TESTING OF A COMPLETE BRIDGE AT SCALE WITH FRP STAY-IN-PLACE STRUCTURAL FORMS FOR CONCRETE DECK FAILING IN PUNCHING SHEAR

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ABSTRACT

Fibre Reinforced Polymer (FRP) Stay-in-Place (SIP) formwork has received increasing attention recently as a system for constructing concrete bridge decks, particularly in slab on girder bridges. The system offers the potential for faster construction and improved durability for bridge decks. While the system has been investigated experimentally and seen some field application, there is an absence of test data regarding the ultimate limit state performance of FRP SIP formwork decks in the context of an actual bridge. Testing is most often conducted on bridge deck sections with free edges which provides a conservative but inaccurate estimate of the actual bridge deck’s capacity. This phenomenon was investigated by comparing the test results of four bridge deck tests including once bridge deck section and three deck panels within a full bridge. The testing was conducted at 1:2.75 scale. Testing of a bridge deck section revealed a 20% drop in capacity when compared to a panel in a full bridge of identical design. The deck section also exhibited a 23% lower post cracking stiffness (in terms of centreline deflection). Comparison of two bridge panels with different spans revealed span to effect on stiffness but a negligible impact on punching shear capacity. Several failure criterion were evaluated against the test results to assess their efficacy in predicting punching shear capacity. Three code equations (AASHTO, CSA A23.3 and CSA S6-06) as well as a failure criterion proposed in the literature were found to be conservative in predicting the capacity of the FRP SIP formwork decks in question.

KEYWORDS

FRP, bridge decks, Stay-in-Place formwork, punching shear, slab on girder, durability, reinforced concrete.

INTRODUCTION

The concept of Stay-in-Place (SIP) formwork for concrete slabs has existed for some time. The system uses panels (usually cold rolled steel or precast concrete) as both formwork for fresh concrete and as bottom reinforcement for the resulting slab. This concept allows for quicker construction and eliminates the task of stripping formwork. It can also result in economies when the permanent SIP formwork acts as a composite bottom layer of reinforcement, replacing the traditional rebar mat. The technique is particularly effective in slabs of regular geometry with parallel supporting girders such as bridge decks.

More recently, this technique has been combined with the durability advantages of Fibre Reinforced Polymer (FRP) composite materials in a system called FRP SIP formwork. Targeted particularly at concrete bridge decks of slab on girder bridges, this system has been investigated experimentally (Alagusundaramoorthy et al (2006), Deiter (2002), Nelson and Fam (2012)), analytically (Cheng and Karbhari (2006), Mufti and Newhook (1998)) and in three major field applications (Reising et al (2004), Berg et al (2005), Matta et al (2006)). Experimental investigations of FRP SIP formwork performance has overwhelmingly consisted of the testing of bridge deck sections in the laboratory. These sections represent a single (or sometimes two) girder to girder span of deck, with a finite width bounded by free edges. These deck sections generally have an aspect ratio (width/length) less than or approaching 1. Due to the considerable difference in boundary conditions, such tests provide conservative results which at best approximate the in-place capacity of an actual bridge deck.

This paper reports on a study aimed at examining this divergence by testing bridge decks under accurate boundary conditions. This has been accomplished by constructing an entire slab on girder bridge at 1:2.75 scale and conducting punch through tests at various locations on the bridge. A similar approach was used by Batchelor et al (1978) to investigate punching shear in traditionally reinforced bridge decks. By testing both bridge deck
sections and test pads within a full bridge, the study illuminates the divergence in performance of these two cases. In addition to discussing the performance of the system vis-à-vis its boundary conditions, the results of testing will also be compared to some failure criteria. The suitability of these predictions will be discussed with recommendations.

**EXPERIMENTAL INVESTIGATION**

**General Design of FRP SIP Formwork System**

The design of SIP Formwork systems tend to be heavily influenced by the geometry of the form panel. In this study, a particular ribbed panel was used as permanent formwork. The panel is composed of Glass FRP (GFRP) and produced using the pultrusion method. In the longitudinal (stiff) panel direction, it consists of a 4.2 mm thick plate stiffened by T shaped ribs. The panel is 40 mm deep including the ribs. An image of the panels can be seen in Figure 1a. In the transverse direction, the panels act essentially as a flat plate. The panel’s longitudinal and transverse properties are different due to fibre architecture. A detailed description of the panels can be found in Nelson et al (2013).

