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STATIC LOAD BEHAVIOUR OF HYBRID FRP-CONCRETE TWO-PANEL TRUSS GIRDERS REINFORCED WITH DOUBLE-HEADED GFRP BARS

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ABSTRACT

The results of an experimental investigation into the behaviour of precast hybrid FRP-concrete, two-panel truss girders under static loading are presented. The investigation is part of a comprehensive research program on the development of a corrosion-free system for short and medium span bridges. The truss girders consist of pretensioned concrete chords connected by vertical and diagonal truss elements made of glass FRP tubes filled with concrete. The truss elements are reinforced and connected to the top and bottom chords by means of long double-headed steel or glass FRP bars. The top and bottom chords are also reinforced with glass FRP stirrups and double-headed longitudinal bars. This new system is expected to reduce the initial and maintenance costs and enhance the durability of bridges, and thus, extend their useful life. Moreover, by replacing the solid concrete web of traditional precast I-girders with the truss elements, the weight of the structure is significantly reduced, and thus, longer spans can be achieved and the number of supporting piers and size of the substructure can be reduced. The main objective of the research presented herein was to examine the efficiency of the FRP in comparison to steel reinforcement. The work included fabricating and testing three large-scale two-panel truss girders with an overall depth of 1.32 m and length of 2.83 m. The test parameters included the type and amount of reinforcement. The girders were tested under monotonic loading up to failure. The tests showed excellent results in terms of strength, stiffness, and connection performance of the truss girders.

Keywords: Bridge girder; concrete-filled FRP tubes; double-headed bars; glass FRP reinforcement; precast concrete.

1. INTRODUCTION

Corrosion of steel reinforcement combined with fatigue loading induced by traffic can cause significant deterioration in concrete bridges. Traditional materials, such as concrete and steel in bridge structures, have a limited service life and high maintenance cost, especially if they are used in a harsh environment such as that of Canada. In addition, producing huge amounts of concrete and steel needed for construction of traditional bridges leads to significant CO_2 emissions. Many rehabilitation methods designed to extend the service life of bridge structures that have deteriorated due to corrosion of the reinforcing steel have been developed and put into practice within the past few years, but none of these methods can be considered as a final solution for bridge deterioration (Firodiya et al. 2014).

The use of advanced composite materials, particularly fibre-reinforced polymers (FRPs), has provided a solution for the durability problem in existing structures. Because of the excellent characteristics of FRPs, such as resistance to corrosion, high strength, light weight, and ease of handling and installation, extensive research has been done over the past two decades on their use for strengthening and rehabilitation of structures. However, not as much effort has been made to develop new bridge systems that make effective use of FRPs, combined with conventional materials, as load-carrying structural components. Use of FRPs in such a hybrid form can significantly increase the structural strength, enhance the durability and sustainability, and extend the useful life to more than twice that of many existing bridges.

An innovative hybrid FRP-concrete bridge system has been developed for short and medium span bridges (El-Badry, 2007). The new system is composed of a cast-*in-situ* concrete deck slab on top of precast, prestressed concrete truss girders. The girders consist of pretensioned top and bottom concrete chords connected by vertical and diagonal truss elements made of FRP tubes filled with high-strength concrete. The pretensioned chords provide the flexural resistance, while the truss elements resist shear. Under gravity loads, the diagonal elements are predominantly in tension, and therefore, connected to the top and bottom chords by means of four long, double-headed FRP or steel bars. The bar heads have excellent anchorage properties and are used to reduce the embedment length of the straight bars and bear against the concrete when the bars are in tension (Ghali and Dilger, 1998). The vertical elements are mainly in compression, and therefore, connected to the chords by means of double-headed bars with shorter embedment length. The FRP tubes serve as permanent formwork and provide confinement to the concrete cores, and thus, increase the cores' compressive strength and protect them from the harsh environment. The new truss system is lighter in weight and more durable than the traditional I-shaped bridge girders. The reduced weight and enhanced durability lead to a reduction in the construction and maintenance costs. The light weight of the girder reduces the required amount of prestressing, allows for longer spans, and results in smaller sizes of substructure or fewer supporting piers in multi-span bridges. The enhanced durability can extend the structure's useful life to 100 years, which is double that of many existing bridges.

Performance of steel-reinforced truss girders of different span-to-depth ratios has been investigated by El-Badry et al. (2014). This paper reports the results of static loading tests on large-scale two-panel girder specimens reinforced with glass FRP stirrups and double-headed bars (Joulani, 2016). The two-panel specimens were selected in order to investigate performance of the truss elements and efficiency of the connections in shear-critical girders. A perspective view of a typical girder specimen is shown in Figure 1.

