STATIC AND FATIGUE PERFORMANCE OF HYBRID PRECAST PRESTRESSED CONCRETE BRIDGE TRUSS GIRDERS

Mamdouh El-Badry, PhD, PEng, FCSCE, Department of Civil Engineering, University of Calgary, Alberta, Canada
Mohammad Moravvej, MSc, Department of Civil Engineering, University of Calgary, Alberta, Canada
Mohammad Aghahassani, MSc, Department of Civil Engineering, University of Calgary, Alberta, Canada

ABSTRACT

An innovative system for short and medium-span bridges comprising transversely spaced hybrid precast prestressed concrete truss girders and a cast-in-situ or precast concrete deck slab has been developed. The girders have top and bottom concrete chords connected by vertical compression and diagonal tension members made of concrete-filled fiber reinforced polymer (FRP) tubes. The vertical compression members are connected to the chords by means of dowels protruding from the ends. The diagonal tension members are connected to the chords using double-headed bars. The chords are pretensioned during fabrication. The girders may be post-tensioned by external tendons after erection to balance the slab weight and to provide continuity in multi-span bridges. All reinforcement and prestressing can be made of corrosion-resistant steel or glass and carbon FRP. The proposed truss girder is thus light in weight with enhanced durability. The light weight allows for longer spans and reduces the initial cost. The enhanced durability reduces the maintenance cost and leads to longer life span. An experimental evaluation of performance of large-scale truss girders under static and fatigue loading is presented. A total of ten girder specimens were fabricated and tested. All girders had identical cross-section dimensions with 1.2 m (4 ft) overall depth. Four of the girders consisted of 2, 4, 6, and 8 truss panels, respectively, with span lengths varying from 2.30 to 9.25 m (7.7 to 30.8 ft), and were tested under monotonic loading up to failure. The remaining six girders consisted of two truss panels each and were tested under cyclic fatigue loading of different level and amplitude. The post-fatigue resistance was also investigated. The tests showed excellent performance of the truss girders in terms of strength, stiffness, and fatigue life.

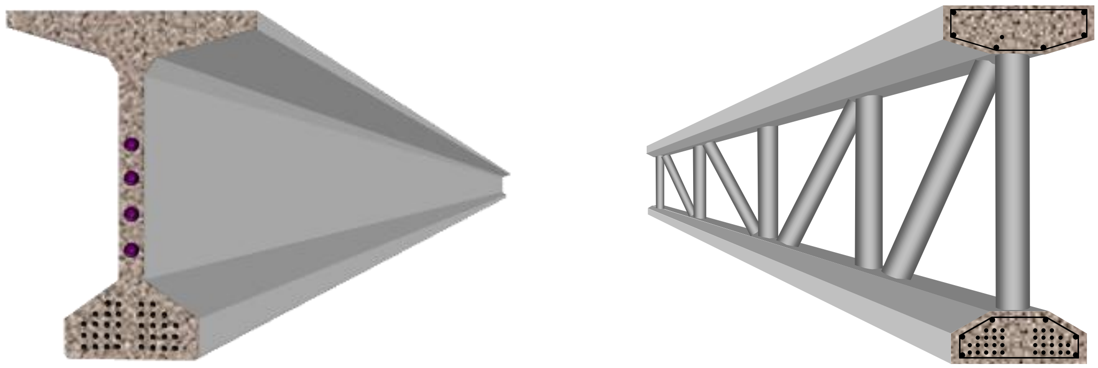
Keywords: Bridges, Fatigue, Fiber reinforced polymers (FRP), Headed bars, Hybrid girder, Precast concrete truss girder.

INTRODUCTION

Short and medium span bridges are commonly built of a concrete slab cast in place on top of steel or precast prestressed concrete I- or U-shaped girders. Figure 1a shows a perspective view of a standard precast concrete I-girder. While these girders can be economical and versatile, they do have some limitations and shortcomings. Their heavy weight limits the span length, requires large amount of prestressing, and presents a challenge for transportation and erection. Pre-tensioning the bottom flange produces initial camber at erection. The camber increases gradually with time due to creep of concrete. The eccentric pre-tensioning force in the bottom flange can produce tensile stresses of magnitude sufficient to cause cracking at the top of the end sections of the girder. The time-dependent effects of creep and shrinkage of concrete and relaxation of prestressed steel can be significant. The girder web thickness can be too small to accommodate continuity post-tensioning. In deep I-girders, thin webs can lead to slenderness causing stability problems during handling and erection. Furthermore, the internal post-tensioning ducts reduce the shear resistance of the already thin web.

