MODEL TESTS FOR CONCRETE HIGHWAY BRIDGES IN MANITOBA
FULLY REINFORCED BY CFRP REINFORCEMENTS

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ABSTRACT

Accepting the challenge of the future in the field of civil engineering smart structure applications, the province of Manitoba is undertaking the responsibility of constructing the first bridge in Canada totally prestressed and reinforced for shear by carbon fibre reinforced plastic (CFRP) reinforcement. Portion of the concrete deck slab is also reinforced exclusively by FRP reinforcements. This paper describes the experimental program conducted at the University of Manitoba, Canada, to examine the behaviour of 1:3.6 scaled model of the bridge girders. The paper also discusses the analytical model used to predict the behaviour. A comparison between the predicted and the measured behaviour in terms of load-deformation and strain distribution are presented.
INTRODUCTION

Service life and serviceability of concrete bridges are significantly affected by deterioration of concrete due to corrosion of steel reinforcement especially in Canada where bridges are exposed to harsh environments characterized by a wide range of temperature variation and use of salt for de-icing in addition to the typical affects of cyclic and impact loads of traffic. Different techniques have been proposed to overcome this problem. The cost of correcting corrosion induced distress in bridges is high compared to the capital cost of the structure. Use of non-metallic reinforcement is considered to be one of the most promising avenues to overcome this problem. The province of Manitoba, Canada, is undertaking the challenge to construct the largest span highway concrete bridge with 32.5 meter span using two types of carbon fibre reinforced plastic, (CFRP), reinforcements for prestressing and stirrups. The two types of CFRP are the Carbon Fibre Composite Cable, (CFCC), produced by Tokyo Rope, Japan and Leadline reinforcing bars produced by Mitsubishi Kasei, Japan. In addition to their excellent corrosion free characteristics, the two types of materials also have outstanding properties in terms of high strength-to-weight ratio, high fatigue strength, low relaxation, non-magnetic conductivity, and ease of handling and installation due to their light weight.

This paper summarizes the experimental program conducted at the University of Manitoba to test a total of six beams, 1:3.6 scaled model of the bridge girders. The main features of this experimental program are draping of the prestressing strands, using CFRP for shear reinforcements, and extending the CFRP stirrups into the slab to provide the dowel action between the deck slab and the main pretensioned girders.

PROPOSED BRIDGE

The bridge is located over the Assiniboine River, Parish of Headingley, Winnipeg, Manitoba. The bridge consists of five spans, 32.5 meters each, covering a total length of 165.1 meters. All the bridge girders are precast pretensioned and simply supported. The girders have an I-section, AASHTO type, transversely spaced at 1.8 meters and supporting 187 mm thick deck slab. Fig.1 shows a typical pretensioned concrete girder of the bridge.

RESEARCH SIGNIFICANCE

The main objective of the experimental program is to evaluate the performance of prestressed concrete I-girders totally reinforced for shear and prestressing by CFRP reinforcements in terms of ultimate resistance, deflection, and flexure cracks. The study is focused on the performance of the CFRP stirrups and its effect on the behaviour of the diagonal shear cracks and their capability to provide the dowel action. The beams were also tested to examine the efficiency of the draped prestressing CFRP tendons with an angle of
four degrees, which is typically used for these type of bridges, to achieve uniform and allowable stress distribution at the ends of the girder.

EXPERIMENTAL PROGRAM

A total of six beams were fabricated by Con-Force Structures Company Ltd., Winnipeg, Manitoba, Canada for this experimental program. All test beams were 1:3:6 scale model to the bridge girders with an overall length of 9.3 meters. The beams had an I-shape cross section of an overall depth of 500 mm. Similar to the prototype girders 40 percent of the prestressing reinforcement were draped at a distance of 40 percent of the span from the beam ends with four degrees angle as shown in Fig.2. The hold down system used for draping the strands consisted of 33 mm diameter stainless steel pins and sleeves free to rotate as shown in Fig.3. The top slab, which was 500 mm wide and 50 mm in depth, was casted after a minimum age of 7 days of casting the pretensioned beam. The stirrups extended from the pretensioned beam to the top slab to provide the dowel action similar to the deck slab of the bridge as shown in Fig.4. The test beams were designed to have the same span-to-depth ratio of 17.8 and the same induced stresses in the section due to prestressing as the bridge girders. Due to lack of information in the literature on the performance of FRP as shear reinforcement, various stirrup sizes were used to study their effect on the shear and flexural behaviour.

