BEHAVIOUR OF A MODEL CONCRETE BRIDGE DECK
REINFORCED BY CFRP

Kenneth CHARLESON\textsuperscript{1}, Amr ABDELRAHMAN\textsuperscript{2}, Sami RIZKALLA\textsuperscript{3}
and Walter SALTZBERG\textsuperscript{4}

ABSTRACT: Headingly, Manitoba will be the site of the first bridge in Canada to incorporate carbon fibre reinforced polymer (CFRP) reinforcement into the bridge. Prior to the construction, a full scale model of a portion of the bridge deck was built and tested under static loading conditions at the University of Manitoba. The 200 mm thick model deck slab has a plan area of 2950 x 7200 mm and is supported by four precast girders at 1800 mm on centre. The design represents a cut out section from the actual bridge. The effects of stiffness of the supporting girders and diaphragms have been incorporated into the model to simulate the overall structural response of the slab. Optical fibre sensors as well as conventional electric resistance strain gauges were used to monitor the strain of the CFRP. Behaviour of the deck slab, including the mode of failure under static loading conditions, are presented.

KEYWORDS: bridge, carbon, concrete, deck, FRP, fibre, optic, model, punching, slabs.

1 INTRODUCTION

Billions of dollars are spent annually on maintenance and repair of concrete highway bridges in Canada. The maintenance and repair budget is extremely high in comparison to the original capital investment of the structure. Most bridge deterioration can be attributed to corrosion of reinforcing steel due to their exposure to chloride environments. Fibre reinforced polymer (FRP) provides an effective preventative measure to this problem due to their non-corrosive properties.

Construction is underway for the first bridge in Canada to incorporate carbon fibre reinforced polymer (CFRP) into the deck slab. The bridge is located in Headingly, Manitoba, Canada. The overall length of the bridge is 165.1 metres and consists of five spans each 32.5 metres as shown schematically in Figure 1. The superstructure consists of 1816 mm deep prestressed "I" girders with a 275 mm concrete deck slab reinforced to act in composite action with the girders. The bridge is designed for a modified HSS 25 (MSS 22.5) vehicle loading as specified by the AASHTO [1] bridge code. One section of the deck slab will be reinforced with CFRP in

\textsuperscript{1} University of Manitoba, Canada, M.Eng.
\textsuperscript{2} ISIS Canada Network, University of Manitoba, Canada, DR
\textsuperscript{3} ISIS Canada Network, Professor of Civil Eng’g, Univ. of Manitoba, Canada, DR
\textsuperscript{4} Manitoba Highways and Transportation, Bridges and Structures Dept., Canada, P.Eng.
replacement of the conventional steel reinforcement as shown in Figure 1. Fibre optic sensors will be attached to the CFRP reinforcements used for both prestressing the girder and reinforcement of the slab to monitor the strains, and therefore providing information about overall structural response.

![Figure 1: Headingley Bridge](image)

This paper presents the results of testing a full scale model of the Headingley bridge deck reinforced by CFRP. The model is a continuous slab supported by four girders and tested under a quasi static single concentrated load simulating the wheel load of a truck. Two spans have been tested under different conditions of lateral restraint of the supporting girders. The tests were conducted at the University of Manitoba. The strain distribution in the CFRP reinforcement was monitored using optical fibre sensors as well as electrical strain gauges attached to the surface of the reinforcement. Overall response, crack pattern and failure mode of the slab are presented.

2 DECK SLAB MODEL

The full scale model of the slab was used to simulate the response of the bridge deck slab subjected to an equivalent vehicle loading. The model, shown in Figure 2, represents a cut out section of the bridge. The model is supported by four precast concrete beams at 1800 mm centre to centre with steel stirrups extended into the slab to provide composite action. The model includes two cantilevers of 725 mm clear span. The overall dimensions of the model is 7200 mm x 2950 mm with a 200 mm slab thickness. Tests were conducted on the centre and west spans as shown in Figure 2, designated by test #1 and test #2 respectively.

The model is reinforced with 10 mm indented Leadline CFRP bars produced by the Mitsubishi Chemical Corporation, Japan. The bars are made using Dialead coal tar pitch based continuous carbon fibre and an epoxy resin. The primary reinforcing in the bridge deck slab, which is the
bottom reinforcing in the transverse direction, is specified based on the equivalent steel
reinforcement ratio specified by the Canadian code [3]. The area of the CFRP was provided
based on modification of the same requirements to account for the lower elastic modulus of FRP
compared to steel as follows;

\[ A_{\text{FRP}} = 0.003 A_s \left( E_s / E_{\text{FRP}} \right) \]  \hspace{1cm} (1)

where \( A_{\text{FRP}} \) is the specified area of FRP reinforcement, \( A_s \) is the gross area of the concrete cross
section, and \( E_s \) and \( E_{\text{FRP}} \) are the moduli of elasticity of steel and FRP respectively. The remaining
reinforcement is specified based on the minimum reinforcing requirements of steel which is 0.2
percent in each direction.

![Model Bridge Deck](image)

**Figure 2: Model Bridge Deck**

Eight fibre optic sensors were attached to the CFRP reinforcement in the central span of the
model. Bragg grating fibre laser sensor systems were used to monitor the sensor readings. The
sensors were paired with conventional electric resistance strain gauges to confirm the effectiveness
of the fibre optic sensors cast in this severe environment. A total of sixty-six conventional strain
gauges were attached to the surface of the CFRP bars. Concrete strain of the slab was measured
using Pi-gauges and lateral and vertical deformation of the entire model were monitored by
LMT/L.VDT's and dial gauges. The equivalent wheel load was applied using an 1800 kN capacity,
stroke controlled, Enerpac hydraulic jack applied through a 225 x 575 mm steel plate as shown
by Figure 3. This simulated the static action of a vehicle load as specified by the AASHTO Bridge
design Code [1]. The load for both tests was cycled three times at levels of 200, 400, 600 and
800 kN. The load for each test was then increased to failure.
Steel restraint straps were used at the ends of the west span supporting beams during test #2 to constrain the lateral movement and to simulate the torsional and lateral stiffness provided by the end and middle diaphragms of the bridge as shown by Figure 2. Finite element analysis was used based on uncracked sections of the slab and beams to design the restraint. Steel restraint straps were not used for Test #1.