Bridge engineers are continually challenged to design structures with longer spans, low initial cost, and enhanced durability and service life performance. These challenges can be met through utilization of new material technologies. Use of advanced composite materials, particularly fiber reinforced polymers (FRP), has emerged as a promising solution for durability problems caused, for example, by corrosion of steel and harsh environment. The excellent characteristics of FRPs, which include corrosion resistance, high strength, light weight, and easy handling and installation, have motivated extensive research and significant increase in their application in recent years, particularly for strengthening and rehabilitation of aging infrastructure and as replacement of steel reinforcement in new construction. However, not as much effort has been made to develop new bridge systems that effectively make use of FRP structural components as load carrying elements. New structures can be cost effective when the design concepts lead to reduced amounts of materials, lighter weight, and simplified and accelerated fabrication and construction procedures. However, the low rigidity of FRPs can limit their use in new bridges if all the structural components are made only of FRP. This limitation can be overcome when FRPs are used in combination with other conventional construction materials such steel and concrete in a hybrid form.

An innovative corrosion-resistant system for short- and medium-span bridges has been recently developed[1]. The system consists of precast prestressed concrete truss girders and cast-*in-situ* or precast concrete deck. The truss girders consist of top and bottom concrete chords connected by vertical and diagonal members made of hollow glass FRP tubes filled with concrete. A perspective view of a typical truss girder is shown in Fig. 1b. The bottom chord is pre-tensioned with prestressing strands and, along with the top chord, provides the flexural capacity of the girder. The truss web members resist the shear forces. All reinforcement can be made of FRP, stainless steel or any other type of corrosion-resistant material. Advantages of the new system include reduced self-weight and enhanced durability. The light weight reduces the required amount of prestressing and the load on the supports or allows for longer spans, resulting in smaller size of substructure or number of supporting piers in multi-span bridges and, hence, reduction of the initial cost. The potential enhancement in durability reduces the maintenance cost and can extend the useful life of the structure to 100 years instead of the 50 years for which many of the existing bridges were designed but failed to achieve.



(a) Standard Precast Concrete I-Girder (b) Hybrid FRP-Concrete Truss Girder

Fig. 1 Perspective view of the standard I-girder and the new hybrid FRP-concrete truss girder

Description of the truss girder is given in detail in the following section. An experimental program for performance evaluation of large-scale truss girders under static and fatigue loading is then presented.

DESCRIPTION AND CHARACTERISTICS OF THE TRUSS GIRDER

The bridge system is composed of a concrete deck slab cast-*in-situ* or made of precast panels placed on top of truss girders spanning in the longitudinal direction and spaced in the transverse direction. Each girder consists of top and bottom concrete chords connected by precast vertical and diagonal truss web members (Fig. 2a). The bottom chord is pre-tensioned with prestressing strands and, along with the top chord, provides the flexural resistance of the girder. The chords are reinforced with stirrups and non-prestressed longitudinal bars at the stirrup corners (Figs. 2b and c). The web members are arranged in N-shaped panels and, along with the top and bottom chords, form a Pratt truss in which the verticals are predominantly in compression and the diagonals are mainly in tension to provide the shear capacity of the girder. The truss members are made of concrete-filled FRP tubes (CFFT) produced prior to the chords. The FRP tubes serve as stay-in-place formwork, confine concrete in the compression members, provide reinforcement to the tension members, and protect the concrete from the environment.

Double-headed reinforcement (straight bars with anchor heads at their ends) connects the diagonals to the chords (Figs. 2a and b). The vertical members are connected to the chords by means of dowels protruding from the ends of the verticals (Figs. 2a and c). For a bridge under moving load, the vertical members may carry tension and the diagonals may be subjected to compression. In the design of such a bridge, double-headed bars should be used to reinforce all the truss members and connect them to the top and bottom chords, and the diagonal CFFT should be designed to carry the maximum compression anticipated from the moving load.

Double-headed studs are used in the girder's top chord to connect the deck slab (Fig. 2b). For ease of production, it is advantageous to cast the chords in a rotated position, while the verticals and the diagonals lie in a horizontal plane.

