THE USE OF CFRP IN PRECAST PRESTRESSED HIGH PERFORMANCE CONCRETE FOR INTEGRATED BRIDGE DECK SYSTEMS

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ABSTRACT

This paper discusses the potential use of carbon fiber reinforced polymer (CFRP) prestressing strands in steel-free precast concrete bridge elements, as an alternative solution to improve the service life of bridge superstructures. The research encompasses the use of CFRP draped prestressing strands in integrated precast bridge elements similar to those used recently in a major bridge deck reconstruction project in Montreal, where steel prestressing cables were used. The bridge elements consist of a series of stemmed panels with variable depth, using high performance concrete. The main purpose is to investigate the structural behavior of deck panels made of CFRP and to compare the results with similar panels reinforced with prestressing steel cables. The development of a field reliable anchorage system for the CFRP strands is also presented. The testing of full-scale specimens indicates the good performance of the prestressed steel free deck panel at the serviceability and the ultimate strength levels.

Keywords: CFRP Strands, Precast, Prestressed, High Performance Concrete

1. INTRODUCTION

In recent years, the accelerated deterioration of bridges and the cost of their repair and rehabilitation have become a major concern. Since chloride-induced steel corrosion is one of the major worldwide deterioration problems for steel reinforced concrete bridges, a steel-free concrete deck offers an attractive alternative to insure long-term durability. In this paper, a new technique for an integrated bridge deck system using fiber reinforced polymers (FRP) is presented. Longitudinal prestressing draped strands made of carbon fiber reinforced polymers (CFRP) were used as the main stem reinforcement along with glass fiber reinforced polymers (GFRP) for the deck slab and shear reinforcement.

The design of the integrated deck panel is based on the Canadian Highway Bridge Design Code[1] (CAN/CSA-S6-00) and the ISIS Canada design manual[2]. However, the anchorage of the CFRP strands to the stressing system is a major difficulty because of the sensitivity of the FRP to transverse stresses[3].

2. EXPERIMENTAL BRIDGE DECK PANNELS

2.1 BRIDGE DECK SYSTEM

An innovative continuous multi-span prestressed precast concrete bridge system has been developed for a major bridge deck reconstruction project recently completed in Montreal, Canada, and using two-direction post-tension cables on site to ensure continuity[4]. All reinforcements for that project were in steel.

For the current research, the same moulds were used to construct two experimental deck panels having the same geometry. Each panel consists of a 7” (180 mm) thick concrete deck slab with two integral stems of variable depth (Figs. 1 and 2).