3 TEST RESULTS AND DISCUSSION

The primary requirement of this experimental program was to verify the suitability of CFRP Leadline as a reinforcement for the Headingley bridge deck slab. The evaluation includes serviceability requirements and ultimate capacity under design and factored loading conditions. Ultimate loads for test #1 and #2 were 1000 kN and 1200 kN respectively with failure occurring by punching shear in both cases. The specified wheel loading according to AASHTO is 89 kN. The maximum measured strain in the bottom Leadline reinforcement at ultimate for test #1 was 4 millistrain, compared to an ultimate value of 16 millistrain. Under design load conditions, the maximum measured deflection of the slab was less than 1 mm.

3.1 CRACK PATTERN AND FAILURE MODE

The overall crack pattern for test #1 and #2 were identical and is shown for test #1 in Figure 4. Within the first 400 kN load cycle, longitudinal cracks were observed extending on each side of the loaded area. Radial cracks were observed at load levels of 600 kN and 400 kN for test #1.
and #2 respectively. Longitudinal cracks projecting in a predominant diagonal direction were observed during the load cycles of 600 and 800 kN. The top surface of the slab was cracked in a rectangular shape running parallel and adjacent to both support girders and extending perpendicular towards the middle of the slab approximately 250 mm on each side of the loaded area. At failure, the top surface of the displaced concrete cone was approximately equal to the loaded area. The failure mode of the slab in both tests was punching shear. The overall behaviour of the slab is described as a hybrid type of behaviour which is a combination of flexural and punching shear. Predominant longitudinal cracks and cracks on the top surface of the slab adjacent to the support beams reflects the flexural behaviour mechanism. However, the localized radial cracks at the bottom of the slab as well as the failure of the concrete around the loading plate were clear evidence of punching shear failure.

![Figure 4: Test #1 Crack Pattern at Failure (slab underside)](image)

3.2 LOAD-DEFLECTION

The overall load deflection relationship in the last cycle for tests #1 and #2 is shown in Figure 5. The load deflection is shown for all the cycles of test #2 in Figure 6. The first cracking of the slab occurred at 220 kN and 200 kN for tests #1 and #2 respectively. Successive cycles of loading increment caused relatively small or negligible permanent deformations. Figure 7 compares the rotation of the supporting beams for test #1 and #2. The slab behaviour indicates that there was a considerable increase in the stiffness due to installation of the end restraint straps on the supporting beams.

3.3 STRAIN DISTRIBUTION

The maximum observed tensile strain in the bottom reinforcement in the main direction is 4 millistrain and 3.1 millistrain for both tests #1 and #2 respectively. This compares to strain in the top CFRP reinforcement in the main direction of 1.4 millistrain and 3.4 millistrain. As expected, the strains for both tests were high in the vicinity of the load and reduced gradually toward the free end of the slab.
3.4 PERFORMANCE OF OPTICAL FIBRE GAUGES

Optical fibre gauges mounted on CFRP reinforcing bars in the model performed very well through the test. All gauges provided stable readings through load cycling and up until failure. Strain readings of one of the optical fibre gauges positioned immediately under the loaded area and a conventional electric resistance strain gauge located adjacent is shown in Figure 8. The comparison shows an excellent correlation.

3.5 PREDICTION OF ULTIMATE LOAD CAPACITY

Predictions for ultimate load were made based on punching shear equations provided by CSA [3], ACI [4], OHBDC [2] and a model developed by Newhook and Mufti [5]. The predictions
are shown in Figure 9. Results indicate that the CSA code provides a very good prediction for test #1 despite the fact that the code equation is for slabs reinforced by steel. The ACI and OHBDC code equations provide a conservative result. The model by Newhook and Mufti, which accounts for compressive membrane forces developed by lateral girder restraint, overestimates the prediction for both tests.

![Graph showing comparison of strain gauges](image)

**Figure 8: Test #1 Comparison of Response of Fibre Optic and Conventional Strain Gauges**

![Bar chart showing failure loads](image)

**Figure 9: Deck Slab Failure Load Predictions**

### 4 CONCLUSION

Based on testing of a full scale model of a concrete bridge deck reinforced by CFRP, the following conclusions can be drawn:

1. Ultimate loads of 1000 kN and 1200 kN for tests #1 and #2 respectively were sustained by the model compared to an unfactored static design load of 89 kN as specified by the AASHTO bridge design code. This represents an acceptable level of safety. Failure for both tests occurred by punching shear.
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(2) Adding external lateral restraint to the supporting girders increased the failure load by 20% but did not change the failure mode. Therefore, the presence of the diaphragm would increase the load carrying capacity of the bridge deck slab.

(3) The model displayed satisfactory performance in terms of serviceability requirements. The deflection and strains were very small at the service load limit.

(4) Optical fibre sensors mounted on the CFRP bars within the model showed very good correlation with conventional electrical resistance strain gauges indicating satisfactory performance.

(5) The predicted failure load for test #1 based on the CSA code is satisfactory. Prediction of the punching shear load according to ACI and OHBDC codes are conservative.

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