pull-out specimens are given in Figs. 13 and 14. It should be noted when comparing the plotted load–displacement response and load–strain response that the free length and total cross-sectional area were not identical for each connection type; nevertheless, they are plotted together here for convenience. The first specimen to be tested contained embedded studs without confinement. The load was applied by gripping the studs with a nut threaded onto the end of each stud; the threads failed at a peak load of approximately 300 kN. A number of cracks were observed at the concrete surface indicating that the group of studs were beginning to pull out of the concrete, and the studs were yielded as shown by the load–strain response in Fig. 14. For the specimen with confined studs, three nuts were threaded onto each bar to prevent premature failure of the threads; this specimen displayed a similar behavior up to a peak load of approximately 305 kN before the group of studs began pulling out of the concrete. As shown in Fig. 15, failure occurred by block pull-out of the concrete surrounding the studs forming wide cracks at the concrete surface as well as bond splitting cracks propagating outwards from the outer vertical bars. The test was stopped when the load dropped to 50% of the peak load at a displacement of approximately 25 mm. The third specimen, containing four embedded rebar dowels displayed a similar type of behavior with a block pull-out occurring at a peak load of approximately 260 kN. As shown in Fig. 14, the rebar dowels pulled out prior to reaching their yield capacity. Based on these results the specimen with confined studs had the best performance among the three specimens tested.

5. Analytical predictions

The expected resistance of the shear connections according to the shear-friction theory is given in the Canadian Highway Bridge Design Code (CHBDC) by Eqs. (1) and (2) [3]:

$$v = 0.75(c + \mu\sigma) \tag{1}$$

$$\sigma = \frac{A_{studs}F_y}{A_{cv}} + \frac{N}{A_{cv}} \tag{2}$$

where v is the shear resistance of the pocket, c is the cohesion coefficient, μ is the friction coefficient, σ is the normal stress at the interface, A_{studs} is the cross-sectional area of the studs, F_y is the yield stress of the studs, A_{cv} is the area served by one pocket, and N is the normal force applied by the weight of the deck.

According to Clause 8.9.5.1 of the CHBDC, for concrete placed against hardened concrete with a clean surface and not intentionally roughened, c shall be equal to 0.25 MPa, and μ shall equal 0.60. Since the push-off tests were conducted vertically, N is taken as zero. A_{cv} is taken here as the area of the shear plane, or 540 mm by 600 mm giving a total area of 324,000 mm^2. The total area and yield stress of the three 19 mm diameter studs are 854.6 mm^2 and 350 MPa, respectively, resulting in a predicted shear resistance of 0.6 MPa. Similarly, the total area and yield stress of the rebar dowels are 800 mm^2 and 400 MPa, respectively, resulting in a predicted shear resistance of 0.63 MPa. In comparison, the experimental shear resistance provided by the confined studs, unconfined studs and rebar dowels prior to the cohesion loss