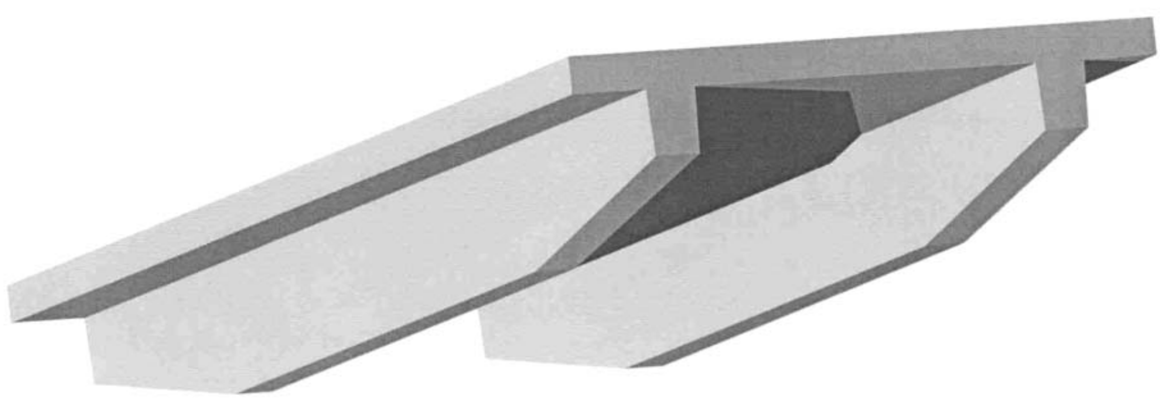

Figure 1. Deck Model

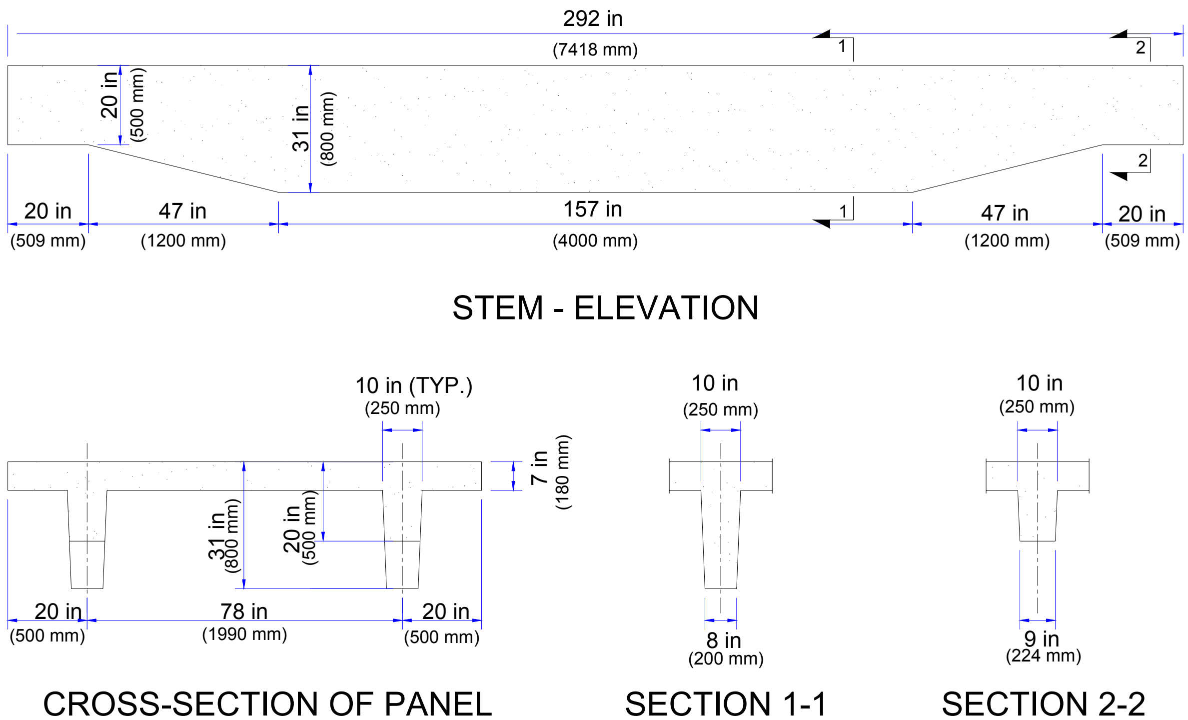

Figure 2. Typical Deck Panel

2.2 REFERENCE CONFIGURATIONS OF EXPERIMENTAL PANNELS

The reinforcement, particularly the prestressing, of the two experimental panels were designed to resist the tandem axle load of 73 kips (325 kN) prescribed in the CHBDC[1], at the service and ultimate load levels. The theoretical development to design for the flexural strength of concrete elements prestressed with CFRP strands is presented in Burke and Dolan[5]. High performance concrete 8700 psi (60 MPa) was used in the design of the two experimental deck panels.

The first panel was designed using only steel reinforcement for the prestressing cables, stirrups and slab flexural bars. Four 5/8” diameter draped prestressing cables and three deviators were used for each stem (Fig. 3) to resist the flexural moments. The required prestressing at transfer is 0.74 of the ultimate capacity of the steel cables. The reinforcement also includes 3/8” diameter stirrups in the stems for shear, and two beds of 5/8” diameter bars in each direction in the slab.

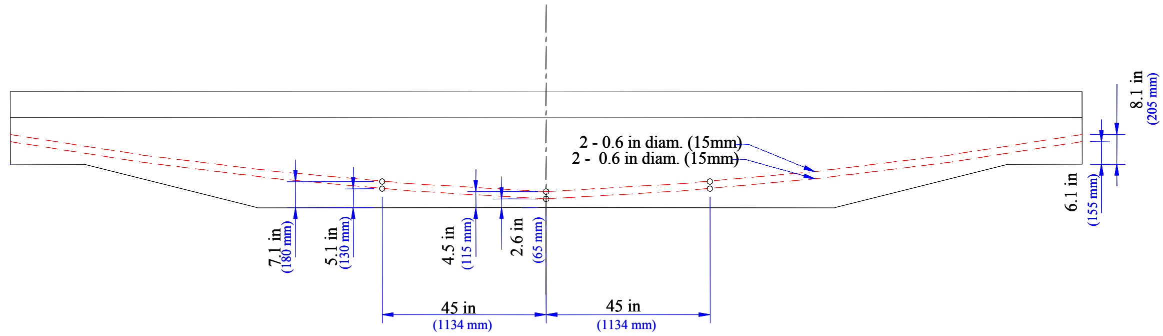

Figure 3. Prestressing Steel – Layout of Cables

The other panel was designed using no steel reinforcement whatsoever. Six prestressed 3/8" (10-mm) diameter draped indented Leadline CFRP strands, produced by the Mitsubishi Chemical Corporation of Japan, were used in each stem to resist the flexural moments. This panel also has three deviators as shown in Fig. 4, and the required prestressing at transfer is 0.55 of the specified capacity of the CFRP strands. The deviators for the four steel cables were modified to hold six CFRP strands (Fig. 5). In addition, the reinforcement of the steel free deck had 3/8" GFRP stirrups and 5/8" GFRP bars for the slab, both produced by Hughes Brothers Inc. distributed under the name of Aslan 100 GFRP. Each stirrup consists of two "C" shaped parts, having 3/8" (9.5 mm) diameter, with bents fabricated at the factory (Figs. 6 and 7). The slab reinforcement consists of 5/8" (15.9 mm) diameter straight bars.