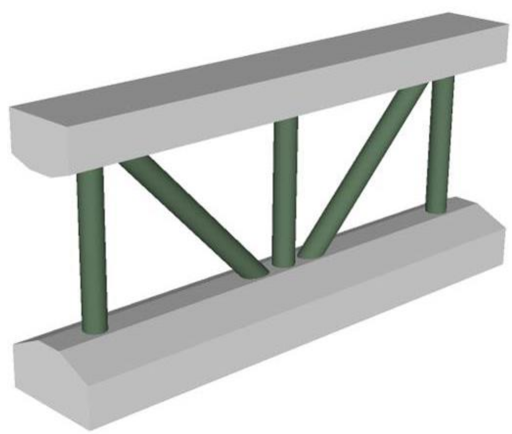


Figure 1: Perspective view of a two-panel hybrid FRP-concrete bridge girder specimen

2. EXPERIMENTAL PROGRAM

The experimental program included fabricating and testing three large-scale two-panel truss girders with an overall depth of 1.32 m and length of 2.83 m. The girders were tested under static loading increasing from zero to failure. The first two girders were reinforced with glass FRP bars with different bar diameters, 12 mm and 16 mm, of long double-headed bars, while the last girder was reinforced with steel bars for comparison. All the reinforcements, including stirrups, longitudinal bars, and double-headed bars for the first two girders, were made of GFRP bars to provide a corrosion-free truss girder. The dimensions and the amount of prestressing were the same in all girders. Several parameters, such as deformation of the truss girders, cracking pattern, strain in the concrete filled FRP tubes (CFFT), and strain in all reinforcements including longitudinal bars, stirrups and long double-headed bars, were measured.

2.1 Description of the Girder Specimens

The girders consisted of pre-tensioned top and bottom concrete chords connected by vertical and diagonal truss elements. The truss elements were made of glass FRP filled with concrete and connected to the top and bottom chords by means of long double-headed bars. Under gravitational loads, the diagonal elements were predominantly in tension and the vertical elements were mainly in compression. The truss elements served as permanent formwork and protected the concrete core, and thus, increased its compressive strength and shielded it from the harsh environment. All the reinforcements including stirrups, longitudinal bars, and double-headed bars for the first two girders were made of GFRP bars, while the last specimen had steel reinforcements. A bundle of two long, double-headed deformed bars were used to connect vertical tubes to the chords. Each diagonal element contained four long double-headed bars with different embedment lengths to provide gradual failure when they were in tension. The studs were placed in a pattern to provide adequate clear distance between the parallel bars such that the concrete could be placed and compacted satisfactorily for the development of an acceptable bond. The heads of the long double-headed bars were embedded in the concrete chords and bore against the concrete when the studs were in tension. The top cord consisted of four pre-tensioned 15 mm, low-relaxation, seven-wire steel strands stressed to 70% of the ultimate tensile strength load. The bottom cord contained seven strands to provide additional flexural capacity to the chords. The arrangement of these strands was kept constant for all girders.

All specimens had 2 panels and identical dimensions. The length of the girders was 2830 mm and the depth was 1320 mm. All the girders had a 250 mm overhang at the chords. The depth of the vertical truss elements was 800 mm and the diagonal elements were assigned to be at 45 degree with a length of 1132 mm. Figures 2a and 2b present the typical dimensions of a 2-panel truss girder.

Specimen designations had general format of "Gi-2F(S)-j"; where "Gi" refers to girder number, 2F or 2S denotes a 2-panel girder with either FRP or steel reinforcement, respectively, and "j" refers to diameter of the double-headed bars.

2.2 Reinforcements

Two different reinforcing methods, steel and GFRP reinforcements, were used for the longitudinal bars and stirrups in the chords and vertical tubes. Three different types of long headed bars were used in three specimens to connect the diagonal truss elements to the chords: 12 mm GFRP, 16 mm GFRP, and 12.7 mm steel bars. Each diagonal element contained four long, double-headed bars with different lengths, while the vertical elements included two bundle long double-headed bars of 1100 mm length. In the two corrosion-free GFRP girders, double-headed, deformed GFRP bars with a size of 16 mm where used as longitudinal bars. 12 mm GFRP stirrups were used as transverse reinforcement in the chords. For steel reinforcing girders, a deformed bar of size of 10M and 15M was used for all stirrups and longitudinal reinforcement, respectively. Long, double-headed steel bars with a 12.7 mm diameter stem were used to connect the truss elements to the chords in the steel specimen. The 40.2 mm heads were attached to the ends of the steel bars by a stud welding process. A minimum cover of 30 mm was used in both top and bottom chords in all specimens. A diagram of the truss girder reinforcement is given in Fig. 2, including the details of the vertical and diagonal truss elements (Fig. 2c and d), double-headed bars, embedment length, chord reinforcements, and prestressing details.

