Performance of Headed Shear Stud Clusters

for

Precast Concrete Bridge Deck Panels

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ABSTRACT

The current edition of the Canadian Highway Bridge Design Code (CHBDC) provides limited guidance for the design of composite bridges with precast concrete deck panels and steel girders. The shear connection is critical in determining the overall performance of the composite superstructure and is commonly achieved with a grout blockout in a precast concrete deck panel, which accepts a cluster of tightly spaced steel shear studs. No specific equations are found in the CHBDC for the design of shear studs used in this system. Furthermore, the current longitudinal spacing limitation of 600mm for shear studs in the CHBDC is not conducive to the construction of precast concrete decks where an increased spacing of the stud cluster is desirable.

An experimental testing regime was conducted using push-test specimens constructed with precast concrete panels connected to steel flanges with steel studs within a circular grout pocket. Specimens were tested to failure to determine the ultimate capacity of a stud cluster and to investigate the reduction in ultimate strength after cyclic loading.

A 36m simple span composite superstructure was designed to the CHBDC. A parametric study is being conducted on the spacing of stud clusters to investigate the serviceability limit state of the composite superstructure using data captured from the push test specimens.

Results from the study indicate that clustered shear studs embedded in high strength grout for construction using precast deck panels are suitable for CHBDC requirements and preliminary investigations show that exceeding the code maximum stud spacing of 600mm can be acceptable.

INTRODUCTION

The Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00, has provisions to allow for the design of full depth precast concrete deck panels made composite with steel girders, but questions surrounding the shear stud connection linger. Current code equations for determining the ultimate limit strength of the shear stud were empirically derived from push tests conducted by Ollgaard (1971) on cast in place concrete specimens. The fundamental differences of shear studs used for the precast concrete deck system are that the shear studs are tightly packed into clusters at discrete locations along the girder and the studs are embedded in grout that transfers load to the concrete panel. The limiting longitudinal spacing of 600mm was originally suggested by Slutter and Fisher (1967) to keep the concrete and steel in contact, but it is desired to relax this requirement for the precast concrete panel system to improve constructability. The CHBCD requires two independent calculations for the design of shear studs; a check of the stud strength for ultimate limit strength and a check of the endurance limit for fatigue limit states. Oehlers (1995) showed that there is a reduction of the monotonic strength of the stud exposed to repeated loads. This potential for stud failure due to overload prior to reaching the endurance limit is a concern especially since the stud clusters cannot be visually inspected for fatigue

Many examples of bridges utilizing full depth precast concrete bridge deck panels exist in service today. The system is particularly popular for bridge deck rehabilitation projects, where a rapid construction technique is required to minimize traffic disruptions during construction. Another advantage of this deck system is found on the many remote construction sites, where quality cast in place concrete can be difficult to produce. **Figure 1** illustrates the stud clusters used on the Powerline Creek Bridge, Highway 37, near Terrace BC, constructed in 2004.
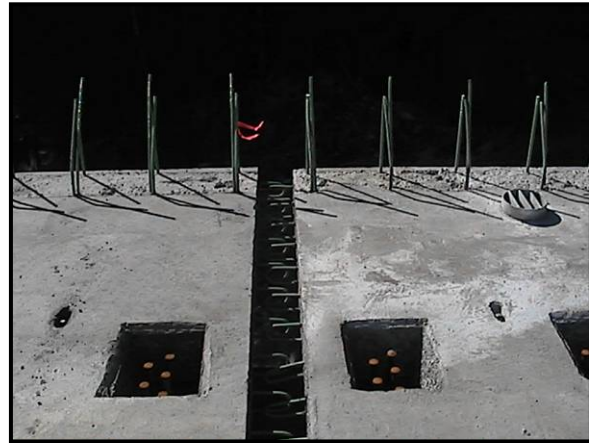
Figure 1 Powerline Creek Bridge