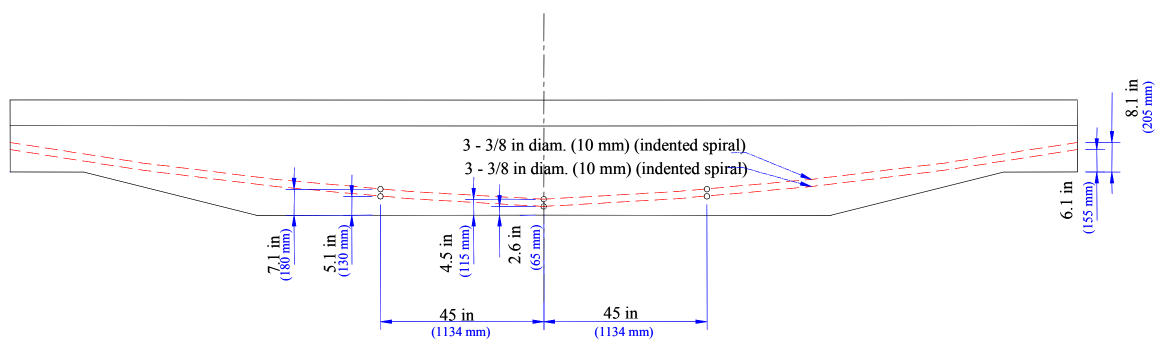

Figure 4. Layout of CFRP Leadline Strands

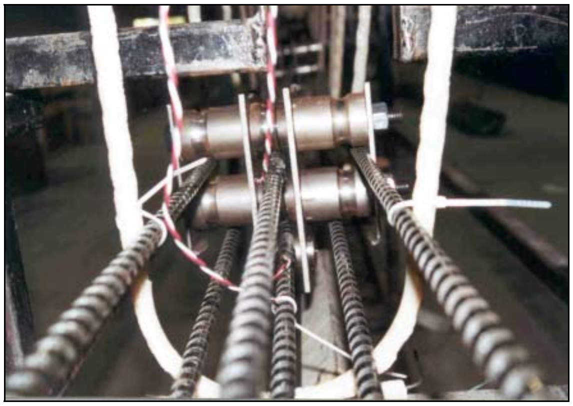

Figure 5. Hold-down deviators for CFRP strands

Figure 6. Stirrups (C-shape)

Figure 7. GFRP reinforcement configuration

3. CFRP STRAND TENSIONING SYSTEM

Although reliable systems are easily available for steel prestressing cables, the situation is different for CFRP strands[3]. Accordingly, the development of an easy-to-use field-reliable tensioning system is one of the main objectives of this research program.

To be able to use conventional field units to apply the prestressing forces, couplers for the steel cables were adapted to become steel/CFRP couplers. The adaptation consists only of replacing one threatened steel-anchorage end by an anchorage for the CFRP strands with the same thread. Since the steel/CFRP coupler is not cast in the concrete, the deck panel remains steel free. However, the choice and development a CFRP-anchorage that fits the standard 2” (50 mm) cable spacing is more challenging.

3.1 ANCHORAGE FOR CFRP STRANDS

The first system tested consists of a thick-walled steel tube in which the CFRP strand is bonded using an epoxy-gel adhesive (Fig. 8). The bond strength of the epoxy on the CFRP is increased by the indentation of the strand, which provides some mechanical load transfer. The inner diameter of the tube is reduced at the extremity to provide effective mechanical load transfer. This system was not sufficiently field-reliable because of the difficulty to completely fill the space without leaving voids. Such a bonded anchorage system might become more interesting for factory pre-assembling in a controlled environment.

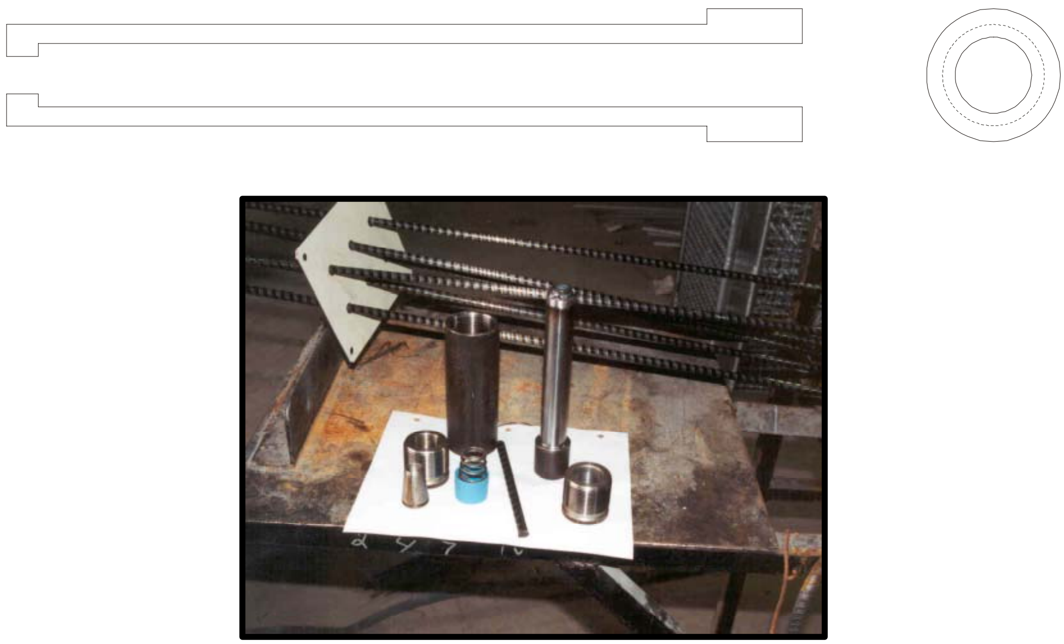

Figure 8. Bonded thick-walled steel tube anchorage

The second system tested was derived from the tapered sleeve/wedges system used for steel cables. It consists of an interior tapered steel sleeve with four aluminum-tapered wedges (Fig. 9). For about the half of the length of the sleeve the interior tapering angle is less than the angle of the wedges in order to reduce the transverse stresses on the strand at the extremity of the anchorage. Also, the wedges were fabricated in aluminum, a relatively soft metal, to minimize damage on the CFRP strand. In the laboratory tests, this system was able to transfer about 80% of the specified tension load capacity prior to the failure of the CFRP strand near the anchorage. In the field, this anchorage system was easy to install, but failed by slippage at 55% or less of the specified tension load capacity.

Figure 9. Steel sleeve/aluminum wedge anchorage

The third CFRP anchorage system tested was developed by Sayed-Ahmed and Shrive[6]. It consists of an interior low-angle-tapered steel sleeve, four low-angle-tapered steel wedges, and a thin inner copper sleeve (Fig. 10). The taper angle of the wedges is greater than the taper angle of the steel sleeve. Thus, upon insertion of the wedges into the sleeve, the wider end of the wedges form a contact before the narrower end in order to reduce the transverse stress on the CFRP strand at this critical section. The inner copper sleeve is composed of two parts placed in the interior channel of the wedges. The outer diameter of the inner sleeve matches the diameter of the wedge channel, while its interior diameter matches the CFRP strand. The strand is held by the sand-blasted surface of the inner copper sleeve, which is placed to diffuse the stresses.