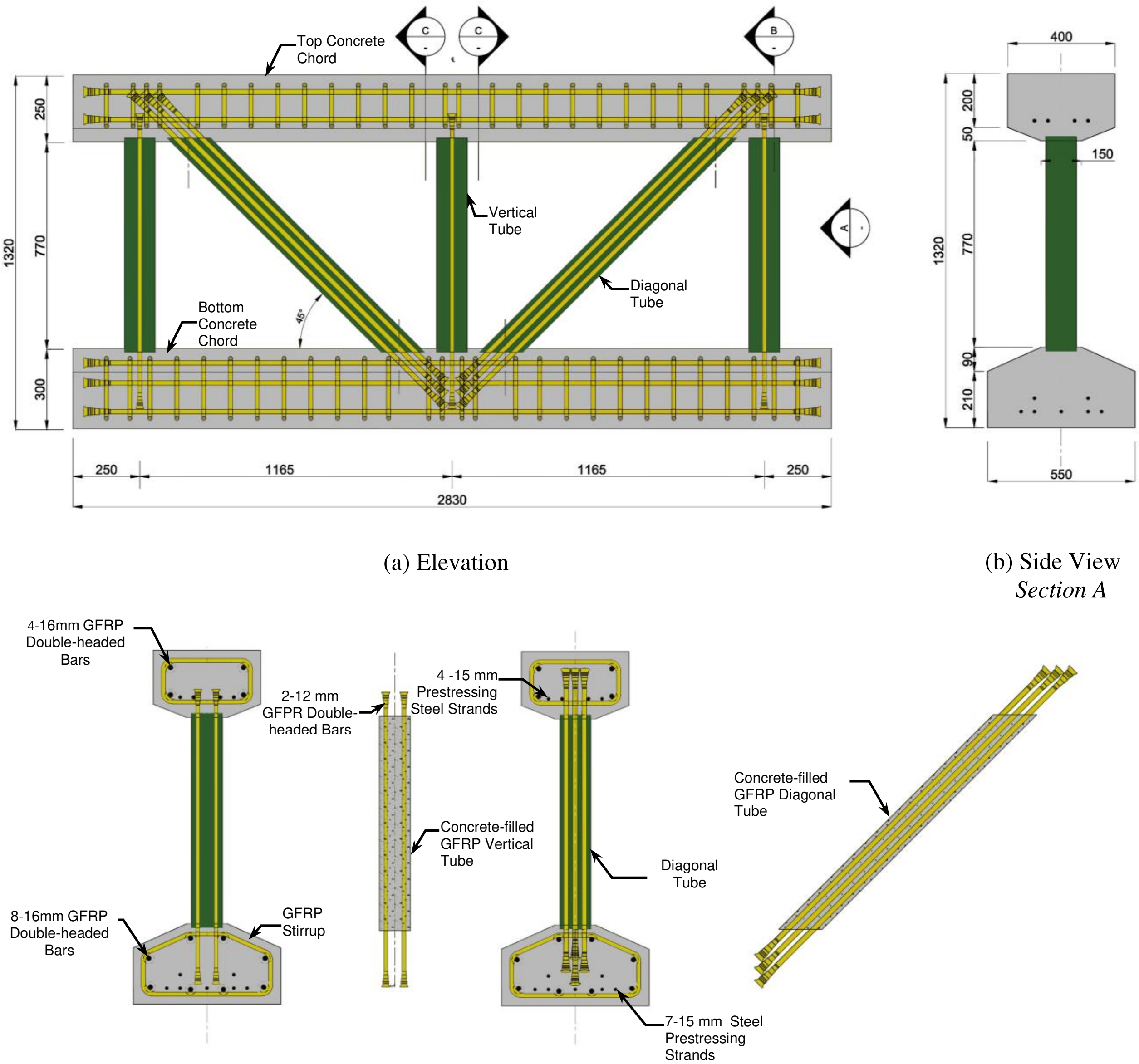
2.3 Material Properties

2.3.1 Concrete

The target 28-day compressive strength of the concrete was chosen to be 70 MPa for both the chords and truss elements. The strength of the concrete, at the time the prestressing was released, was specified at 35 MPa. A maximum coarse aggregate size of 10 mm was chosen to obtain a satisfactory consolidation of concrete around the headed-bars in the truss elements and near the heads. Standard type GU Portland cement was used in the concrete mix with a water to cement ratio of 0.26 and air entrainment around 8%. Washed and dried sand with a size of 5 mm was used as a fine aggregate. The average results of testing the 3 concrete cylinders for compressive and tensile strength are presented in Table 1.

Table 1: Concrete strength on the day of testing

Specimen	Compressive Strength (MPa)	Tensile Strength (MPa)
G1-2F12	69.0	5.2
G2-2F16	71.4	5.3
G3-2S12	71.3	5.2



(a) Elevation

(b) Side View
Section A

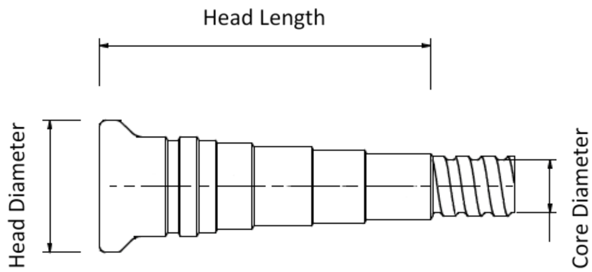
(c) Vertical Member
Section B

(d) Diagonal Member
Section C

Figure 2: Typical dimensions and reinforcing details of the truss girder specimens