Three of the beams were prestressed by five 15.2 mm CFCC seven wires cables and reinforced by three different sizes of double legged CFCC stirrups. The three stirrups sizes were 7.5 mm seven wire cables, 5 mm seven wire cables, and 5 mm solid cable. Two beams were prestressed by ten 8 mm Leadline bars and reinforced by two different configurations of Leadline stirrups, single and double legged stirrups. The Leadline stirrups had a rounded corner rectangular cross section of an equivalent area of 7 mm diameter bars. The sixth beam was prestressed by five 13 mm conventional steel seven wires strands and reinforced by steel double legged stirrups as a control specimen. The stirrups for all test beams were spaced uniformly every 110 mm within the I-shape portion of the beam. Typical configuration of the single and double legged stirrups are shown in Fig.5.

Since the scale factor is not applicable to the unit weight of the concrete, the resulting stresses at the top surface of the girder, due to the affect of prestressing and self weight, exceeded the allowable tensile stresses. Therefore, temporary external post-tensioning was used to reduce the tensile stresses at the top fibres, as shown in Fig.2 and Fig.4.

Testing Scheme

The testing program was conducted at the Structural Engineering and Construction R&D Facility, University of Manitoba, using the set up shown in Fig.6. Spreader beams were used to apply four concentrated loads to simulate an equivalent truck loading condition. Lateral support were provided at four locations along the span, as shown in Fig.7. A ± 1.2 million pounds MTS testing machine was used to apply the load statically using deflection
control mode. Deflection at mid-span was measured using linear motion transducers (LMT). Dial gauges were used to monitor any possible slip of the tendons or relative slip between the pretensioned beams and the top slab. Demec point stations of the "Rosette" type were used at the high shear stresses locations to measure the strain in three directions in the web. Other demec point stations were located at the mid-span zone to measure the strain distribution along the section. The two external steel strands were released before testing as soon as the machine was in full contact with the tested beams.

Test Results

This paper presents the test results of four beams. Three beams prestressed by 15.2 mm CFCC and reinforced by three different types of CFCC stirrups, 7.5 mm diameter seven wires cables, 5 mm diameter single wire and 5 mm diameter seven wires cables. The fourth beam was prestressed by 8 mm Leadline bars and reinforced by Leadline double legged stirrups. Fig.8 shows the predicted and observed load-deformation curves of the tested beams. All beams behaved linearly up to cracking with the same stiffness and after cracking with 69 percent reduction in the stiffness of the beams prestressed by CFCC and 66 percent reduction of the stiffness of the beam prestressed by Leadline up to failure. The three beams prestressed by CFCC with different stirrups sizes showed identical flexural capacity. The beam prestressed by Leadline failed at 13 percent higher load level in comparison to the beams prestressed by CFCC due to the higher tensile strength of the Leadline. No slip was observed for the prestressing cables and bars neither at the end of the beam nor between the girder and the top slab up to failure for all the tested beams.

Failure Mode

Two beams prestressed by CFCC failed by rupture of the lower draped cable at the location of steel pin, 400 mm outside the constant moment zone, followed by rupture of the three bottom straight cables and finally rupture of the top draped cable as shown in Fig.9a. The third beam failed by rupture of the bottom straight cables. Failure of the bottom draped cable, which was located 50 mm above the bottom straight cables, occurred first due to the high localized stress concentration induced at the pin location due to draping. Rupture of the cables was accompanied by horizontal cracks and spalling of the concrete cover along the tendons level due to release of the elastic strain energy after rupture of the tendons. These findings are consistent with the results of the testing conducted by Abdelrahman and Rizkalla (Abdelrahman, Tadros and Rizkalla 1993).