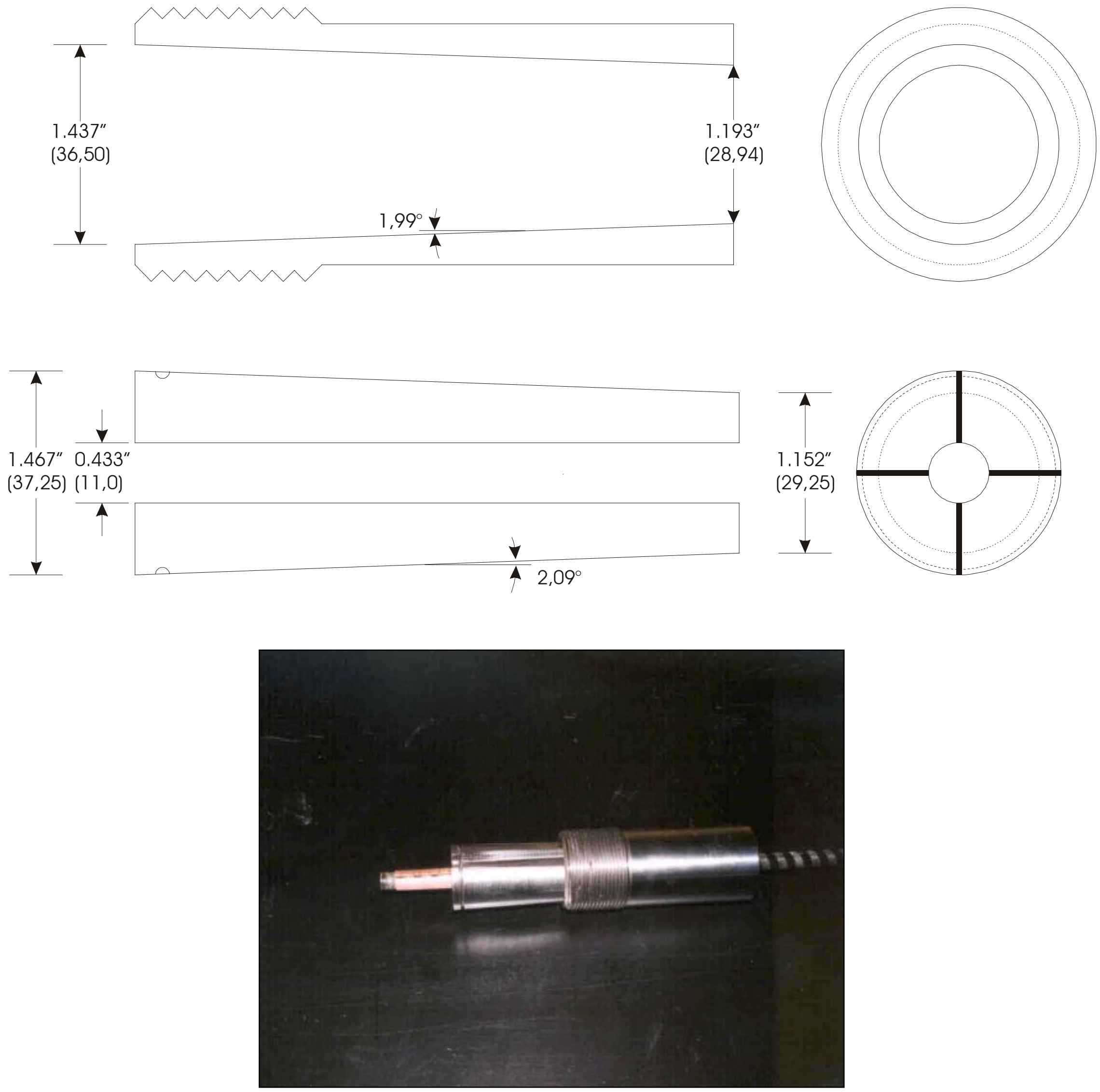

Figure 10. Low angle sleeve/wedge anchorage

The laboratory tests on the low-angle sleeve/wedge anchorage were successful as the full specified tension load capacity was obtained prior to the failure of the CFRP strand. Despite this success, this anchorage system occasionally failed by slippage at less than 55% of the specified tension load capacity during the prestressing of the test deck panel. The failure was partly attributed to the damage on the indentation of the CFRP strand due to the numerous tentative of stressing them, and partly to the difficulties to achieve equal stresses on the four wedges of the anchorage.

The last system tested is the same as the third one, except that it had only three wedges. In the laboratory tests, the full specified tension load capacity was obtained again, but this system exhibits less scatter in the results since such a three-piece system is self centering and self-equilibrates the force. This fourth anchorage system will be used for the future test specimens.

4. CONSTRUCTION OF EXPERIMENTAL PANELS

4.1 STEEL PANEL

The reference panel, with steel reinforcement only, was fabricated at the precast plant in the same way as all the deck panels for the Jacques-Cartier bridge replacement project, except for the strain gauges bonded on the cables, stirrups and slab reinforcing bars. The experienced fabrication team pointed out that the reference configuration did not account for bars to hold in place the bottoms of the stirrups. Thus, two 5/8" steel bars were added in each stem, which resulted in a substantial increase in the flexural strength of the deck panel. The as-built cross section at mid-span is shown on Fig. 11. The concrete strength at prestress transfer, at 17 hours, was 4800 psi (33 MPa).