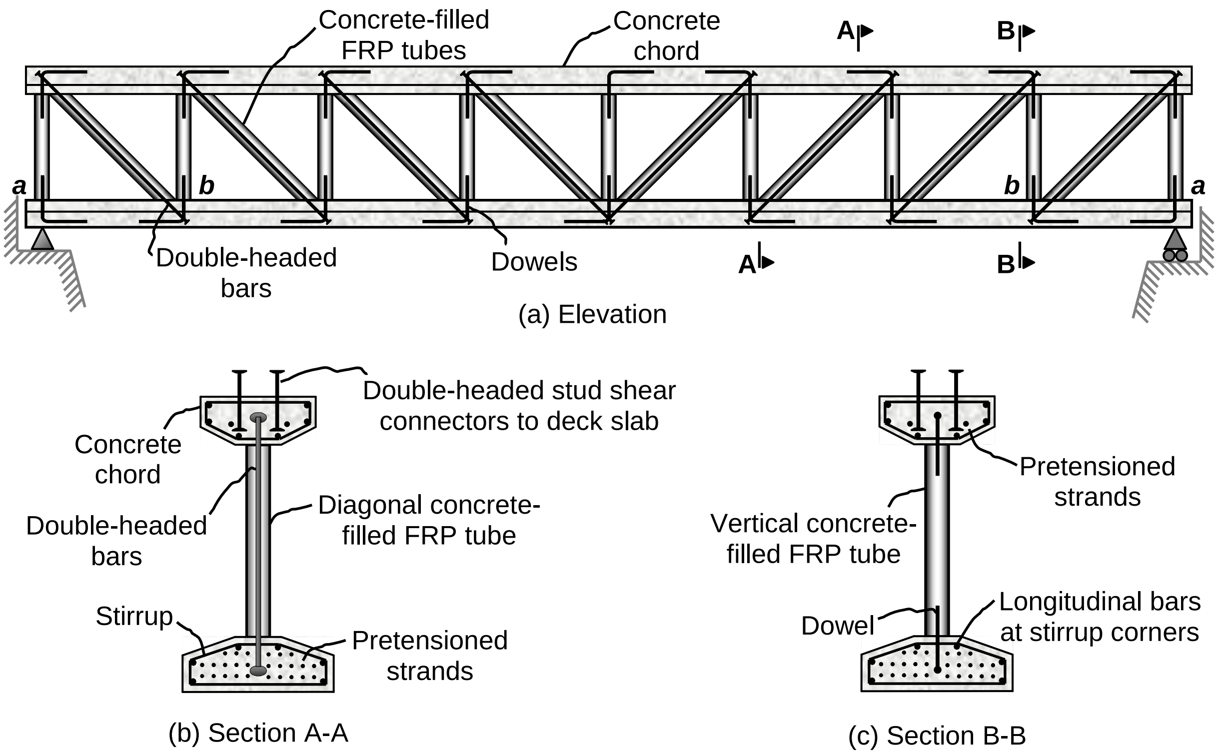


Fig. 2 Details of the hybrid FRP-concrete truss girder

It should be noted that, for this hybrid girder, the Pratt truss configuration is more preferred over a Howe truss configuration. In the latter, the web members form inverted N-shaped panels with the verticals mainly in tension and the diagonals predominantly in compression. For the same truss dimensions and under the same applied load, the diagonal compression in the Howe truss will be higher than the vertical compression in the Pratt truss, which may require larger size of the diagonal CFFT, and hence increase in self-weight. Also, longer compression members are not desirable in terms of stiffness and stability, particularly for deep girders. Furthermore, in the Pratt truss, the concrete chord of the end panels (*a*-*b* in Fig. 2a) is subjected to zero or very small tension. The length of these panels can be more than sufficient for the transfer of prestressing. In the Howe truss, on the other hand, the bottom chord of the end panels is subjected to tensile force of magnitude close to or equal to the support reaction, thus requiring extension of the chord beyond the girder end for transfer of the prestressing.

After erection, the girders may be post-tensioned with external tendons to balance the deck weight, to provide continuity in multi-span bridges, and to resist subsequent loads on the bridge. In each girder, an external tendon can be harped (held down) to the bottom chord at one or two points within the span and held up to the top chord at one point near the intermediate supports in continuous bridges. No deviators are required at the harping points. The horizontal parts of the tendons between the harping points pass through ducts placed inside the bottom chord in a single-span bridge, or inside both the top and bottom chords in a continuous bridge. Deviation of the tendons from the horizontal is done at the location of the truss joints. The ducts can be left ungrouted for easy replacement of the tendons, or can be grouted to achieve bond between the horizontal parts of the tendons and the concrete chord(s).