PUSH TEST EXPERIMENTS

The experimental push test program included 15 push test specimens, with the number of 16mm diameter studs per cluster being the main variable. Nine of the specimens were loaded monotonically to determine the ultimate strength of 5 different stud cluster groups, shown in **Figure 2**, with the remaining 6 used to determine the residual ultimate strength after being exposed to cyclic loading. Observations of the failure mode of the stud groups are provided and comparisons made to the code equations provided in CAN/CSA-S6-00 and to the results of research completed by others.
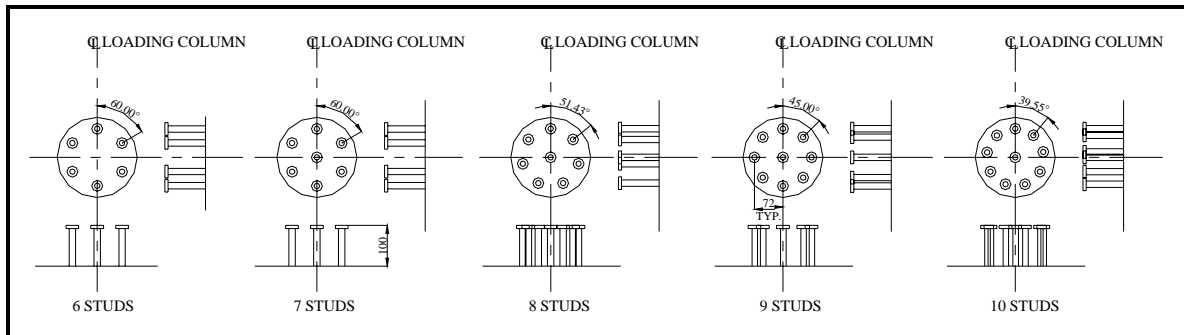
Figure 2 Stud Cluster Geometry

Push test specimens were constructed to determine the shear stud cluster behavior and are detailed in **Figure 3.** The 16mm steel studs used in the testing represent a scale factor of 0.73 for 22mm diameter studs commonly used in bridge construction. Consequently, the push test specimen components were scaled to 73% to reproduce the geometry of a typical full-scale stud cluster. The stud clusters were welded to 350AT steel plates which were laid horizontally and the precast concrete slabs placed on the steel plates. The

precast slabs were all cast at the same time using the same concrete mix. The stud cluster configuration chosen is similar to those used in bridge construction. This allowed the shear stud clusters to be grouted to the precast concrete panels in the horizontal position simulating construction techniques and using the same grout mix for similar specimens. To isolate the performance of the shear studs, petroleum jelly was spread over the top surface of the plate to remove friction and inhibit the chemical bond between the grout and the steel. The plates were bolted to a loading column, consisting of a custom fabricated welded steel wide flange beam that was bolted to the hydraulic actuator. This eliminated the need for expensive and time consuming welding and required only ten plates since they were reused, after the monotonic testing was completed, by welding new stud groups onto the opposite face of the plates. The grout pocket diameter was chosen to suit the scaled down specimens resulting in the selection of a 200mm diameter corrugated steel pipe with a wall thickness of 1.6mm. The base of the precast concrete specimens was uneven and required a smooth bearing surface to ensure even distribution of pressure from the floor reaction and to provide a specimen that was plumb. This was accomplished by placing the concrete panel in a mortar bed of Hydro-Stone®, a gypsum cement produced by the United States Gypsum Company.
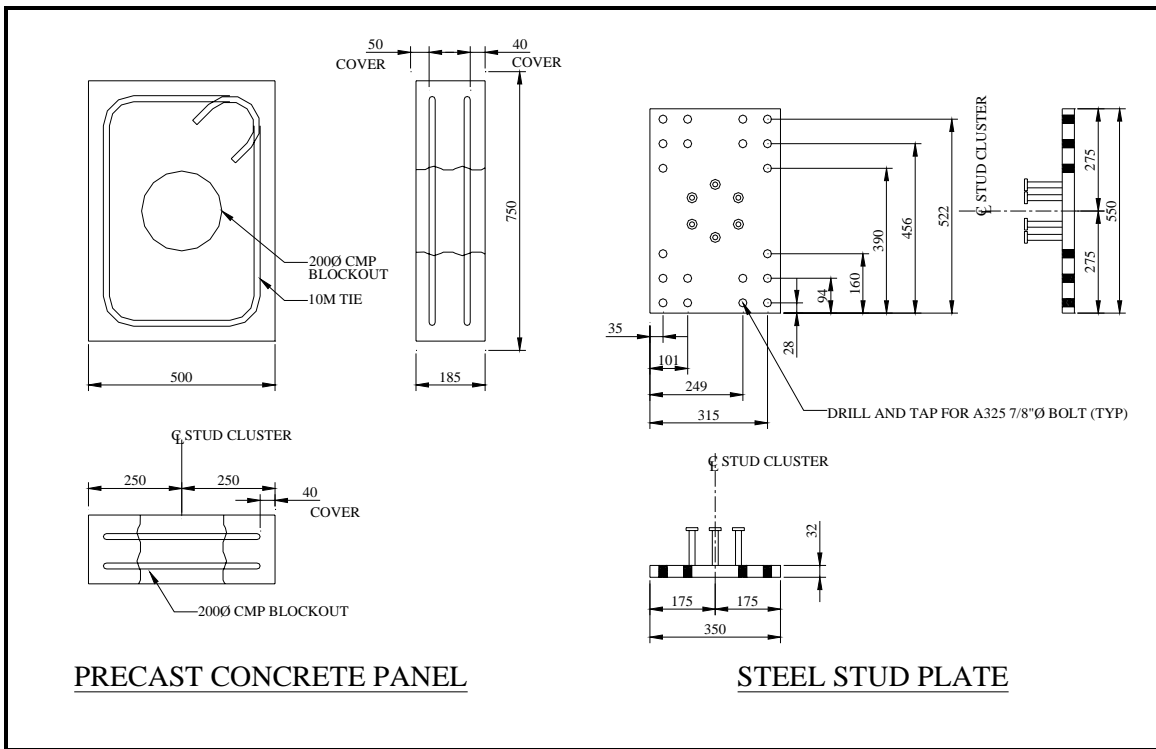
Figure 3 Concrete Panel and Steel Plate for Push Test Specimens