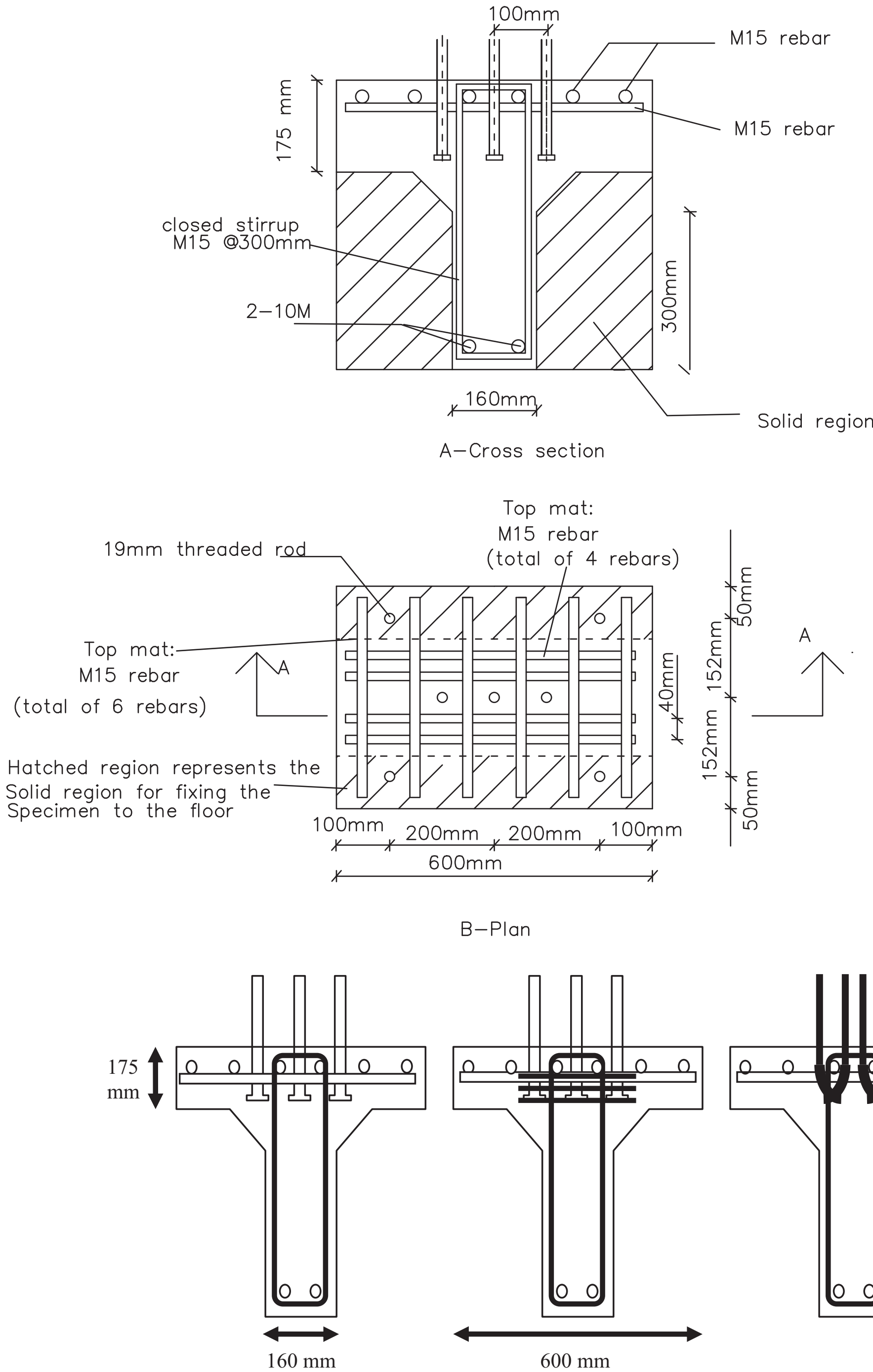


Fig. 6. Schematic of reinforcement configurations for pullout specimens.

and reduction in stiffness were 0.93 MPa, 0.74 MPa and 0.71 MPa, respectively, thus far exceeding predicted values in all cases. The ratio of experimental-to-predicted values were 1.54, 1.23 and 1.13 for the confined studs, unconfined studs and rebar dowels, respectively. Although the dowels displayed an increase in ultimate resistance compared to the studs, the increase in strength was also accompanied by a significant increase in crack width and displacement.

It is assumed that the direct shear strength of each connection type can be conservatively estimated to be proportional to the number of rows of studs or dowels in each shear cluster, as the added confinement provided by additional rows will have a beneficial effect on overall shear resistance, if any. Note that this is not the case for the pull-out strength of the connection which will be governed by a block pull-out failure mode.

