BEHAVIOUR OF CONCRETE BRIDGE DECK
MODEL REINFORCED BY CARBON FRP

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ABSTRACT

This paper describes an experimental program and analytical model of a bridge deck slab reinforced by carbon fibre reinforced polymers, CFRP reinforcement. The model consists of three continuous spans and two cantilevers with a thickness of 200 mm. Each span and the two cantilevers were tested independently using a concentrated load simulating a truck wheel and three different boundary conditions. Optical fibre sensors were used to monitor the strain of the CFRP reinforcement in conjunction with conventional electrical strain gauges. Analytical model was carried out using two finite element softwares using linear elastic and non-linear analysis and accounting for cracking of the concrete. The in-plane membrane forces induced by the different lateral restraints was determined. Test results, failure mechanism, comparison between measured and calculated values are presented.

INTRODUCTION

Deterioration of concrete bridge decks are mainly attributed to corrosion of the steel reinforcement due to exposure to chloride environments. Fibre reinforced polymers, FRP, offers an effective alternative to the use of conventional steel in these instances due to its non-corrosive characteristics. FRP has been used successfully as reinforcements and for prestressing several concrete highway bridges in Canada, USA, Europe and Japan (Rizkalla and Tadros 1994 and Minosaku 1992).
Construction of the world’s largest span bridge using CFRP as prestressing and shear reinforcement for four girders and for part of the deck slab was completed in Headingly, Manitoba in October 1997 (Rizkalla et al. 1998). Before the construction of the bridge, testing of six pretensioned concrete beams of 1:3.6 scale model of the bridge girders was performed (Fam et al. 1995). In this paper, the results of testing a full scale model of the deck slab is reported. The testing was conducted at the University of Manitoba to examine the constructibility and different limit state behaviour of concrete deck slab using CFRP. Since the lateral restraint of the slab provided by the supporting girders and the diaphragms influences the ultimate capacity of bridge decks due to the induced in-plane compressive forces, the stiffness of the model and the supporting beams was designed to simulate the same stiffness of the prototype.

The membrane forces are affected by the torsional stiffness of the main girders, stiffness of the cross diaphragms and the presence of the adjacent slabs. It is also affected by span-to-depth ratio of the slab, and reinforcement ratio. The Ontario Highway Bridge Design Code, (OHBDC 1991), accounts for the effect of these forces and permits the use of less flexural reinforcement than would be required by other codes, provided that at least two cross diaphragms are provided at the support lines. The OHBDC also requires that the slab extends at least 1.0 metre beyond the centre line of the external supporting girder.

The bridge deck model was analysed using two finite element programs. The failure mode and ultimate carrying capacity were examined under the effect of a single concentrated load acting on a contact area specified by the modified AASHTO Code for HSS 25 (MSS 22.5) truck. Effect of lateral and longitudinal restraint of the slab on the membrane action and the ultimate punching shear capacity was studied. The lateral restraint was used to simulate the effect of cross diaphragms and the restraint in the other direction simulated the continuity of the slab in the longitudinal direction. Effect of edge stiffening on the performance of the slab was also studied. Serviceability of the slab, strain distribution and performance of optical fibre sensors attached to CFRP reinforcement are also evaluated.

**BRIDGE DESCRIPTION**

CFRP reinforcement is used exclusively for prestressing and as shear reinforcement for four girders of the world’s largest span bridge. One section of the deck slab with two lane width is reinforced by CFRP reinforcement in replacement of the conventional steel reinforcement. Taylor bridge is located on Provincial Road No. 334 over the Assiniboine River in the parish of Headingley, Manitoba, Canada. The total length of the five equal span bridge is 165.1 m. Each span consists of eight I-shaped precast prestressed concrete girders spaced at 1.8 m centre-to-centre as shown in Fig. 1. The I-girders are of AASHTO type with 1.83 m depth. Five cross diaphragms are located in the transverse direction for each span and spaced at 8.1 m. The deck slab is 275 mm thick and is designed to act in a composite action with the bridge girders. The top 70 mm of the concrete slab is used as a wearing surface.
BRIDGE DECK MODEL

A full scale model of concrete bridge deck with overall dimensions of 7.2 x 3.0 x 0.2 metres was constructed. The model was supported by four 350 x 750 mm precast prestressed concrete beams with steel stirrups extended in the slab to provide the composite action. The 3 metres span beams were simply supported and spaced at 1.8 metre centre-to-centre. The model included three spans and two cantilevers as shown in Fig. 2. The slab was reinforced by indented Leadline CFRP bars produced by Mitsubishi Chemicals Corporation, Japan. The bottom reinforcement was 2-10 mm bars spaced at 125 mm in the main direction and 1-10 mm bar at 125 mm in the other direction. The top reinforcement consisted of 1-10 mm bar at 125 mm in each direction. The CFRP bars are produced using Dialead coal tar pitch based continuous carbon fibre and an epoxy resin. The bars have a linear tensile stress-strain characteristic with an ultimate guaranteed strength of 2250 MPa and an elastic modulus of 147 GPa as reported by the manufacturer. The average concrete compressive strength and elastic modulus were 59 MPa and 36,000 MPa, respectively.