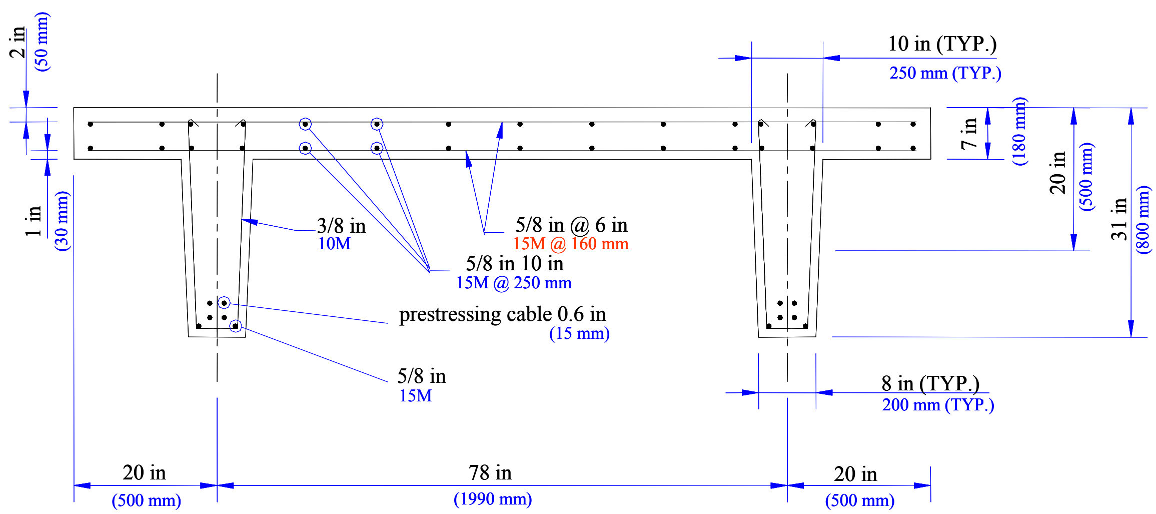

Figure 11. Steel panel – as- built mid-span cross section

4.2 FRP PANEL

The same problem of holding the stirrups in place occurred with the FRP panel. In this case, only one 5/8" GFRP bar was added above the mid-height of each stem. This solution was mostly devised to minimize the increase in flexural strength. Nevertheless, it was impractical to place the GFRP bars at the same positions as those for the steel bars because of the variable depth of the deck panel. All GFRP reinforcement was tied with tie-wraps to provide a steel-free deck for that panel. Strain gauges were also bonded on CFRP strands, GFRP stirrups, and slab GFRP reinforcing bars on this panel.

The difficulties with the anchorage system for the CFRP strands made it impossible to fabricate the FRP panel at the precast plant. The mould was transported to an outdoor site at the University of Sherbrooke. The third system of anchorages was used for the tensioning of the CFRP strands of the deck panel. It happened that the second strand to be tensioned slipped out of the anchorage when it reached the target load of 21.3 kips (95 kN) and, ruptured into multiple parts. It was decided to proceed with the others strands with caution; that is, to stop tensioning at lower levels than designed when a suspect noise was heard. Finally, two strands slipped-out of their anchorages without damage and stayed in place with no tension, two were removed because they ruptured when they slipped out, and the eight remaining had various tensions ranging from 35 to 55% of the specified capacity. The total prestressing in one stem was 62% of the reference configuration, while it was only 50% in the other stem, for the FRP deck panel. The as-built cross section at mid-span is shown on Fig. 12. The concrete strength at prestress transfer, at 8 days, was 3600 psi (25 MPa).

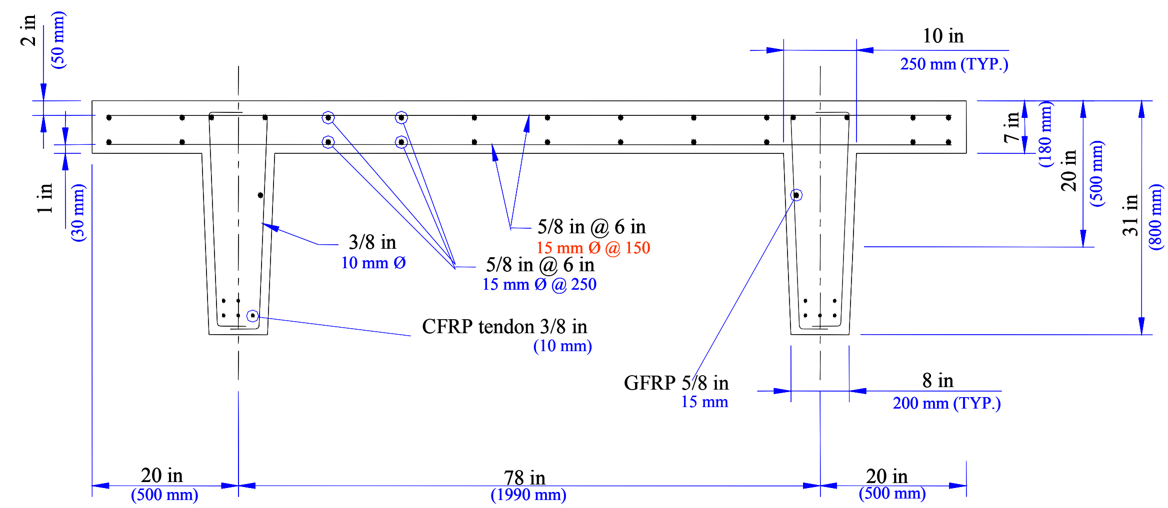

Figure 12. FRP panel – as-built mid-span cross section

5. TEST RESULTS

5.1 EXPERIMENTAL SET-UP

Because many strain gauges were destroyed or damaged during the prestressing process, it is virtually impossible to make comparisons of the two deck panels with strain gauge measurements. Thus, the experimental results presented here focus on the load–deflection behavior. Figure 13 shows the position of the applied load pattern, which is representative of a double axle of the design truck load CL-625 of the CSA S6-00 standard. Three linear variable differential transducers (LVDTs) were installed beside each stem to measure the deflection at mid-span and intermediate positions (Fig. 13). The movement of the four supports was also monitored with dial indicators giving a total of ten displacement readings.