SELF-WEIGHT COMPARISON WITH THE STANDARD I-GIRDERS

One of the main advantages of the proposed precast truss girder is the reduction of self-weight. To give an insight into how much reduction can be anticipated, a comparison with the standard I-girders is made below.

The Canadian Precast/Prestressed Concrete Institute (CPCI) in its Design Manual[4] recommends a number of standard I-shaped sections for slab-on-girder bridge construction. The CPCI girder sections vary in depth from 900 to 2300 mm (3 to 7.7 ft). The girders are designated CPCI 900, CPCI 1200, . . ., CPCI 2300. Dimensions of the girder sections are shown in Fig. 3. Table 1 presents a comparison of the self-weight of the CPCI standard I-girders with that of truss girders of corresponding depth. In this comparison, dimensions of the top and bottom concrete chords of the truss girders are taken equal to those of the top and bottom flanges of the standard I-girders. Only the concrete web of the I-girders is assumed replaced by CFFT elements in the truss girders. Tube diameter of 150 mm (6 in.) is used in the calculations. As can be seen, a reduction in weight ranging from 22.6% to 36.6% can be achieved by using the truss girder system. The table also shows that the deeper the section the greater the reduction in weight.

EXPERIMENTAL PROGRAM

An extensive experimental program is in progress at the University of Calgary aiming at investigating the constructability and structural performance of the proposed hybrid bridge system and its components under various loading conditions. Static and fatigue loading tests have been carried out on isolated segments of the truss girder comprising one vertical and one diagonal CFFT elements connected to portions of the top and bottom chords. Different types of FRP tube and connection of the truss elements to the chords have been tested. Details and results of these tests have been reported[2,3]. The following sections present another part of the program consisting of testing a total of ten large-scale truss girder specimens under static and fatigue loading. All the ten girders had the same overall depth of 1.2 m (4 ft) and identical cross-section dimensions of the concrete chords. Dimensions and type and amount of reinforcement of the vertical as well as those of the diagonal CFFT truss elements were the same in all girders. Four of the girders consisted of 2, 4, 6, and 8 truss panels, respectively. The span length varied from 2.31 to 9.24 m (7.6 to 30.3 ft), resulting in span-to-depth ratio varying from 1.925 to 7.7. The four girders were tested under monotonic static loading up to failure. The remaining six girders consisted of two truss panels each and were tested under cyclic fatigue loading of different level and amplitude. Details of fabrication, instrumentation and testing of the truss girder specimens are given below. Results of the tests are also reported and discussed.

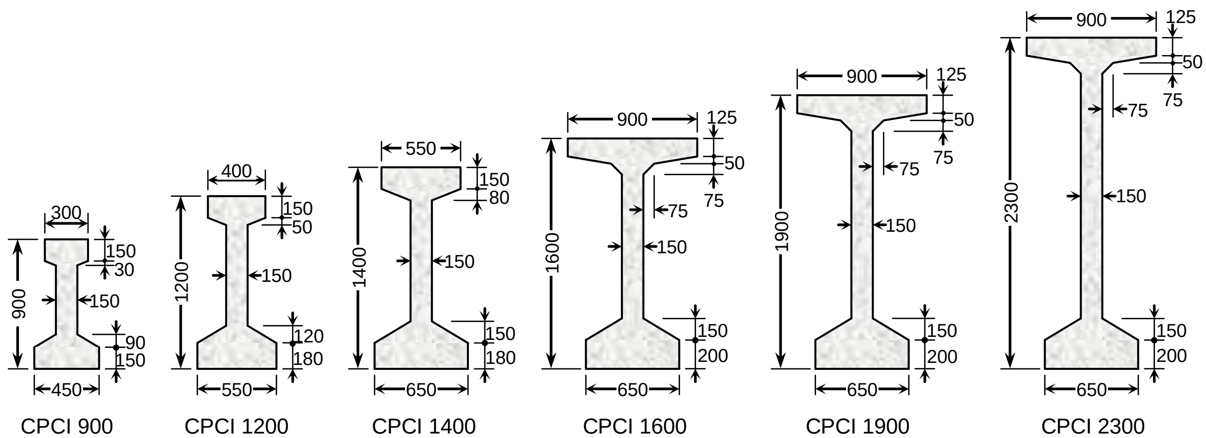


Fig. 3 Dimensions of the CPCI standard I-girders (1 in. = 2.54 mm)