2.3.2 Double-Headed GFRP Bars

A double-headed GFRP bar is a ribbed reinforcing bar made of corrosion resistant glass fibres, which are bound by a vinyl ester resin. FRP bars, in contrast to steel, are linearly elastic up to failure. The mean value of the short term tensile strength of the bars is about 1000 MPa. The material properties of the GFRP bars are presented in Table 2. The end heads are used to reduce the embedment length of the straight bars and bear against the concrete when the studs are in tension. The head geometry insured the load was distributed over the entire head length and provided the required tensile splitting forces at the head (Fig. 3a). The 16 mm double-headed GFRP bars were used as longitudinal bars in the top and bottom chords (Fig. 2a). GFRP stirrups were used as transverse reinforcement in the chords so that the corrosion-free specimens were reinforced only with GFRP bars, (Fig. 2a and 3b). GFRP bars cannot be permanently deformed or bent post production; therefore, stirrups must be ordered directly from the manufacturer. Two different sizes of GFRP stirrups were used for the top and bottom chords. The bending process considerably reduced the ultimate strength of GFRP bars to nearly 40% of straight bars. The 12 mm GFRP bars were used as internal reinforcement for the vertical CFFT elements. GFRP double-headed bars of 12 mm and 16 mm were also used to reinforced the diagonal CFFT elements and connect them to the concrete top and bottom chords (Fig. 2a and d).



(a) Anchoring Head of GFRP Bar

(b) GFRP Stirrups and Longitudinal Bars

Figure 3: GFRP reinforcement

Table 2: Material properties of GFRP bars

Type of reinforcement	Core Diameter (mm)	Cross Sectional Area (mm^2)	Ultimate Short term Tensile Strength (MPa)	Long-term Tensile Strength (MPa)	Module of Elasticity (GPa)	Design Value Long-term Tensile Strength (MPa)
ϕ16	16	201	> 1000	580	60	445
ϕ12	12	113	> 1000	580	60	445
Stirrups	11.6	106	> 700	250	55	190

2.3.3 Steel Reinforcement

Long, double-headed steel bars were used to connect the vertical and diagonal truss elements to the chords in the "G3-2S12" specimen. The double-headed bars had a deformed stem with 12.7 mm diameter to provide an adequate bond to the concrete. The 40 mm diameter head was attached to the end of the bars by a stud welding process to satisfy the ASTM A1044 specification, which required that the area of the head of the studs be at least 10 times the area of the stem. Also, standard grad 400 deformed bars of two different diameters were used as internal reinforcement in the top and bottom chords in the steel specimen. A size of 15M was used as longitudinal reinforcement in the top and bottom chords. A size of 10M was used as shear stirrup reinforcement in the concrete chords. Table 3 lists the tensile properties of all types of steel reinforcement used in the truss girders.

Table 3: Tensile properties of all types of steel reinforcement in the truss girders

Type of reinforcement	Core Diameter (mm)	Cross Sectional Area (mm^2)	Module of Elasticity (GPa)	Yield or Proof Strength (MPa)	Ultimate Strength (MPa)
Double-headed bar	12.7	127	184.0	503	620
10M Bar	11.1	100	201.8	455	645
15M Bar	15.9	200	196.6	430	580
7-wire Strand	15.2	140	192.5	1792	1943

2.3.4 Glass FRP Tubes

Glass FRP tubes were used as vertical and diagonal truss elements in all specimens. The tubes consisted of fibers with approximately 70% in circumferential and 30% in longitudinal directions. The tubes had inner and outer diameters of 110 mm and 114 mm, respectively. The mechanical properties of the tubes were provided by the manufacturer according to ASTM D2105-01 and D1599-99 and are listed in Table 4.

Table 4: Material properties of GFRP tubes

Tensile Strength (MPa)		Tensile Modulus of Elasticity (GPa)		Compressive Strength (MPa)	Compressive Modulus of Elasticity (GPa)	Ultimate internal pressure (MPa)
Long.	Circum.	Long.	Circum.	Long.	Long.	
240	480	20.6	29	240	20.6	13.8