Four studs were machined and tested to determine the average tensile strength of 538 MPa. The average compressive stress of nine 300mm x 150mm concrete cylinders cast for the concrete slab was found to be 41.8 MPa. Target Traffic Patch Coarse Mix, a shrink compensated, steel fiber reinforced, high early strength premixed grout was used to attach the concrete panels to the steel plates. The selected grout is commonly used for precast deck panel construction in British Columbia. Both the grout and the concrete

were allowed to cure for over 28 days prior to testing the cylinders and push test specimens to achieve a consistent strength for all tests.

The push tests had two physical setups: unsymmetrical (one precast panel) for monotonic loading and symmetrical (two precast panels) for cyclic loading. The unsymmetrical configuration illustrated in **Figures 4** and **5** was adopted for the monotonic push tests to determine the ultimate and residual strength of the stud cluster. Contrary to the test procedures of other researchers, the testing of one panel at a time was chosen to maintain system stability at failure and ensure complete failure of each panel. This also allowed data to be collected for the full range of the loading for each concrete panel tested without having to make assumptions regarding the distribution of load and also reduced demands on the hydraulic system. The symmetrical setup was chosen for the cyclic load system to reduce the testing time requirements. The two panels were loaded for 250,000 cycles, then one panel removed and a new panel bolted to the loading column for another 500,000 cycles, resulting in 250,000, 500,000 and 750,000 cycles. For the cyclic loading, it is assumed that the load was shared equally between each panel.

The vertical location of the load cell attached to the restraining beam was chosen to simulate the boundary conditions of the symmetric configuration, producing similar tensile components in the stud group, thereby allowing a direct comparison of the test results with symmetric push tests conducted by other researchers.


Figure 4 - Unsymmetrical Test Setup Front   **Figure 5 - Unsymmetrical Test Setup Back**