Regarding the pull-out tests, the ultimate capacity of one stud is specified as 106 kN when the full shear cone is developed. In order to develop the full shear cone, the specified spacing of the studs is 182 mm; the capacity is expected to decrease as the spacing is reduced. According to the specifications provided by the manufacturer, the pull-out capacity of the studs in the given configuration accounting for group action would be 202 kN (compared to 318 kN when group action is ignored). The experimental failure load in tension for a group of three studs was 300 kN, which gives a ratio

Fig. 7. Reinforcing cages and formwork for pullout specimens.

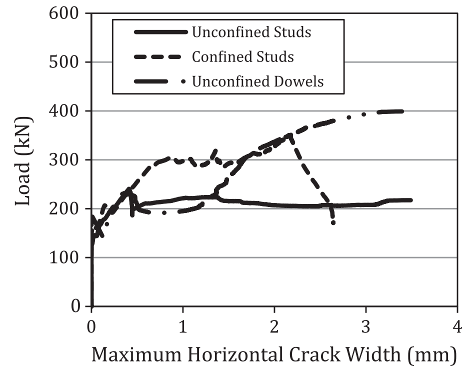


Fig. 10. Load-crack width response of push-off specimens.

Fig. 8. Direct tension pullout test setup.

Fig. 11. Push-off test specimen after the appearance of a horizontal crack at mid-depth.

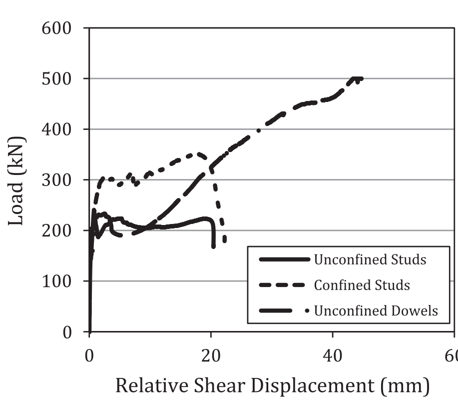


Fig. 9. Load–displacement response of push-off specimens.

of experimental-to-predicted resistance of 1.49. It is clear that the reduction due to group action underestimates the capacity of the studs and concrete in tension.

As additional rows of studs are added, the pull-out strength per stud is expected to reduce due to group action. According to the manufacturer's specifications, the expected capacity of 3 rows of 3 studs spaced at 100 mm in both directions is 357 kN, which is likely a conservative estimate. Pull-out of the group of studs is not expected to be a governing mode of failure in practice owing to the presence of the deck slab. However as explained earlier, an effective shear connection should be able to resist significant tensile stresses prior to pulling out of the reinforcement from the concrete substrate to provide clamping force to the dilating crack between the slab and the girder. Similar conclusions may be drawn with the hooked rebar dowels.

6. Design example

Although further testing is required to fully validate the performance of the proposed connections in a full-scale bridge, a design example for an interior girder in a continuous bridge consisting of two 35 m spans is provided as a reference. Given a deck depth and

(a) girder side (b) deck side with confinement (c) deck side without confinement

Fig. 12. Rupture of shear studs.

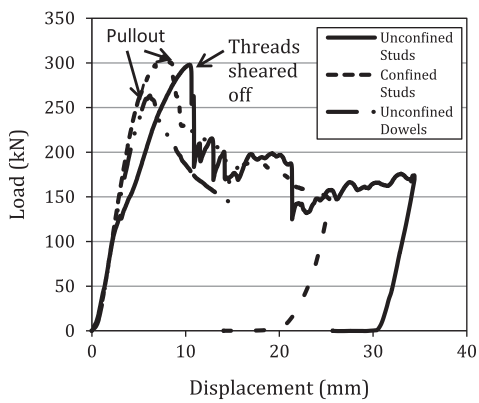


Fig. 13. Load–displacement response of pull-out specimens.