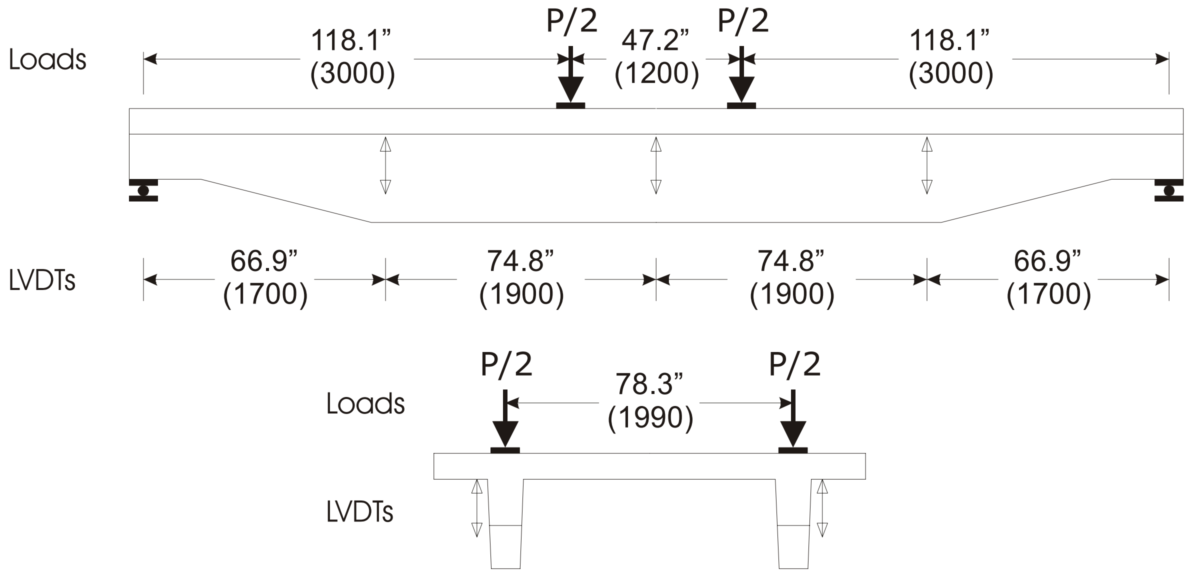

Figure 13. Positions of applied loads and measured displacements

A hydraulic loading set-up was specifically designed to test the panels (Fig. 14). It consists of two heavy steel beams on which the supports of the panel are placed. At one end the supports are fixed and, at the other end, longitudinal displacement is allowed to avoid the creation of undesirable axial stress in the panel. The supports allow for the rotation of the panel at both extremities. The heavy steel beam and the panel are inside the loading frame composed of two beams and two high strength bars. The bottom beam reacts on the heavy steel beam, while the top beam reacts on the division beams. A hydraulic jack is used to apply tension to the high strength bars. The two division beams are located over the stems to transfer the load to the panel. The system applies an equal force on four points located over the stem. Each stem supports two load points longitudinally spaced at 47” (1200 mm). The forces are recorded from the load cells installed on each of the two high strength bars used to apply the load to the panels.

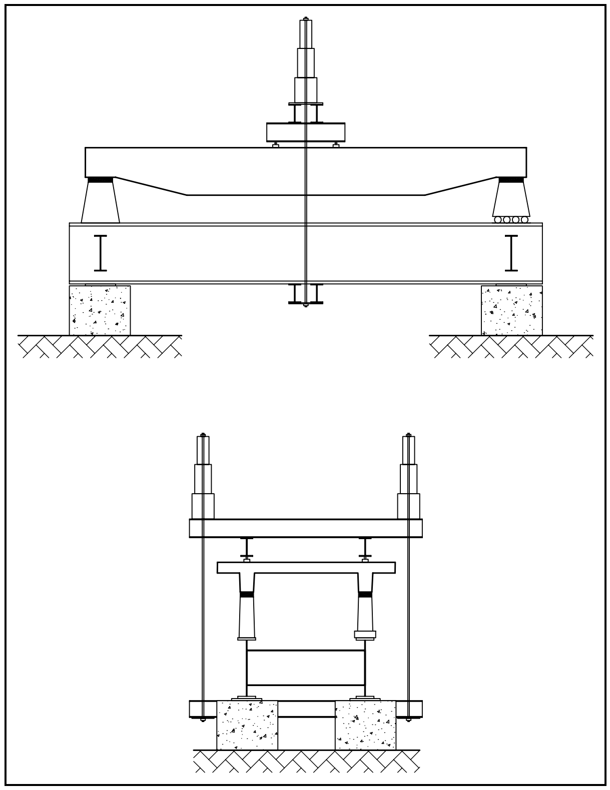

Figure 14. Loading set-up

5.2 BEHAVIOR UNTIL CRACKING

The first loading step consists of increasing the load up to the cracking of the stems of the deck panel in the vicinity of the loads, and then removing the load. This allows us to study the behavior within the service load range. Figure 15 shows the results for the two experimental deck panels along with the weight of the design load truck CL-625 of 73 kips (325 kN). The horizontal axis is the mean mid-span displacement of the two stems. The settling of the supports, measured with dial gauges, is accounted for and therefore the net displacements are presented. The vertical axis is the total load supported by the panel, the sum of the two load cells plus the weight of the loading frame. Since the loading frame, weighing 8.9 kips (40 kN), is supported by the panel, the graph begins at that load value.