The design criterion of the model was based on achieving the same rotation of the supporting beams of the model as the bridge girders. Based on a finite element analysis performed on the bridge girders, a steel strap was added to the model at the ends of the supporting beams, as shown in Fig. 2, to simulate the cross diaphragms in the bridge. The stiffness of the steel straps was selected based on the finite element analysis.

Testing Scheme

The slab was loaded using a quasi-static single concentrated load using a hydraulic jack of 1800 kN capacity as shown in Fig. 3. The load was monitored using a load cell of the same capacity as the hydraulic jack. The load was applied through 225 x 575 mm steel plate to simulate the contact area of a truck wheel according to the AASHTO Bridge Design Code 1996. A neoprene pad was placed between the steel plate and the slab to avoid local crushing of the concrete surface. The strain of the CFRP reinforcement was monitored using 64 electrical strain gauges as well as eight optical fibre sensors.

The slab was tested at three different locations as shown in Fig. 2. The applied load was cycled three times every 200 kN. The first test of the mid-span slab was performed using steel straps connected to the two ends of the supporting beams to restrain the rotation and lateral movement up to a load of 600 kN. At this load level, the measured steel strain indicated yielding of the straps and, therefore, the slab was unloaded. The end restraint were removed and the slab was reloaded without the presence of the straps up to failure. Test # 2 was conducted on the outer span of the continuous slab using stiffer straps cross section up to failure. Test # 3 included testing of the other outer span using the same strap used in test # 2. The edges of the slab in the third test were stiffened using four HSS steel sections of 203 x 203 mm supported horizontally using four-25 mm diameter Diwidag bars to restrain the slab in the direction of the supporting beams as shown in Fig. 2. The longitudinal restraint of the slab was provided to simulate the continuity of the slab in the traffic direction.
TEST RESULTS

The slab failed due to punching shear for the three tests at load levels of 1000, 1200, and 1328 kN, respectively. The failure load is more than 10 times the service load of 90 kN according to the AASHTO code. The presence of stiffer end restraint increased the ultimate carrying capacity of the deck slab by 20 percent. Edge stiffening and continuity of the slab increased the capacity by an additional 12 percent.

Load-Deflection Behaviour

A typical load-deflection relationship of the deck slab under repeated load is shown in Fig. 4. The load-deflection was linear up to cracking and non-linear after cracking with reduced stiffness. The residual deflection was less than 1.0 mm for test # 2 after applying three cycles at a load level of 800 kN. This behaviour is attributed to the linear characteristics of the CFRP reinforcement. The envelops of the load-deflection relation of the three tests are shown in Fig. 5. The non-linearity of the load-deflection was more pronounced for the test without end restraint due to the presence of extensive cracks at the top and bottom of the slab. The stiffness of the slab at the second and third tests with end restraint was similar, however, the deflection was slightly higher than that in the first test since the top surface of the slab at the support location was already cracked before the application of the load.

Crack Pattern and Failure Mode

Cracking at the bottom of the slab was observed at load levels of 100, 92 and 132 kN for the three tests. This indicates that edge stiffening increased the cracking load by 40 percent. The cracks on the top surface occurred at load levels varying from 240 to 380 kN. Crack patterns at the bottom of the slab are compared in Fig. 6 for the three tests at load level of 600 kN. Flexural cracks were more pronounced in the first test without end restraints than the second test with end restraint. Radial cracks were more pronounced in the third test due to the presence of edge stiffening and end restraint.

The failure mode of the three tests was punching shear of the deck slab as shown in Fig. 7. After failure, the slab was cut using a diamond saw to measure the angle of the plane of failure, (θ), with the horizontal. The angle θ was measured to be ranging from 35° to 43° for the first test and equal to 35° for the second test. For the third test, it was found that the angle θ was 20° for the crack running in the direction of the supporting beams towards the far end of the stiffened edge. (1.9 m away from the edge of the slab) and 80° for the crack running towards the closer end of the stiffened edge.