Loading Routine

The monotonic loading was conducted under displacement control until failure of the stud cluster was observed. The cyclic loading, shown in **Figure 6** was a unidirectional sinusoidal wave, conducted under load control conditions at a frequency of 1 Hz for a predetermined number of cycles, namely 250,000, 500,000, and 750,000. The load range related to a Class C highway in CAN/CSA-S6-00. A class C highway has an Average Daily Truck Traffic (ADTT) of 250, providing 5.8 million fatigue cycles in 75 years for two design lanes. To determine the residual strength of the shear stud group, the monotonic load procedure previously described was used, testing one panel at a time.
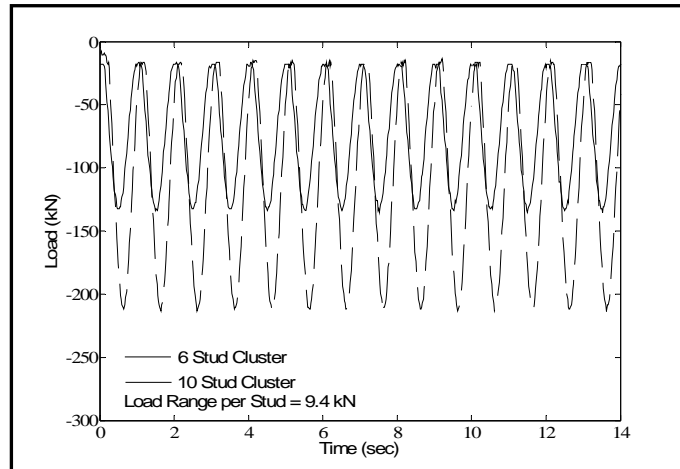
Figure 6 Unidirectional Cyclic Loading

Instrumentation

For the monotonic loading instrumentation, a set of nine linear variable displacement transducers (LVDTs) and 2 load cells were implemented. Two LVDTs were used to record the relative slip between the steel plate and the concrete panel positioned just below the center of the stud cluster. Two LVDTs were used to record the out of plane displacement of the concrete panels and two measured displacement of the restraining beams. A pair of LVDTs recorded any displacement of the load cell mounted on the restraining beam and the last LVDT was used to measure the stroke of the hydraulic actuator. Instrumentation for the symmetrical setup was similar.

Test Summary

A total of nine push test specimens were loaded monotonically to determine the ultimate strength of the stud cluster, six were used to determine the residual strength of the stud cluster after being exposed to cyclic loading. **Table 1** provides the labeling convention for the push test specimens and the loading condition.

Table 1 Push Test Specimen Summary

Panel No.	Studs per Cluster	Test Type*	Monotonic Setup**	Cyclic Setup**	No. Cycles
6A	6	M	A	-	-
6B	6	M	A	-	-
6C	6	C/M	A	S	250,000
6D	6	C/M	A	S	500,000
6E	6	C/M	A	S	750,000
7A	7	M	A	-	-
8A	8	M	A	-	-
8B	8	M	A	-	-
9A	9	M	A	-	-
9B	9	M	A	-	-
10A	10	M	A	-	-
10B	10	M	A	-	-
10C	10	C/M	A	S	250,000
10D	10	C/M	A	S	500,000
10E	10	C/M	A	S	750,000

*   M = Monotonic Loading, C = Cyclic Loading
** S = Symmetrical, A = Asymmetrical

STUD CLUSTER PERFORMANCE

Ultimate Strength

A total of nine push test specimens were loaded monotonically to determine the ultimate strength of the stud cluster. **Table 2** provides a summary of the test results.

Table 2 Monotonic Push Test Summary

Panel No.	Max Load (kN)	Max Slip (mm)	Average Grout Compressive Strength (MPa)
6A	693.2	8.7	45.6
6B	746.0	7.9	45.6
7A	836.2	12.3	59.9
8A	890.6	12.9	63.1
8B	987.4	9.1	63.1
9A	1036.1	14.0	53.7
9B	1043.9	12.0	53.7
10A	1186.7	13.1	60.9
10B	1173.5	16.7	60.9