In this case, the FRP panels act as formwork and are cast monolithically with a concrete deck 65 mm in thickness. An orthotropic grid of GFRP rebar is added to the top of the slab for shrinkage and crack control purposes.

**Construction of Slab on Girder Bridge**

A 4 girder slab on girder bridge was constructed at 1:2.75 scale during the course of this investigation. Construction of the bridge can be seen in Figure 1b. The bridge was designed at full scale using Canadian code requirements and scaled geometrically. It was designed as a precast prestressed concrete superstructure with a composite deck slab. Due to the difficulty of scaling prestressed concrete girders, composite steel-concrete girders were used with their flexural and torsional stiffness closely matched to that of the original concrete girders.

The bridge was constructed in a similar fashion to conventional reinforced concrete, with girders being placed on neoprene bearings at either abutment. Placing of the panels followed the construction process previously used for this configuration of FRP SIP formwork by Nelson et al (2012). After placing FRP form panels, GFRP rebar was placed and the entire slab was poured by hopper.

In addition to FRP SIP formwork, some of the bridge deck was constructed using conventional reinforced concrete. Small diameter deformed steel wire was used as reinforcement and traditional wood formwork was employed during construction. The effective depth of both the conventional RC and FRP SIP formwork decks were identical.

**Test Setup and Instrumentation**

Figure 2 shows a punching test in progress on the bridge deck. These tests were conducted in various locations on the deck, but always on the centreline between adjacent girders. The tests were spaced far enough apart that the damage caused by one pad failure was judged not to affect the next adjacent pad. Observations during testing and mapping of crack patterns supported this assertion. Loading was provided by a hydraulic ram and was
applied using a simulated neoprene tire patch conforming to AASHTO (2007) specifications. A spherical seat was used to remove any eccentricity. Load was monitored using a 445 kN load cell. A variety of sensors were used to gather real-time deflection and strain data, including strain gauges, Pi gauge transducers and Linear Potentiometers (LP). They were located to permit deflected shape and strain distributions to be calculate for the bridge. Additionally, LPs were used to measure the girder deflections during testing. These deflections were subtracted from readings taken on the deck itself to remove any effects of superstructure deflection from the results.

RESULTS AND DISCUSSION

Experimental results

The results of four tests are considered in this article. The four punching tests were conducted on specimens of various geometries and designs as follows:

(1) Constructed using FRP SIP formwork and top FRP rebar. This deck section test at 1:2.75 scale spanned 665 mm from girder centre-to-centre and had a width of 604 mm. It is representative of the aspect ratio of deck sections commonly tested in the laboratory at full scale (this test was completed as part of Nelson et al (2013)).

(2) Similar to test 1 but part of the full bridge deck. This test pad is identical to test 1 except for its boundary conditions (the width of this deck section is effectively larger than 4000 mm).

(3) Similar to test 2, except the span in this case is 886 mm. It is also part of the full bridge.

(4) A conventional reinforced concrete deck pad, part of the full bridge. Span and boundary conditions are identical to test 3.

Figure 3 plots the load deflection results of these four tests. The deflection data for these tests were measured at the centreline of the span, directly below the application of load.

The effect of boundary conditions can be seen clearly by comparing the results of test 1 and test 2. Although both decks possess the same span and are of the same design, the finite nature of the bridge deck section results...
in a lower stiffness and strength. Both test pads failed in punching shear, test 2 at 149.4 kN and test 1 at 119.6 kN, a reduction in capacity of 20%. The post-cracking stiffness of test 2 and test 1 were 32.9 kN/mm and 25.3 kN/mm respectively, a 23% decrease. The effect of span length is illustrated by test 2 and 3. Deflections are significantly higher in the longer span; however punching shear load is almost unaffected. This is an unexpected result as large numbers of punching tests of conventional reinforced concrete slabs shows a strong correlation between deflection (slope) and punching shear load (Guandalini (2005)). On the other hand, most codes do not consider the span or deflection/slope in calculating punching shear capacity. Comparing test 3 and 4, we can see that the pre cracking stiffness of the steel deck is higher due to the greater section depth h, while after cracking the similar effective depth results in comparable stiffness.