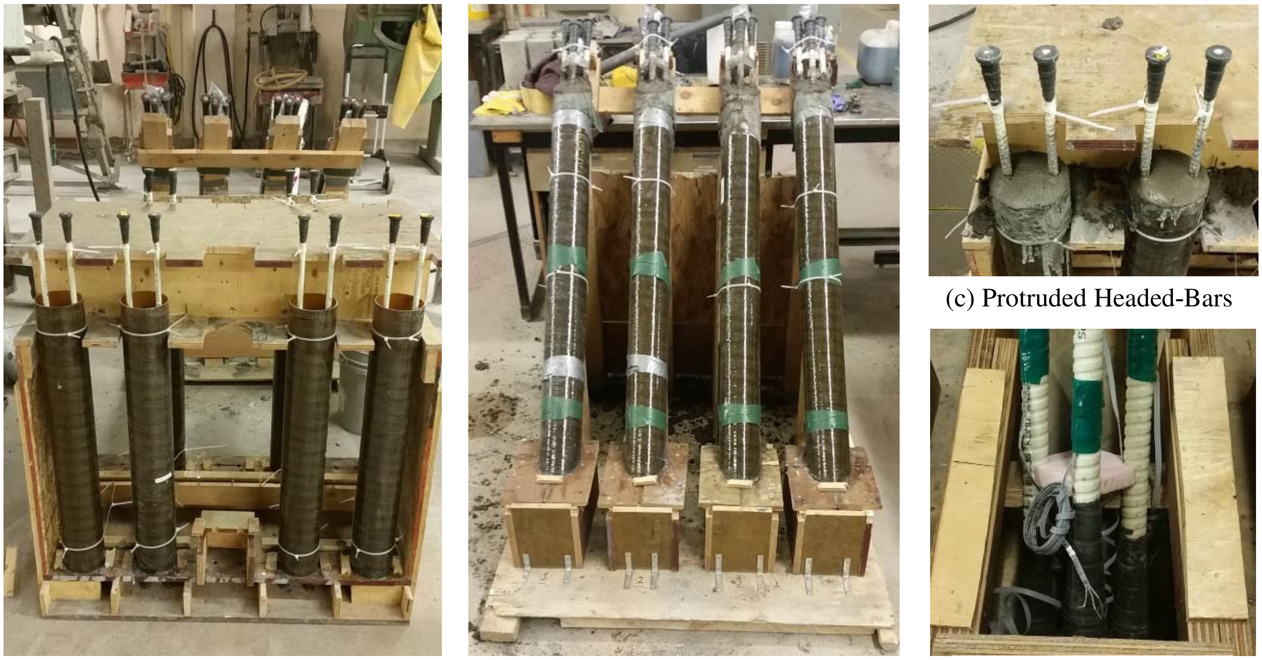
2.4 Fabrication of the Truss Girders

The girders were fabricated in the Structural Laboratory in the Civil Engineering Department at the University of Calgary. All girders were cast in two separate phases. In the first phase, the vertical and diagonal truss elements were cast by placing the FRP tubes in special wooden forms, which were fabricated in such a way that all double-headed bars and the GFRP tubes were fixed in place to ensure that the double-headed-bar arrangement stay accurate in all tubes. Figure 4 shows the arrangement of the double-headed FRP bars and the GFRP tubes in the wooden forms. The heads protruding from the tube ends provide adequate embedment length to connect the tubes and the chords (Fig. 4c). On each of the diagonal GFPR bars, at the interface of the elements and the chords, a strain gauge was placed to measure the related strain (Fig. 4d). The tubes were filled with concrete and were left for two days to sufficiently harden before placing them in the main formwork. In the second phase, the precast truss elements were placed along with the FRP or steel cage and the prestressing strands. For easier fabrication, the girders were cast in a horizontal position. Removable wooden pieces were used for fixing the truss elements along the main form. The prestressing steel strands were passed through the top and bottom chords, and they were prestressed and anchored to the prestressing bulkheads at the ends. Figure 5a shows the girder formwork just before the casting. The chords were cast and cured under the plastic sheet (Fig. 5c). The wooden forms were stripped, and the prestressing strands were released after the concrete reached a compressive strength of 35 MPa. Figure 5d shows the lifted truss girder in a vertical position after stripping the wooden form.

2.5 Test Setup and Instrumentation

Each girder was tested under static monotonic loading up to the failure point. Girders were tested while they were simply supported on two rollers. The 1.3 MN capacity hydraulic actuator was used to apply a static load in a displacement-controlled mode. One lateral support was placed at the location of the applied load and two lateral supports were placed near two supports to restrain any out of plane displacement that may occur with the top chords. The bottom chord was laterally supported only at mid-span. Figure 6a depicts a schematic view of the test setup of a 2-panel truss girder. Figure 6b shows the actual 2-panel truss girder during the static loading test.