Figure 7 provides a combined plot of the load slip characteristics for the monotonic load tests. The initial stiffness of the studs is similar and is fairly consistent, showing that additional studs provide no appreciable increase in stiffness. There is also a trend of increased plastic deformation with an increase in the number of studs. All the test specimens exhibited ductility, which is a required characteristic to allow load sharing among stud groups throughout the length of the superstructure. The premature failure of one stud is believed to have resulted in the sharp drop in capacity for Specimen 6A at a slip of 0.5 mm.
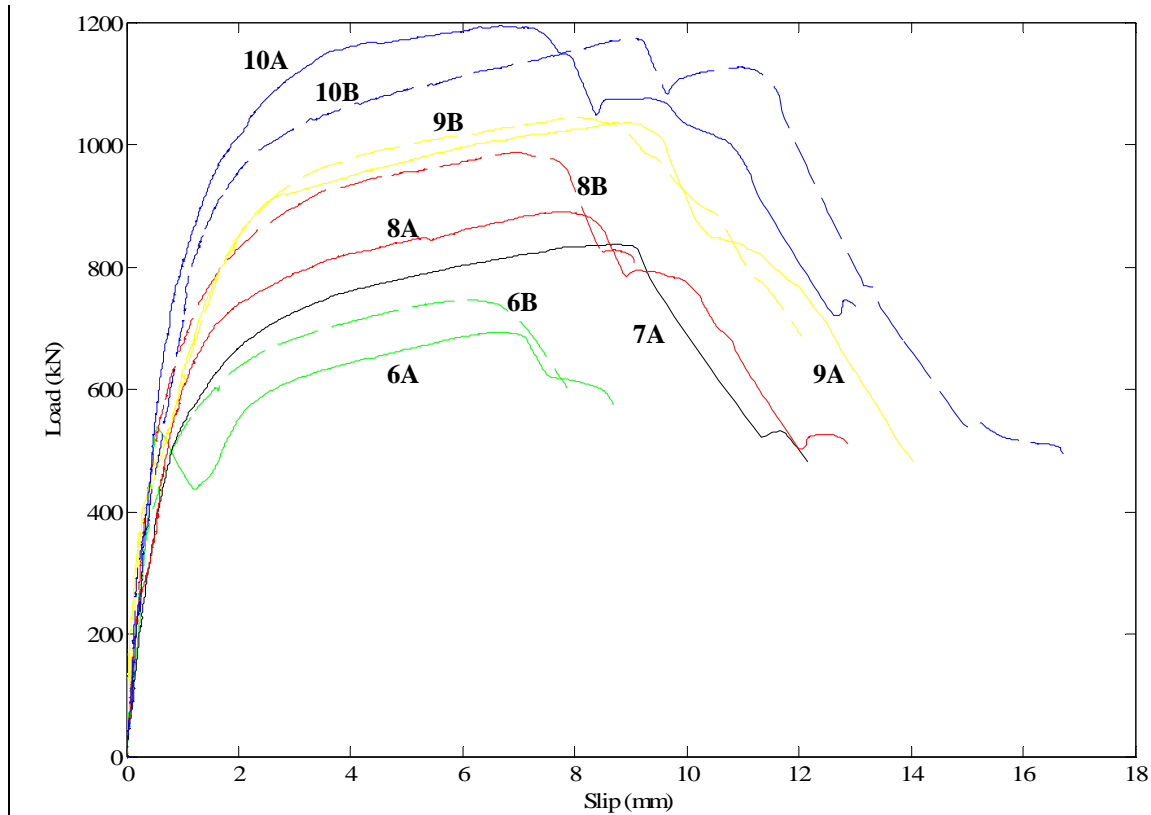
Figure 7 Combined Ultimate Strength of Monotonic Push Tests

Shim et al (2002) proposed the following equation for the ultimate strength of a shear connector in a precast concrete panel. Equation 1 was deemed to be suitable for predicting the ultimate strength of a shear connector, accounting for a grout bedding layer between the concrete panel and steel plate, provided that the grout is greater than 55 MPa.

$$Q_u = \alpha(0.362 A_{sh} + 18.714)$$
$$\alpha = 1 - 0.0086(B_h - 20)$$

(1)

where

Q_u = ultimate strength of a single shear stud, kN

α = reduction factor for the bedding thickness (1.0 for no bedding)