Test results also demonstrate the proximity of the punching shear load to the point of flexural failure for the test slabs. Figure 4 plots longitudinal (traffic direction) and transverse strains measured in the FRP formwork panels underneath the load pad. Maximum tensile strains reached just over 0.003 at the moment of punching shear. This is small compared to the rupture strain of the FRP of 0.0121 and 0.0088 (transverse and longitudinal respectively). On the other hand, measured concrete compression strains reached -0.0027 in test 1 and -0.0034 in test three, indicating the test was approaching the point of concrete crushing.

Figure 4. Panel strains from test 1 and 3

Comparison with failure criteria

Design codes dealing with reinforced concrete often present a method for calculating the punching shear capacity of a concrete flat slab. In addition, more accurate (less conservative) failure criteria have been proposed academically which take into account additional factors such as curvature in calculating punching shear capacity. The test data presented herein will be compared to clauses from two design codes and a theoretical failure criterion. Punching shear capacity will be calculated using AASHTO LRFD, CSA A23-3, CSA S6-06 and a failure criterion proposed by Muttoni (2003).

The codes all use a similar formula for calculating punching shear capacity, based on determining the shear stress resistance at some critical parameter. In all three cases, this is ultimately based on the concrete strength and some empirical constants. The AASHTO method for determining punching shear resistance is as follows:

\[
V_p = \left(0.17 + \frac{0.33}{\beta_c}\right) \sqrt{f'_c} b_0 d_v
\]  

(1)

Where \(\beta_c\) is based on the geometry of the loaded area, \(f'_c\) is the concrete compressive strength and \(b_0\) and \(d_v\) are the properties of the critical shear section. In the case of AASHTO, the shear perimeter is the smallest perimeter around the concentrated load such that it remains at least \(d_v/2\) away from the edge of the load. It is evident from this formula that the method does not take into account the state of deformation of the deck at the point of failure and therefore predicts punching shear independent of span or boundary conditions (except where boundary conditions impinge on the critical shear section). The A23.3 method uses three formulas, the lesser of which determines the shear stress resistance of the concrete. In this case, the following formula governs:

\[
V_c = \left(1 + \frac{2}{\beta_c}\right) 0.194 \sqrt{f'_c}
\]  

(2)

Where \(\lambda=1\) for normal density concrete. This stress is then applied to a critical shear section (similar to above) to calculate the resistance to punching shear. The S6-06 method follows essentially the same process only with
slightly different empirical constants. Note that none of the calculations herein have been executed with load or resistance factors.

Figure 5 displays the test results mentioned earlier along with the four failure criteria discussed in this section. All the criteria predict the capacity of the sections conservatively, particularly in the case of the full bridge test pad (2, 3 and 4). The AASHTO method can be seen to be overly conservative in predicting the capacity, under predicting the punching shear capacity of the tests by between 32 and 53%. The A23.3 and S6-06 methods are more accurate, but still under predict results of full bridge pads. The failure criterion developed by Muttoni falls in the centre of the code predictions but is expected to provide more accurate results over a larger range of test data. It is more effective because it considers the effect of slab deformation on punching shear capacity.

Figure 5. Comparison of test results and predicted capacity

CONCLUSIONS

Boundary conditions play a critical role in assessing the performance of FRP SIP formwork bridge decks. In this study several scaled test results are reported which examine these effects, comparing results from a bridge deck section test to test pads within a full slab on girder bridge. These results were also compared to several failure criteria, both theoretical and code based, to assess their ability to predict punching shear failure in FRP SIP formwork bridge decks. The following pertinent conclusions can be drawn:

1. Bridge deck section tests (the most common laboratory test setup used to investigate FRP SIP form systems) produce results which do not represent the actual performance of full bridge decks. Deck section tests are conservative in their representation of strength and stiffness, in this case by 20 and 23% respectively.
2. Girder to girder span has a significant effect on the deflection performance of bridge decks but a much less pronounced effect on the punching shear capacity.
3. Results from a conventional RC deck and an FRP SIP formwork deck indicate similar post-cracking stiffness. The RC deck exhibited higher pre cracking stiffness and 15% higher capacity.
4. Code equations from the AASHTO bridge design guide, CSA A23.3 and CSA S6-06 are conservative in predicting the punching shear capacity of FRP SIP formwork bridge decks, particularly full bridge decks. The CSA standards were found to be more accurate in this particular case.
5. The punching shear failure mode proposed by Muttoni (2003) was of similar effectiveness as the design codes in predicting capacity for the specimens considered here, but it is expected to be accurate over a larger range and configuration of decks.

REFERENCES