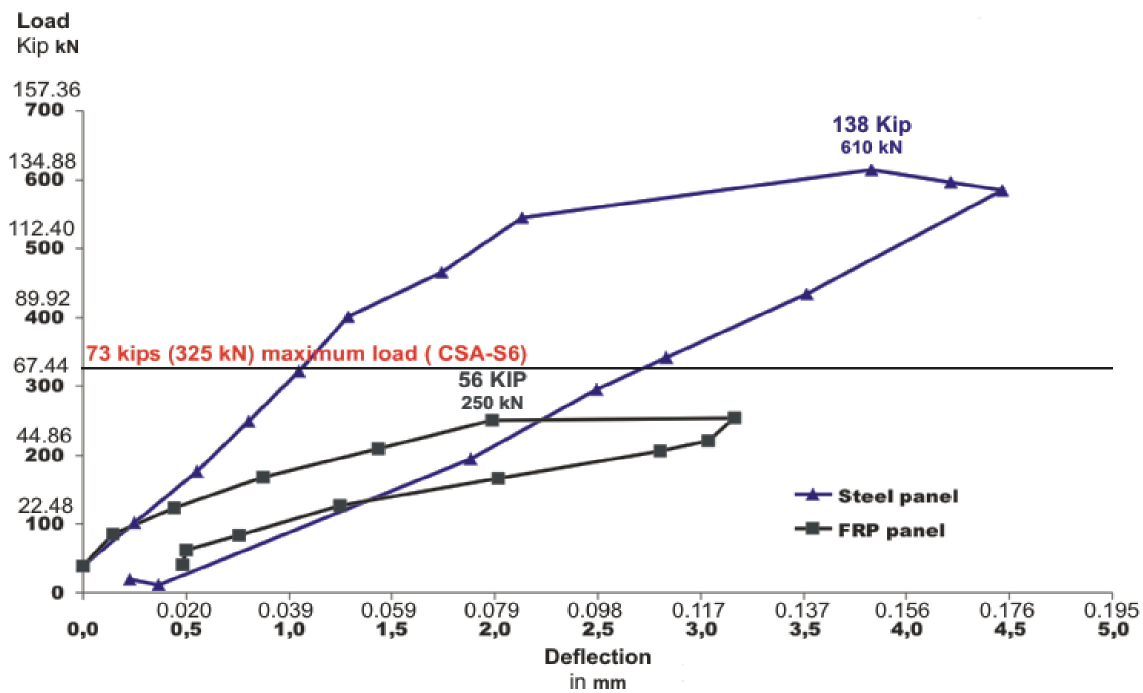

Figure 15. Results for service load range

On Fig. 15 the cracking of the panels is observed when a relatively small increase in the load produces a relatively large displacement. The cracking load for the steel deck panel of 135 kips (600 kN) is well above the design load, as it should be. On the other hand, the cracking load of the FRP deck panel was less than the design load. The lack of sufficient prestressing (56% of the expected level), due to the problem with the anchorage system for the CFRP strands, is largely responsible for this. The situation is also worsened by the low strength of the concrete. At the time of the tests, the concrete strength was 4300 psi (30 MPa) for the FRP panel and 10700 psi (74 MPa) for the steel panel. Had the prestressing and the concrete strength been in accordance with the design specifications for the FRP panel, the cracking load would have been in the range of 110 kips (500 kN). The difference in the concrete strength, and therefore in elastic modulus, is the main cause of the different uncracked stiffnesses of the two panels. When the load was removed, the displacements did not return exactly to zero, thus confirming the presence of damage and cracking. The measurements at intermediate positions show good symmetry and corroborate the same observations. The maximum deflection under service load prescribed in the CHBDC is ¼” (6 mm) thus, the panels meet this requirement easily.

5.3 BEHAVIOR UP TO FAILURE

For the second loading step, the panels are loaded to failure to observe the behavior at the ultimate state. The results for the mid-span deflection of the two experimental deck panels are shown on Fig. 16. The axes are the same as for the preceding figure. The steel panel exhibits large ductility and the loading process was stopped before fracture of the reinforcing steel. From these results the ultimate strength, corresponding to the yielding of the steel, is identified as being a load of 225 kips (1000 kN) and a displacement of 0.9” (23 mm). It is worthy to recall that the steel panel had passive flexural reinforcement, which provides 15% of the strength.