A_{sh} = cross sectional area of the shear stud shank mm^2

B_h = bedding layer thickness mm

7

Figure 8 compares the ultimate strength observed in the monotonic push tests and the strength predicted by Eq. 1 and the CHBDC requirements. Based on the test data, the ultimate strength of the stud cluster scales linearly with the number of studs and the CHBDC equation provides a lower bound to the monotonic strength. Linear regression line returns a value of 118.5 kN per stud with a correlation coefficient value of 0.97 for the monotonic push tests. The equation of Shim et al. predicts a value of 91.5kN, with the studs having a tensile strength of 502 kPa, approximately 93 percent of the tensile strength of the studs used in this experiment. The plot shows that the ultimate shear strength of stud cluster exceeds the tensile load of the studs, which is consistent with the results of other researchers.
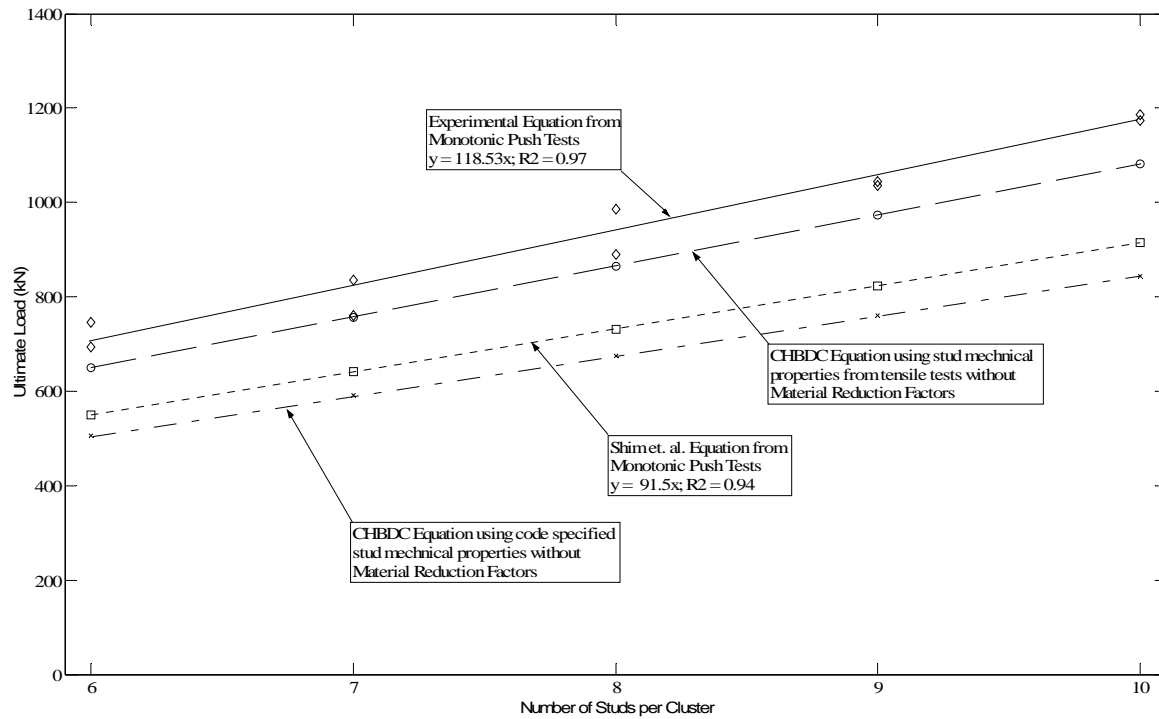
Figure 8 Monotonic Strength Comparison

Stud Cluster Failure Mechanism

The studs failed near the base of the stud for all monotonic loading tests. There were three distinct zones of failure observed among the different specimens: at the plate; at the top of the weld collar; or along a secondary shear plane approximately 6mm to 8mm above the base of the plate (or approximately 0.6 times the diameter of the stud). Contrary to the observations of Ohler (1971), where the stud was deformed in curvature over the full height, the studs deformed mainly in shear for only the first 6-8 mm, with the upper portion of the stud remaining undeformed. In some instances, some portions of the weld collar were also sheared from the plate.

8

Crushing and powdering of the grout was observed for a depth of approximately 8mm to 10mm at the leading edge of the studs, as they bore against the grout (**Figure 9**). The trailing edge of the studs pulled away from the grout, leaving a gap consistent with the slip reading from the LVDTs. There were no indications of overlapping zones of influence between two studs in the same cluster. No other damage was observed in the grout pocket. The steel corrugated metal pipe grout pocket provides a confining force for the grout pocket, minimizing damage to the grout medium.

Figure 9 Grout pocket damage

Cracking of the precast concrete panel was observed; this is collateral damage as a result of the grout pocket bearing against the concrete. The cracking pattern was found to be consistent between specimens. Cracking was for the full depth of the panel with, in some instances, a vertical crack occurring above the grout pocket at about the 1000 kN load range, attributed to the effect of the grout pocket expanding horizontally. Due to the location of instrumentation, cracking of the concrete panels did not influence the slip data reported here.