Table 1 Self-weight comparison between CPCI and truss girders

Girder Designation	h_{total}, mm (in.)	h_{web}, mm (in.)	I-Girder W_g, kN/m (kip/ft)	Truss-Girder W_g, kN/m (kip/ft)	Self-weight Reduction, %
CPCI-900 TRUSS-900	900	480	5.23 (0.358)	4.05 (0.278)	22.6
CPCI-1200 TRUSS-1200	1200	700	7.68 (0.526)	5.76 (0.395)	27.8
CPCI-1400 TRUSS-1400	1400	840	9.91 (0.679)	7.50 (0.514)	26.3
CPCI-1600 TRUSS-1600	1600	1000	11.98 (0.821)	9.02 (0.618)	24.1
CPCI-1900 TRUSS-1900	1900	1300	13.06 (0.895)	9.08 (0.622)	30.0
CPCI-2300 TRUSS-2300	2300	1700	14.50 (0.994)	9.13 (0.626)	36.6

DETAILS AND FABRICATION OF THE GIRDER SPECIMENS

For the truss girder specimens fabricated and tested in this research, dimensions of the concrete top and bottom chords of all girders were chosen equal to those of the CPCI 900 girder section. However, the overall truss girder depth was taken equal to 1200 mm (4 ft), the depth of the CPCI 1200 girder section. The panel length of the truss was taken equal to 1155 mm (3.85 ft) allowing for a 45-degree angle of the diagonal CFFT members. Thus, the span length varied from 2.31 m to 9.24 m (7.6 to 30.3 ft), leading to span-to-depth ratio varying from 1.93 to 7.70 for the 2 to 8-panel girders, respectively. Reinforcement of the girder elements were the same for all specimens except for prestressing of the bottom chord. Figure 4 depicts the dimensions and reinforcing details of a 2-panel girder as example. The bottom chords of the 2, 4, 6, and 8-panel girders, respectively, were pre-tensioned with 4, 6, 8, and 10-15 mm (0.6 in.) 7-wire low relaxation strands stressed to 70% of their ultimate tensile strength. Two strands in the top chord of each girder were needed to eliminate or control the tensile stresses and cracking due to local bending of the chord. The top and bottom chords were reinforced with 10M (# 3) stirrups and 15M (# 5) non-prestressed longitudinal bars arranged as shown in Figs. 4a and c. A minimum concrete cover of 30 mm (1.2 in.) was used for all stirrups.