The beam prestressed by Leadline failed at higher load compared to the beams prestressed by CFCC. Failure occurred by shear at the maximum shear location, 2.6 meters from the support. Before failure, spalling of concrete cover was observed at the bend of the stirrups from the web to the bottom flange which suggests that stretching of the stirrups had initiated the failure of the beam. The brittle failure could be also attributed to rupture of the stirrups since the directions of the cracks were with an angle with respect to the directions of the stirrups. The reduction of the tensile strength of the stirrups with the presence of inclined cracks was discussed by many researchers (Maruyama, Honma and Okamura 1989). The measured concrete strain at the location of the Leadline stirrups at
failure, based on 50 mm gage length, was about 45% of the uniaxial ultimate tensile strength of the Leadline due to inclination of the cracks with respect to the direction of the fibres. The failure of the stirrups caused sudden transfer of the forces to the cracked concrete in the web and the prestressing bars, resulting in crushing of the concrete and rupture of the bars as shown in Fig.9b.

ANALYTICAL MODEL

Behaviour of the beams was analyzed for flexure and shear using computer program "RESPONSE" Version 1.0 (Collins and Mitchell 1991), developed to determine the load-deformation response of a prestressed concrete cross section subjected to any combination of moment, shear, and axial load. The program uses layer-by-layer approach and material characteristics of the reinforcement to determine the moment-curvature response for a given cross section. A parabolic variation of the concrete stresses was assumed for each layer. The program was originally developed to analyze beams prestressed by steel tendons using a modified Ramberg-Osgood function to define the steel stress-strain relationship. The constants, used in the program to identify the material properties, have been changed to represent the properties of CFCC and Leadline reinforcements. Since the test beams included draped strands and the program deals only with individual sections, each beam was divided into 47 sections to obtain the moment-curvature response for each section. Another computer program was written to perform the numerical integration of the curvatures along the span of the beam in order to compute the deflection at different load levels. The predicted load-deflection response for beams prestressed by CFCC and Leadline cables, shown in Fig.8, are in good agreement with the measured values including the cracking load and the stiffness before and after cracking. The predicted failure load of the beams prestressed by CFCC was about 5 percent less than the measured value. This could be due to the early failure due to rupture of the draped strands at the pin location. The predicted failure load of the beams prestressed by Leadline bars was also about 5 percent less than the measured value due to the premature failure in shear as described earlier.

The analytical technique used in the "RESPONSE" program to analyze the shear behaviour of the test beams implements the modified compression field theory (Collins and Mitchell 1991). The theory is based on applying the equilibrium conditions, the compatibility conditions, and the constitutive relationships for the materials linking the stresses and strains to determine the complete load-deformation response of members subjected to shear, moment, and axial load.

The concrete strains at the stirrups locations were measured during the tests using a mechanical demec gauge to evaluate the average strains in stirrups. The gauge length, which was 200 mm, included several cracks developed across the stirrup. The demec point stations were installed at the locations where the maximum stresses in stirrups were expected as shown in Fig.6. Fig.10 shows the measured average strain in the stirrups at stations 1, 3, and 5 versus the load level for the beam prestressed by Leadline bars and reinforced by double legged Leadline stirrups. The predicted average strain at stations 1
and 5 using program "RESPONSE", are also shown in the same figure. General conclusion will be made following completion of the testing of the remaining beams in this experimental program.

CONCLUSIONS

From the present study, the following conclusions can be drawn,

1. Draping of the tendons is feasible for CFRP tendons, however the residual stresses at the draping points could trigger the flexural failure.

2. Shear failure is very brittle and causes breaking the beam to several pieces due to the energy released from rupture of the longitudinal prestressing tendons.

3. No slip was observed of the prestressing reinforcement nor between the top slab and the girder. This suggests that the strength of the CFRP stirrups is adequate to transfer the stresses between the top slab and the beam by dowel action.

4. Presence of the diagonal cracks with an angle with respect to the stirrups reduced significantly the ultimate tensile strength capacity of the stirrups.

REFERENCES


Fig. 1 Typical pretensioned concrete girder

Fig. 2 Detailing of test beam prestressed by CFCC

Fig. 3 Jacking set-up
Fig. 4 Pretensioned beams before and after casting the top slab

Fig. 5 Configuration of CFRP stirrups

Fig. 6 Schematic of test set-up
Fig. 7 Test set-up

Fig. 8 Load-deflection of beams prestressed by CFCC and Leadline
Fig. 9a and b  Failure of beams prestressed by CFCC and Leadline, respectively

Fig.10  Average stirrups strain of beam prestressed by Leadline