Figure 10 Stud Deformation

After all the data was collected from the specimens, the studs were removed from the grout pockets with a concrete rotary drill and jackhammer. **Figure 10** shows the studs during removal from the grout pocket.
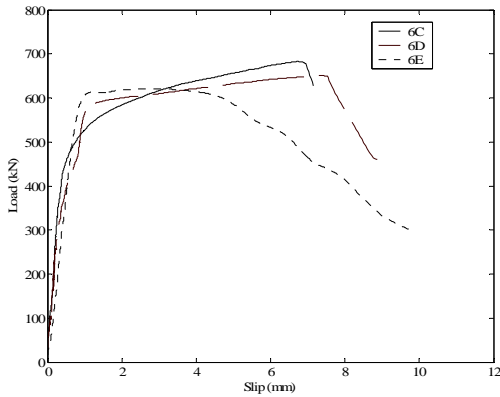
Residual Strength

The CHBDC code assumes that the ultimate strength of a shear stud is independent of the loading history, and only fails upon reaching its endurance limit. Oehlers et al (2000) showed that there is an immediate reduction of the static strength of a shear stud exposed to cyclic loading as result of crack propagation and that there is a linear relation between the residual stud strength and the number of cycles. The study suggests that it is possible that a shear stud could fail prior to reaching its fatigue endurance limit if exposed to overload. Since the shear studs are not accessible to inspection during the service life of the bridge, the fatigue design is critical in the safe performance of the superstructure.

Table 3 and **Figures 11** and **12** provide a summary of the residual strength testing. The residual strength of both the 6 and 10 stud cluster groups shows a modest reduction in the

9

ultimate strength as a result of being exposed to cyclic loading. The damage accumulated from the cyclic loading also tends to reduce the deformation capacity, particularly for 750,000 cycles.

Table 3 Residual Strength Summary

| Panel No.	No. Cycles	Max Load (kN)	Max Slip (mm)	Grout Compressive Strength (MPa)
6C	250,000	682.6	7.2	59.9
6D	500,000	650.7	8.9	59.9
6E	750,000	621.2	9.8	59.9
10C	250,000	1113.2	18.8	66.4
10D	500,000	1125.6	14.4	66.4
10E	750,000	1087.0	12.2	66.4

Figure 11
Residual Strength 6 Stud Cluster
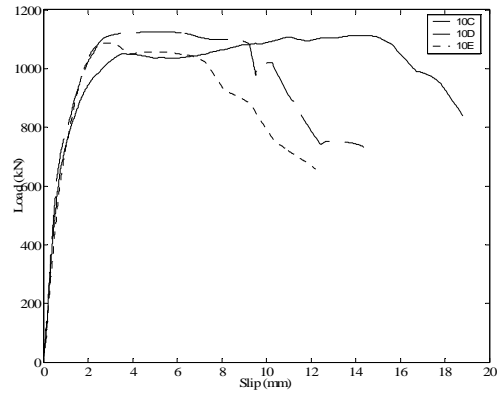
Figure 12
Residual Strength 10 Stud Cluster

Figure 13 provides a plot of the residual strength of the 6 and 10 stud clusters versus the number of cycles. As witnessed by Oehlers, after cyclic loading the shear stud connections experienced a reduction of the ultimate (residual) strength of the shear stud connections. The 6 stud cluster group shows exceptional fit with linear regression, with the 10 stud cluster also presenting a linear trend. The data points corresponding to the "zero" cycles is the average of the two monotonic test results.
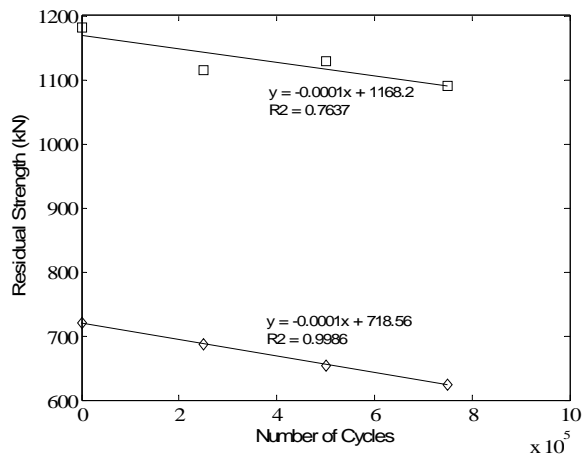

Figure 13: Residual Strength vs. No. Cycles

BRIDGE MODEL

Bridges with pre-cast deck panels were designed to evaluate the spacing limits for the shear stud connections. The superstructure was designed to maximize the flexibility, since this would have the largest impact on the serviceability limit states. The superstructure design was completed using the "simplified methods of analysis" in CAN/CSA-S6-00 and finite element plate models generated from the design to check the serviceability limit states using the commercially available analysis package STAAD.Pro. Details of typical superstructure design not contributing to the stiffness of the deck system, such as girder splice design, were specifically not included.