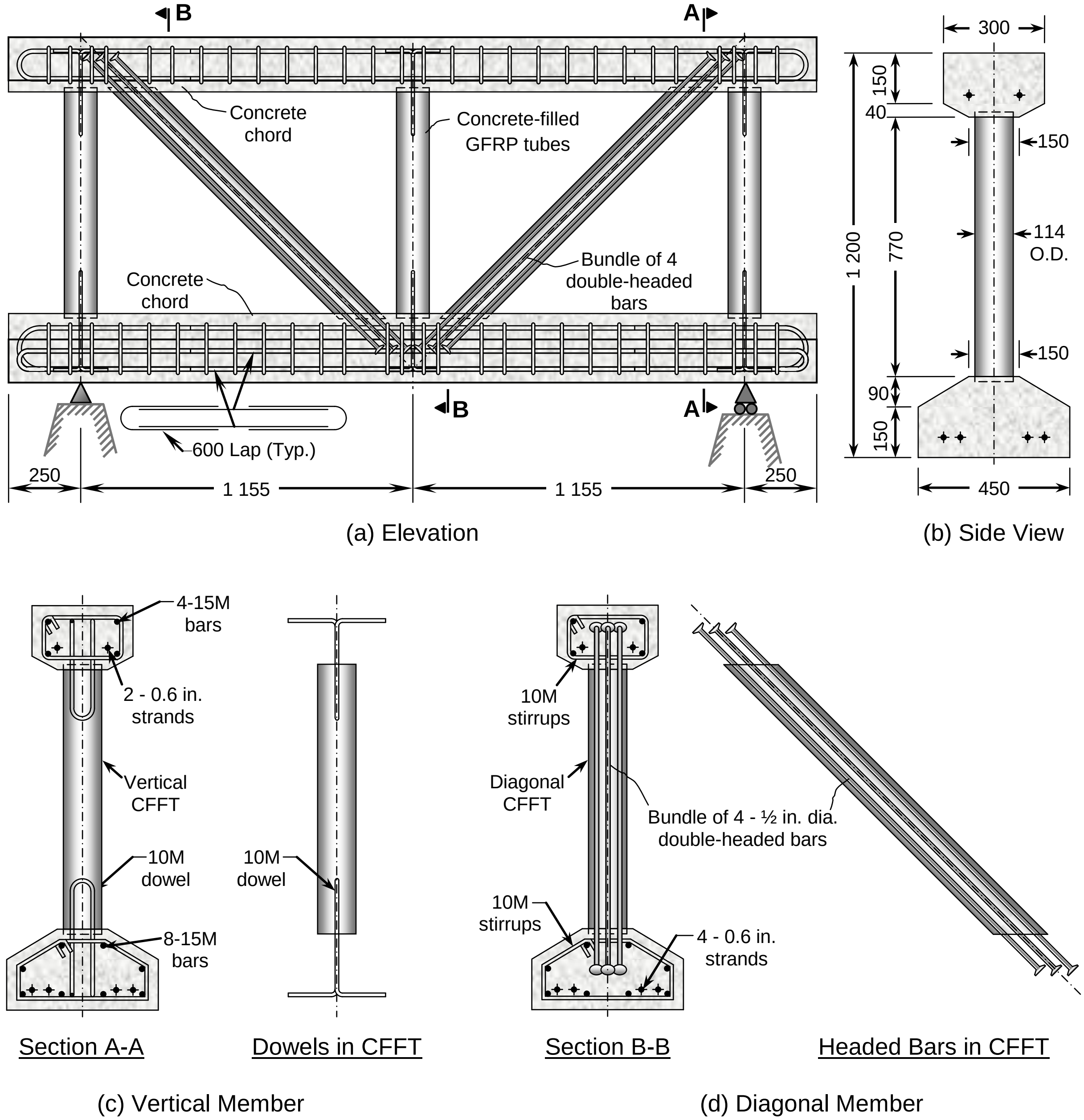


Fig. 4 Dimensions and reinforcing details of a 2-panel truss girder specimen (1 in. = 25.4 mm)

Filament-wound glass FRP tubes with 4 in. nominal diameter and approximately 70% circumferential fibers and 30% longitudinal fibers were used for the truss web elements. The inner diameter and wall thickness of the tube were 110 mm and 1.9 mm (4.4 in. and 0.075 in.), respectively. The CFFT vertical elements were connected to the top and bottom chords by means of 10M (# 3) dowels protruding from the ends of the tubes (Figs. 4a and c). Each diagonal CFFT was connected to the concrete chords by a bundle of four 12.7 mm (½ in.) dia. long double-headed plain bars with the heads embedded in the chords and the stems extending through the length of the CFFT (Figs. 4a and d).

It should be noted that in this series of girder specimens all reinforcement and prestressing were made of conventional steel in order to establish a reference and benchmark for the minimum acceptable performance of girders reinforced with FRP or other corrosion-resistant reinforcement and prestressing. Double-headed bars are selected for this project because of their superior anchorage properties. With an area of the head 9 to 10 times the cross-sectional area of the stem, the full yield strength of the bars can develop immediately behind the head, without the need for the development length that is required in conventional reinforcement[5].

Specimen Designation

The four truss girders tested under static loading were designated as G*i*-*j*D-S, where "G" refers to "Girder", i = 1, 2, 3, or 4 is the girder number, j = 2, 4, 6, or 8 is the number of truss panels in the girder, "D" refers to double-headed bars (as opposed to spliced single-headed bars used in another series of tests), and "S" refers to static loading. The six 2-panel girder specimens tested under fatigue are designated as G*n*-2D-F*i*-*j*, where n = 5, 6, . . ., 10 is the girder number, "2D" refers to 2-panel girder reinforced with double-headed bars, "F" refers to "Fatigue" loading, i is the upper load limit in kN, and j is the load range applied.

MATERIAL PROPERTIES

Concrete

The target compressive strength of concrete for all specimens was 60 MPa (8.7 ksi) at 28 days. The strength at release of pre-tensioning was specified at 35 MPa (5 ksi). Cement type GU was used in the concrete mix with water-to-cement ratio of 0.4 and air entrainment between 5% and 8%. Maximum aggregate size of 10 mm (0.4 in.) was used in order to obtain complete consolidation of concrete in the chords and in the FRP tubes. The concrete compressive and tensile strengths measured at 28 days and the compressive strength measured on the day of testing of the ten girder specimens are listed in Table 2. The values shown are the average of testing 3 standard 100 x 200 mm (4 x 8 in.) concrete cylinders.