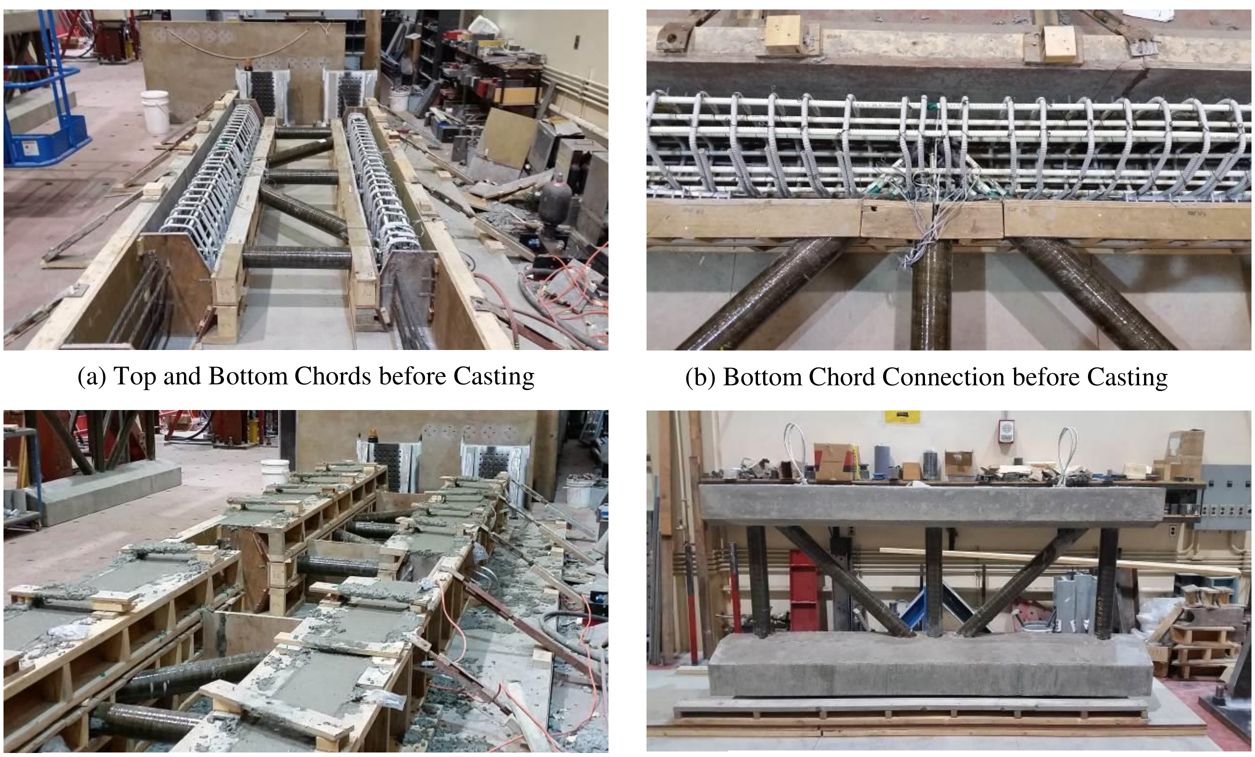
Three laser transducers were used, one underneath the bottom cord at mid-span and two at the centers of each panel, to measure deflection. Seven mechanical transducers were used to measure the shortening of the three vertical truss elements and the elongation in the two diagonal truss elements. Sixteen strain gauges were installed at the surface of all double-headed bars in the diagonal truss elements at the interface of the elements and the top and bottom chord. Strain in the bottom longitudinal bars in the top and the bottom chords were outfitted at the mid-span with strain gauges. Further, strain gauges were placed at the exterior surface of FRP tubes to measure the longitudinal and the circumferential strains in the vertical truss elements.



(c) Protruded Headed-Bars

(a) Vertical Tubes before Casting (b) Diagonal CFFTs after Casting (d) Strain-Gauged Bars

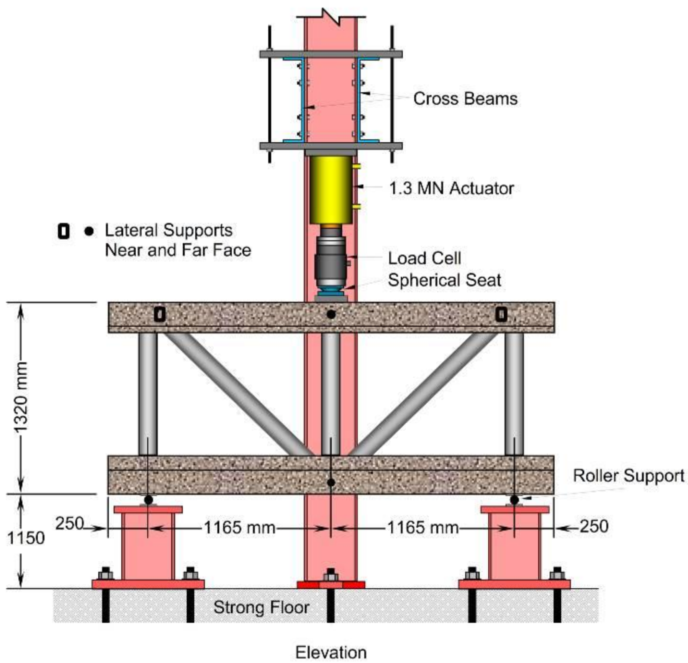
Figure 4: Truss elements before and after casting



(a) Top and Bottom Chords before Casting (b) Bottom Chord Connection before Casting