The superstructure has a 10.98m wide deck providing a 10.2m wide driving surface (**Figure 14**). The deck was designed as a variable depth precast concrete panel system to provide deck crossfall, incorporating standard 810mm high cast in place concrete parapets complete with a steel railing. An allowance was included in the design for a 90mm thick asphalt wearing surface. A three girder superstructure system was chosen providing 3.9m center to center of girders and a 1.59m deck cantilever.
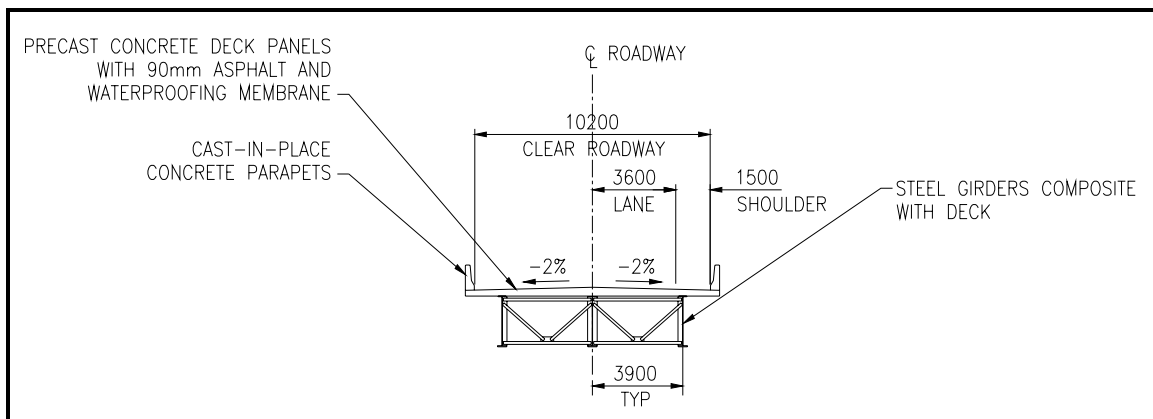
Figure 14 Superstructure Cross Section

The following longitudinal stud cluster spacings were considered: 300mm, 600mm, 1200mm and 2400mm. Three stud cluster stiffnesses were considered: no shear connection, the stiffness value from the push tests, and an infinitely rigid connection. The number of studs in each stud cluster was determined during the design process. The deflections of the superstructures were compared for an exterior girder under self-weight only, with the 300mm spacing with an infinitely rigid connection taken as the base value. The results are summarized in **Table 4**.

Table 4 Relative Superstructure Deflections

	300 mm	600 mm	1200 mm	2400 mm
No Shear Connection	2.49	2.49	2.49	2.49
Experimental Shear Stiffness	1.02	1.02	1.04	1.08
Rigid Shear Connection	1.00	1.00	1.01	1.02

It is observed that even with a longitudinal spacing of 2400mm, there is only an 8% increase of the static deflection. Since the majority of precast concrete deck panels are in the range of 3 meters, the 1200 mm spacing is a pragmatic choice to limit stud cluster spacing, with only a 4% increase in the deflections from the rigid case with 300 mm spacing.

CONCLUSIONS

Based on the push test experiment results and preliminary analysis of the finite element models, the findings from this study suggest that the CHBDC prescribed maximum 600mm spacing of shear studs can be exceeded for shear stud groups if using a grout pocket as described in the experiments. Further parametric studies of the superstructure will consider the change in natural frequency and girder stresses. The push tests also confirm that the CHBCD equations provide a lower bound to ultimate strength of a stud cluster. A reduction of the monotonic shear strength was observed with increased number of cycles but further study is required to predict the residual strength of the stud clusters. It is recommended that other potential grout pocket / stud cluster configurations be push tested to confirm their performance prior to use in construction.

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