Glass FRP Tubes

The mechanical properties of the glass FRP tubes in the longitudinal and circumferential directions were provided by the manufacturer in accordance with the ASTM test methods D2105 and D1599, respectively. These properties are summarized in Table 3.

Reinforcement

Conventional deformed reinforcing bars of size 10M (# 3) and 15M (# 5) were used, respectively, for stirrups and longitudinal reinforcement in the concrete chords (Fig. 4). The

U-shaped dowels with 90-degree bends used for connecting the vertical CFFT to the concrete chords (Fig. 4c) were made of 10M (# 3) bars. Properties of the two sizes of bars as determined from tensile tests are given in Table 4. The tensile properties of the steel strands used for pre-tensioning the concrete chords (Fig. 4b) are also given in Table 4.

Double-headed plain steel bars of 12.7 mm (½ in.) diameter with a 40.2 mm (1.6 in.) diameter head were used to reinforce the diagonal CFFT and connect them to the concrete top and bottom chords (Fig. 4d). The heads were stud welded to the bar and conform to ASTM A1044, which requires the area of the head to be at least 10 times the area of the bar. Tension tests were performed on the bars with and without the heads. The tensile properties as determined from the tests are listed in Table 4. Values shown in the table are average of tests on 3 samples of each reinforcement type.

Table 2 Concrete strength

Girder Designation	Compressive Strength, MPa (ksi)		Tensile Strength at 28 Days,,MPa (ksi)
	At 28 Days	On the Day of Testing	
G1-2D-S	67.6 (9.80)	70.1 (10.2)	5.1 (0.74)
G2-4D-S	67.8 (9.83)	68.9 (10.0)	4.9 (0.71)
G3-6D-S	69.6 (10.1)	71.3 (10.3)	5.0 (0.73)
G4-8D-S	64.2 (9.31)	71.1 (10.3)	4.1 (0.59)
G5-2D-F550-200	60.4 (8.76)	64.5 (9.35)	5.5 (0.80)
G6-2D-F500-200	58.8 (8.53)	66.3 (9.61)	5.1 (0.74)
G7-2D-F450-150	55.8 (8.09)	56.4 (8.18)	4.3 (0.62)
G8-2D-F430-140	55.1 (7.99)	55.2 (8.00)	4.4 (0.64)
G9-2D-F500-200	56.8 (8.24)	75.4 (10.9)	5.1 (0.74)
G10-2D-FV-V*	72.0 (10.4)	78.0 (11.3)	5.2 (0.75)

* V refers to variable upper load limit and variable load range

Table 3 Mechanical properties of glass FRP tubes

Tensile Strength, MPa (ksi)		Compressive Strength, MPa (ksi)	Tensile Modulus, GPa (ksi)		Compressive Modulus, GPa (ksi)	Poisson's Ratio
Long.	Circum.	Long.	Long.	Circum.	Long.	
240 (34.8)	480 (69.6)	240 (34.8)	20.6 (2987)	29 (4200)	20.6 (2987)	0.16-0.26

Table 4 Tensile properties of all types of reinforcement used in the truss girders

Type of Reinforcement	Diameter, mm (in.)	Cross-sectional Area, mm^2 ($in.^2$)	Modulus of Elasticity, GPa (ksi)	Yield / Proof Strength MPa (ksi)	Ultimate Strength, MPa (ksi)
10M (# 3) bar	11.1 (0.38)	100 (0.16)	201.8 (29260)	455 (66)	645 (93.5)
15M (# 5) bar	15.9 (0.63)	200 (0.32)	196.6 (28500)	430 (62)	580 (84.1)
7-wire strand	15.2 (0.60)	140 (0.22)	192.5 (27900)	1792 (260)	1943 (282)
Double-headed bar	12.7 (0.50)	127 (0.20)	200.0 (29000)	465 (67)	560 (81.2)