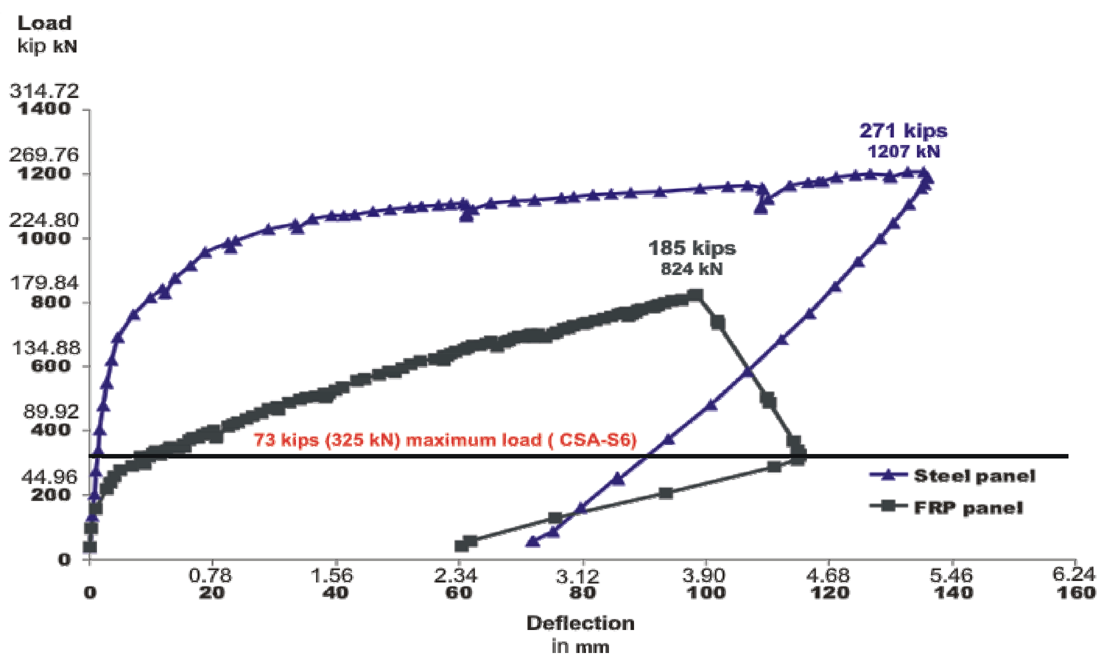

Figure 16. Results for ultimate load

The FRP deck panel exhibits a significant post cracking stiffness up to failure, instead of the usual plateau observed for steel reinforced concrete. Nevertheless, signs of distress, such as excessive deflections and large cracks, appeared well before actual failure and provided clear warnings of imminent failure, if not conventional “ductility”. The failure occurred by the rupture of all of the strands of only one stem. A single very wide crack, in the range of 1" (25 mm), at the mid-span section indicated that the deviator was the initiator of the strand rupture (Fig. 17). The reduction of CFRP strand tensile strength caused by deviators has been observed by other researchers[7]. Because the experimental set-up provides an equal load on both sides, as soon as one stem loose its load-carrying capacity, no additional load can be applied. For the FRP deck panel, failure was observed under a total load of 185 kips (825 kN) for a mid-span displacement of 3.9" (100 mm). Although it seems quite a bit less than that for the reference steel panel, it is in fact very close when the lost strands are accounted for. The lower than expected concrete strength of the FRP panel had little influence on the resistance of the deck panel because of the small depth of the neutral axis at the ultimate state. The measurements at the intermediate positions showed good symmetry up to failure.

Figure 17. Ruptured FRP Deck Panel Stem

Even though the reinforcements planned for the reference configuration of the panels have not been implemented in the built specimens, the experimental results indicate that the designed prestressing level is appropriate for the selected truck load.

6. CONCLUSIONS

This paper reports one phase of a research program aimed to demonstrate, through comparisons between steel prestressing reinforcement and FRP materials, the influence of the type of reinforcement on the serviceability, strength and mode of failure for bridge deck systems. Two full-scale bridge deck panels were fabricated and tested: one with steel reinforcement serving as a reference, and the other reinforced with CFRP prestressing strands and GFRP bars and stirrups. The results from the flexural loading test on the steel-free panel demonstrate that FRP reinforcement can provide the required load capacity.

Major difficulties were encountered in the development of a field-reliable anchor system for CFRP prestressing strand. Thus far, three systems have been tested; however, further research and development is required to address this aspect.

6.1 FOLLOW-UP WORK

During the summer of 2003, two other specimens will be built and tested. They will have only one stem, and the stirrups will be held only by the slab reinforcement to obtain fully prestressed sections. The latest anchorage system will thus be field tested. Subsequently, a numerical model will be developed and calibrated with all the experimental results according to their as-built characteristics.

7. ACKNOWLEDGEMENTS

This research project was funded by SNC-Lavalin Inc., and the ISIS Canada Network of Centres of Excellence. The support of Dr. Luc Lainey, Senior Vice-President and General Manager of SNC-Lavalin Inc. is greatly appreciated. KWN is Canada Research Chair in Advanced Engineered Material Systems and the support of this program is gratefully acknowledged.

8. REFERENCES

1. Canadian Highway Bridge Design Code CAN/CSA-S6-00, CSA International, 2000, Toronto, Ontario, Canada, 752 p.
2. ISIS Canada, "Reinforcing Concrete Structures with Fiber Reinforced Polymers", 2001, 153 p.
3. Reda Taha, M.M. and Shrive, N.G., "New Concrete Anchors for Carbon Fiber-Reinforced Polymer Post-Tensioning Tendons - Part 1: State-of-the-Art Review/Design," *ACI Structural Journal*, Vol. 100, No. 1, 2003, pp. 86-95.
4. Zaki, A.R. and Mailhot, G., "Deck Reconstruction of Jacques-Cartier Bridge Using Prefabricated HPC Panels", *The First fib Congress*, Osaka, Japan, October 2002.
5. Burke, C.R. and Dolan, C.W., "Flexural Design of Prestressed Concrete Beams Using FRP Tendons," *PCI Journal,* Vol. 46, No. 2, March/April 2001, pp. 76-87.
6. Sayed-Ahmed, E.Y. and Shrive, N.G., "A New Steel Anchorage System for Post-Tensioning Applications Using Carbon Fiber Reinforced Plastic Tendons," *Canadian Journal of Civil Engineering*, CSCE, Vol. 25, No. 1, 1998, pp.113-127.
7. Grace, N.F. and Abdel-Sayed G., "Behaviour of Externally Draped CRFP Tendons in Prestressed Concrete Bridges," *PCI Journal,* Vol. 43, No. 5, 1998, pp.88-101.