(c) Top and Bottom Chords after Casting (d) Truss Girder after Casting

Figure 5: The two-panel girder before and after casting



(a) Schematic View of Test Setup

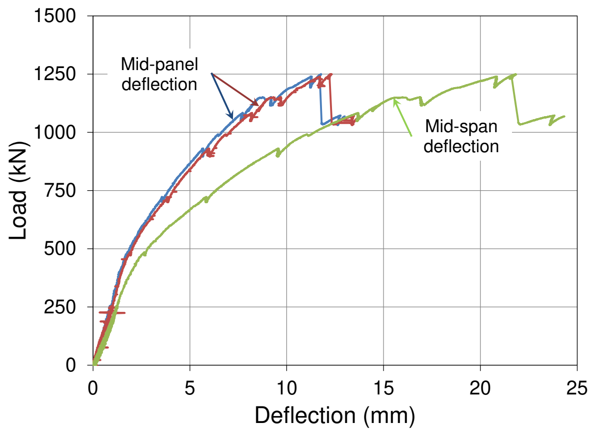
(b) Truss Girder during the Test

Figure 6: Test setup

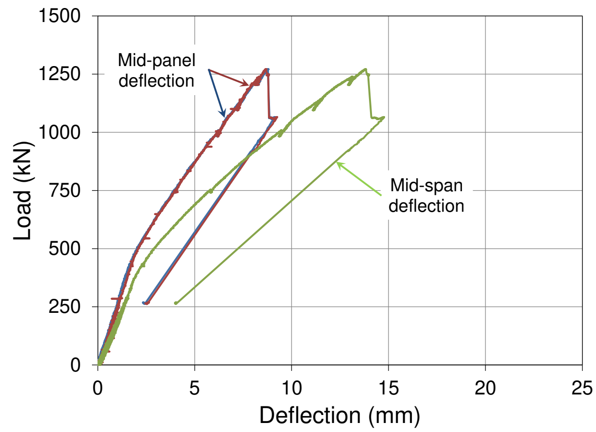
3. TEST RESULT AND DISCUSSION

All girders were tested under the mid-span static point load at a rate of 1 mm/min in a displacement-controlled mode. The static load-deflection response of the girders at the three locations of the laser transducers at the mid-span of the girder and at mid-length of each panel are shown in Figures 7a-7c. Figure 7d also compares the load-deflection behavior of three girders; the load-deflection behaviour indicates that the girders reinforced with GFRP double-headed bars showed a more ductile behaviour than the girder reinforced with double-headed steel bars in the service load zone due to a lower modulus of elasticity of GFRP (60 GPa to 200 GPa). A sudden drop in the load of the first two girders, G1-2F12 and G2-2F16, was due to the crushing of concrete in compression in the top chord at 1249 KN and 1271 KN, respectively (Fig. 8a). However, the overall ultimate load capacity of the girders was governed by the compression of the concrete in the middle CFFT element followed by the rupture of the FRP tube (Fig. 8b). Figure 9 shows the longitudinal and circumferential strain in the middle vertical CFFT element for all girders. The circumferential strain gauge value shows the transverse expansion of the tube due to the compression of the concrete in GFRP tube; however, the longitudinal strain gauges were used to measure the possible bending in the tube. The results show the effect of the bending and the transverse expansion of the concrete in the tube during the rupture of the CFFT tube at high loads.

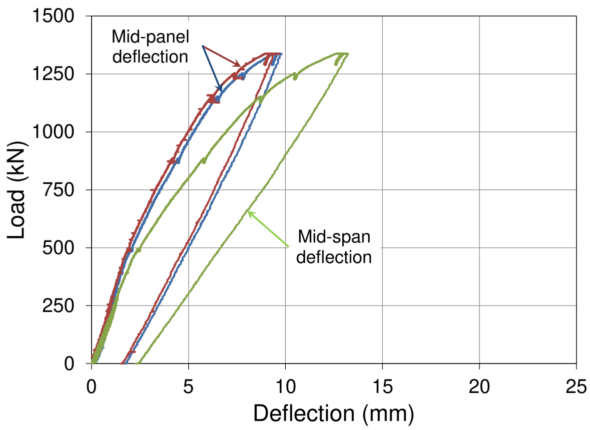
In the girder with steel reinforcement, the actuator reached its maximum capacity of 1338 KN during the test without showing a complete failure. All the double-headed bars in the diagonal elements and the longitudinal flexural bars in the chords yielded at this point, but no significant damage occurred to the girder. However, the initial sign of a rupture in the middle CFFT element was observed at a maximum load of the actuator. The circumferential strain measured in the middle vertical CFFT element was close to the value measured for the rupture of the GFRP tube in the first two girders (Fig. 9). This strain dramatically increased after yielding all the double-headed bars in the diagonal elements indicating that the overall capacity of the truss girder became dependant on the structural properties of the CFFT elements after yielding happened.