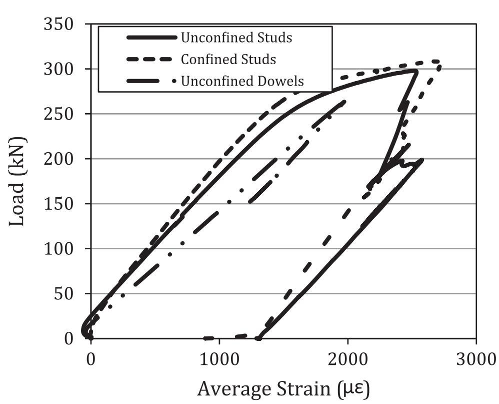


Fig. 14. Load–strain response of pull-out specimens.

width of 225 mm and 9 m, respectively, and three CPCI 1600 girders per span spaced at 3 m on center, the critical un-factored CHBDC design moments due to the CL-625 design truck with dynamic load allowance or a combination of 80% of the CL-625 design truck and the lane load (whichever governs) are given in Table 2. The critical case for the maximum positive live load moment occurs when the design truck is placed such that the center line of the span is half way between the resultant weight of the truck and the 175 kN axle. For the maximum negative live load moment, the critical case occurs when 80% of the design truck load is combined with the lane load where the resultant weight of the truck acts on the intermediate support.

These loads are distributed among the girders according to the provisions of the CHBDC. The moments in the interior girder are calculated using Eqs. (3)–(5):

$$M_g = \frac{SN}{F\left(1 + \frac{\mu C_f}{100}\right)} \frac{nR_L}{N} m \tag{3}$$

$$F = 7.2 - \frac{14}{L} \tag{4}$$

$$C_f = 10 - \frac{25}{L} \tag{5}$$

where M_g is the moment in the girder, S is the transverse spacing of the girders (3 m), N is the number of girders (3), n is the number of design lanes (2), R_L is a reduction factor for multilane loading (0.9), m is the moment per design lane (Table 2), and F and μC_f are correction factors.

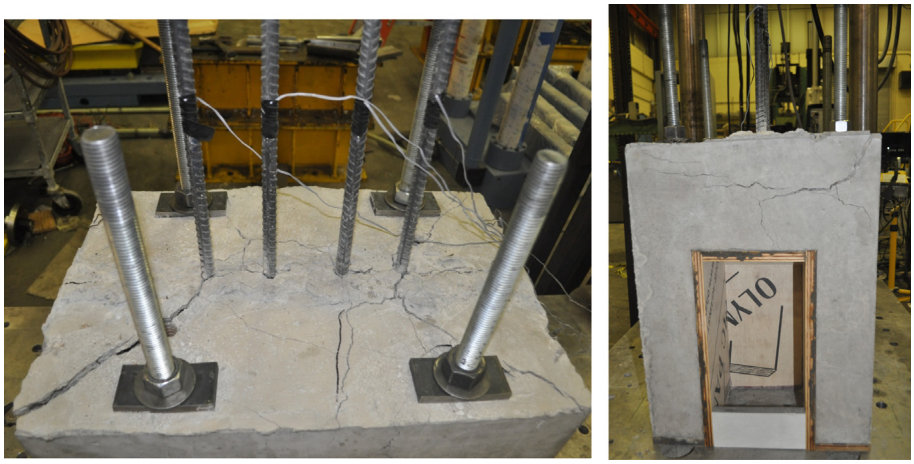
The design live and dead load moments in the interior girder are given in Table 3. Dead loads on the interior girder are based on a 3 m tributary width and include the weight of the slab (16.54 kN/m), wearing surface (3.6 kN/m) and girder self-weight (12.63 kN/m). Total factored moments are calculated using load factors of 1.25 (dead load) and 1.7 (live load).