Strain Distribution

The concrete strain was measured at different location at the top and bottom surface of the slab. The maximum compressive concrete strain measured at the mid-span at failure was 0.0029, 0.0027, 0.0026 for the three tests respectively. The strain in the CFRP reinforcement was also
measured using conventional strain gauges and optic fibre sensors. A very good correlation between the strains measured by the gauges and the optic sensors can be seen in Fig. 8. The tensile strain at failure of the CFRP reinforcement in the main direction was 0.004 and 0.0031 for tests #1 and 2 respectively. While the tensile strain at failure of the CFRP reinforcement in the longitudinal direction was 0.0017 and 0.0014 for tests #1 and 2 respectively.

ANALYTICAL MODEL

The behaviour of the deck slab was predicted using two different programs using linear elastic and non-linear finite element analysis. In the linear analysis, an eight node solid element was used to model the slab and the supporting beams. The slab was divided into three layers within its thickness. The non-linear finite element analysis accounts for biaxial, triaxial state of stresses as well as tension stiffening of the concrete. Cracking in the concrete was also accounted for using a smeared crack model for general 3D stress states taking in consideration crack closure and re-opening under cyclic loading. The slab was divided into three layers using a 20 node brick element. A step-by-step analysis was used in the non-linear analysis. The prestressed supporting beams were installed at the first step, followed by placement of the concrete slab. The load was applied in the middle slab to simulate the first test with end steel straps at a rate of eight kN/step until a load of 600 kN was reached. The slab was unloaded and the straps were removed at zero load. The load was re-applied at an average rate of loading of 3.2 kN/step until failure.

The predicted load-deflection of the slab in test #1 with and without end restraint is shown in Fig. 9 using the linear and non-linear finite element analysis. For test #1 with end restraint, it can be seen that the deflection can be predicted with a very good accuracy using either the linear or non-linear analysis before cracking. After cracking, the linear analysis underestimates the deflection. While the non-linear analysis could predict the deflection of the slab accurately. After removal of the end restraint and since the slab was already cracked, the linear analysis underestimates the deflection. The non-linear analysis showed a residual deformation and could predict the load-deflection relationship accurately.

The maximum tensile strain in the bottom CFRP reinforcement at the mid-span section was predicted as shown in Fig. 10 using the non-linear analysis. The strains are shown for test #1 with and without end restraint. It can be seen that the predicted strains matches very well the measured values.

The distribution of the in-plane membrane forces in the longitudinal direction is shown in Fig. 11 at the support section. The membrane forces are compressive close to the applied load and reduces away from the load. It was also noticed that adding the end restraint increases the compressive membrane forces at the location of the load.

The ultimate capacity of the concrete slab was predicted by the ACI and the OHBDC codes. The codes assume that the plane of failure due to punching shear is at d/2 from the boundary.
of the concentrated load, where \( d \) is the depth of the slab. The codes do not take into consideration the effect of lateral restraint of the supporting beams. The punching shear capacity was predicted to be 806 and 658 kN for the ACI and OHBDC codes, respectively, which is conservative compared to the measured value ranging from 1000 to 1328 kN.

**CONCLUSIONS**

A full scale bridge deck model reinforced by CFRP reinforcement was tested with different boundary conditions. A flexural behaviour was observed, however, the failure mode was due to punching shear. From the test results of this study, the following conclusions could be drawn:

1. The failure load of the slab had a lower bound of 1000 kN which is more than 10 times the service load.
2. Restraining the slab laterally increased the ultimate carrying capacity by 20 percent. Edge stiffening increased the capacity of the slab by an additional 12 percent.
3. Serviceability requirements in terms of cracking and deflection were satisfied for the bridge deck model.
4. The behaviour of the slab can be predicted accurately using a non-linear finite element analysis.
5. Adding lateral restraint increased the compressive membrane force.
6. The ACI and OHBDC give a conservative prediction for the punching shear capacity of the concrete slab reinforced by CFRP.

**REFERENCES**


Fig. 1 Schematic of the layout of the bridge

Fig. 2 Schematic of the bridge deck model
Fig. 3 Model of the bridge deck

Fig. 4 Load-deflection of the second test

Fig. 5 Envelop of the Load-deflection of the three tests
600 kN  600 kN  600 kN

Test #2  Test #1  Test #3
laterally restrain  unrestraint  laterally restraint

and edge stiffening

Fig. 6 Crack pattern at the bottom surface of the tested slab

Test #1
P = 1000 kN
unrestraint

Test #2
P = 1200 kN
laterally restraint

Test #3
P = 1328 kN
edge stiffening

Fig. 7 Punching failure of the slab

Fig. 8 Comparison between the strain
Fig. 9 Predicted vs. Measured Load-deflection of the first test

Fig. 10 Maximum tensile strain in the CFRP reinforcement

Fig. 11 Distribution of the membrane forces in the longitudinal direction of the slab