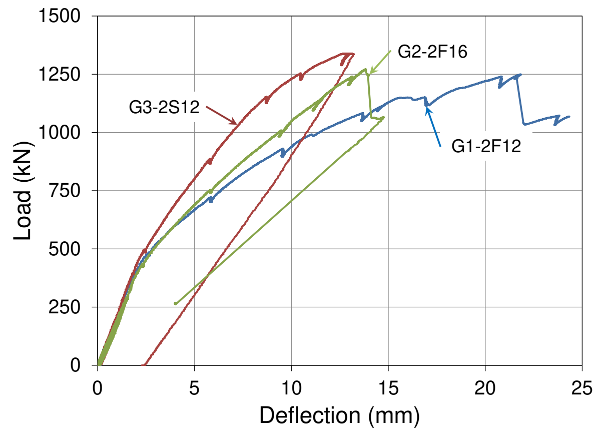
(a) G1-2F12



(b) G2-2F16



(c) G3-2S12



(d) Load-deflection comparison

Figure 7: Load-deflection response

(a) Concrete crushing at top chord

(b) Rupture of middle CFFT element

(c) Concrete crushing at bottom chord

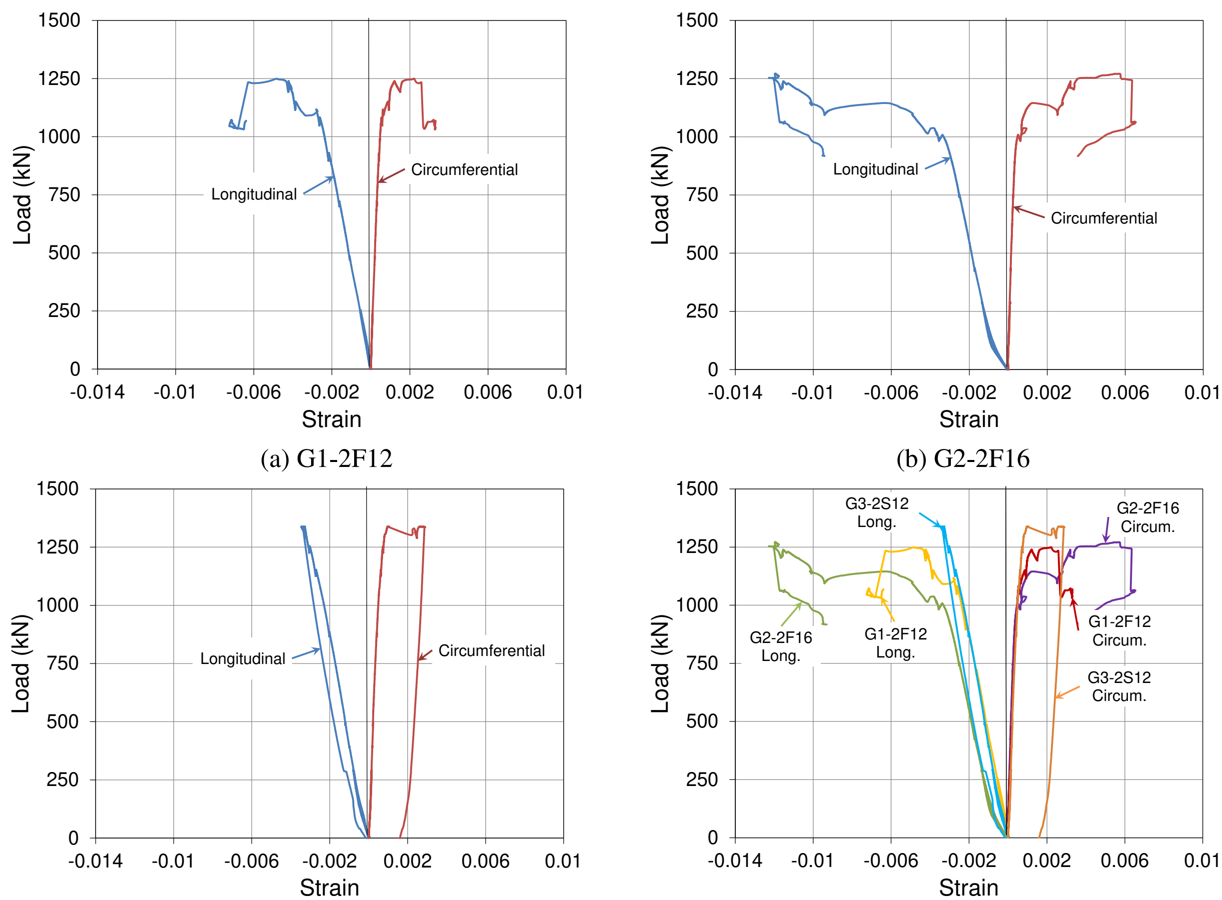
Figure 8: Failure of the girders

The first crack in the concrete chords occurred in the tension side of the top chord for all girders. The cracking load, P_{cr}, the maximum and minimum strains in the double-headed bars at ultimate load, ε_{max} and ε_{min}, respectively, and the girder ultimate load, P_u, are listed in Table 5.

Table 5: Critical loads and strains for static testing of the truss girder

Girder Designation	First Cracking Load, P_{cr} (kN)	Max Strain in Headed Bars at Ultimate, ε_{max} (%)	Min Strain in Headed bars at Ultimate, ε_{min} (%)	Ultimate load, P_u (kN)
G1-2F12	482	1.39	0.80	1249.3
G2-2F16	435	0.75	0.35	1271.1
G3-2S12	395	2.36	0.59	1338.7*

*Actuator reached its maximum capacity.



(a) G1-2F12

(b) G2-2F16

(c) G3-2S12

(d) Load-strain comparison in the middle CFFT element

Figure 9: Load-strain in the middle CFFT element

4. SUMMARY AND CONCLUSIONS

A novel, corrosion-free, hybrid FRP-concrete truss girder has been developed. The web elements of the truss girders are made of concrete-filled GFRP tubes connected to prestressed concrete top and bottom chords by means of double-headed GFPR bars. Using GFRP as a corrosion-free material for structural components and internal reinforcements combined with a novel design provides a solution for the durability problem. Further, the new structure will be lighter and more economical. The results of the experiment showed excellent performance of the truss girders in terms of strength, stiffness, and connection performance. The overall load capacity of the truss girder was governed primarily by the behaviour of the long double-headed bars in the diagonal truss elements. However, the ultimate failure happened due to the rupture of the middle CFFT element following the crushing of the concrete in the top chord. No significant damage happened in the connection of the diagonal CFFT elements to the chords indicating the great performance of heads and an adequate embedment length of the long double-headed bars in the concrete chords.

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