A simplified approximation of the shear force at the interface between the girder and the deck can be computed by dividing the moment by the lever arm to obtain the resultant tension force in the reinforcement. Assuming that the centroid of the prestressing strands is located 110 mm from the bottom of the girder and that the compression resultant is located at the mid-depth of the slab, the lever arm has an approximate value of 1600 mm. The resulting maximum transverse shear between the girder and the deck is then 4901 kN and 5867 kN based on the factored positive and negative moments, respectively.

The positive moment is assumed to act over 80% of the span length, or 28 m, while the negative moment region is taken as 20% of the sum of both spans, or 14 m. Considering three pocket spacings, namely 600 mm, 900 mm and 1200 mm, the number of pockets in the positive moment region is taken as 47, 32 or 24, and for the negative moment region the number of pockets will be 24, 16 or 12. Assuming that the shear force is uniformly distributed among the pockets, the shear force per pocket ranges from 104 kN to 204 kN in the positive moment region, and 244 kN to 489 kN in the negative moment region, depending on the spacing of the pockets.

For a cluster of 9–19 mm diameter studs (three rows of 3 studs spaced at 100 mm in each direction), and accounting for the normal force due to the weight of the deck, the shear resistance of one pocket, as estimated from shear-friction theory is given in Tables 4 and 5 for a pocket size of 900 mm (girder flange width) by pocket spacing.

As shown in Tables 4 and 5, according to the simple design methodology described above the cluster of 9 studs can be used safely at a pocket spacing of 1200 mm over both the positive and negative moment regions.



(a) top view (b) side view

Fig. 15. Cracked concrete surface following pull-out failure.

Table 2
Critical unfactored CHBDC moments per design lane.

Loading case	Positive moment (kN m)	Negative moment (kN m)
Live load	3742	2442

Table 3
Maximum CHBDC moments in interior girder.

Load case	Positive moment (kN m)	Negative moment (kN m)
Unfactored live load	2552	1832
Unfactored dead load	2802	5017
Total factored load	7841	9386

Table 4
Resistance per shear connection in positive moment region.

Pocket spacing (mm)	Applied shear (MPa)	Shear resistance (MPa)	Safe?
600	0.19	0.94	Yes
900	0.19	0.69	Yes
1200	0.19	0.57	Yes

Table 5
Resistance per shear connection in negative moment region.

Pocket spacing (mm)	Applied shear (MPa)	Shear resistance (MPa)	Safe?
600	0.45	0.94	Yes
900	0.45	0.69	Yes
1200	0.45	0.57	Yes

7. Discussion

The experimental results presented represent a preliminary study demonstrating the potential of the proposed shear connection systems for slab-on-girder bridges with precast elements. Although the static performance is promising, additional testing on the dynamic and fatigue behavior of the connections as well as full-scale bridge decks is recommended to fully understand and validate the effectiveness of this setup. Additional tests are currently being planned at the University of Waterloo to address these parameters.

8. Conclusions

The following conclusions can be drawn:

1. The shear connection type consisting of confined studs with an embedment length of 150 mm provided the highest shear capacity at low displacement levels as well as the highest pull-out capacity. This connection type also displayed good ductility sustaining shear displacements of over 20 mm prior to rupture.
2. Confining the concrete around the shear pocket and the embedded studs prevented local crushing of the concrete and increased the concrete contribution to the overall load carrying capacity and stiffness.
3. In the push off tests, the unconfined studs failed at a lower load than the confined studs but they both failed at almost the same load in the pull-out tests. However, the unconfined studs also exceeded the predicted capacity using the shear-friction theory.
4. The rebar dowels demonstrated a higher ultimate capacity in the push-off tests but the increase in resistance was accompanied by large displacements which are not considered practical.
5. It is recommended to subsequently test a larger scale composite section to investigate the global response of the proposed shear connection configuration, as well as the dynamic and fatigue response.

Acknowledgements

Financial support from the Ministry of Transportation of Ontario is gratefully acknowledged. The authors would also like to thank Nelson Stud Welding and Hanson Pipe and Precast for their contribution of materials and facilities.

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