FABRICATION OF THE TRUSS GIRDERS

The girders were fabricated in a local precast plant in Calgary. The vertical and diagonal CFFT truss members were produced prior to the chords. The glass FRP tubes were cut to length and fixed in special wooden frames. The dowels were fixed in position at the center of each end of the vertical tubes. Each four double-headed bars were bundled and inserted in the diagonal tube with the heads protruding from the tube ends. The wooden frame was provided with templates to fix the headed bars in place and to ensure that the centroidal axis of the bundle coincides with that of the tube. The vertical and diagonal tubes were then filled with concrete. Figures 5 and 6 show the vertical and diagonal members, respectively, before and after casting. Figure 7 shows the formwork of the 8-panel girder with the truss members in place in preparation for pre-tensioning and casting the chords. For ease of fabrication the chords were cast in a 90-degree rotated position with the CFFT truss members placed in a horizontal plane as shown in Fig. 7. Figure 8 shows the girder after casting and stripping the forms.

In the 8-panel girder, a 5 mm (0.2 in.) camber was measured at mid-length at release of pre-tensioning. A frame analysis of the girder, assuming rigid connections of the CFFT elements to the chords, resulted in a 3.5 mm (0.14 in.) camber. Casting and pre-tensioning of the girder in a rotated position has the effect of delaying the action of self-weight until after the concrete gains higher strength.

SHIPPING AND HANDLING THE GIRDERS

Following fabrication, the girders were lifted in a flat position, as shown in Fig. 9a, and moved to the steam curing chamber. After curing, the girders were shipped to the University of Calgary’s Structures Laboratory for testing. Due to the light weight of the girders and the geometry and dimensions of their top and bottom chords, it was possible to ship more than one girder at a time as the girders could be stacked in a horizontal position on top of each other on the truck as shown in Fig. 9b. The girders could also be stacked in their horizontal position for storage (see Fig. 9c). These advantages result in saving in the transportation cost and reduction in the overall shipping time, as well as reduction in the space required for storage.

Fig. 5 The vertical truss elements before and after casting

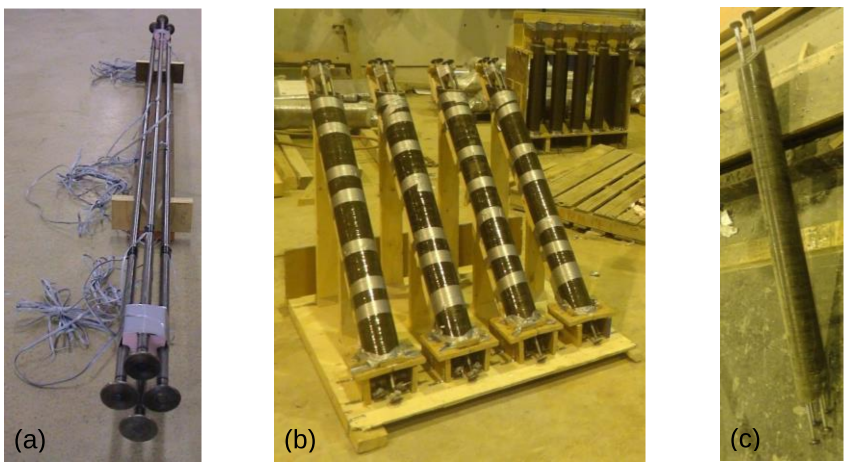


Fig. 6 Bundling the double-headed bars and the diagonal truss elements before and after casting

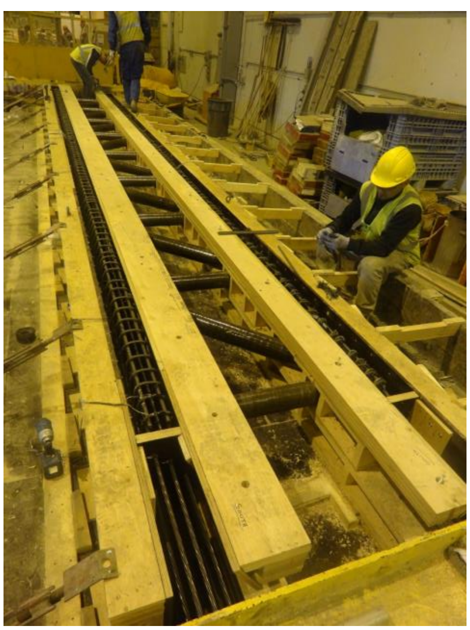


Fig. 7 Chords of 8-panel girder before casting

Fig. 8 The 8-panel girder after casting

(a) Lifting (b) Shipping (c) Storage

Fig. 9 Shipping and handling of the truss girders

TEST SETUP

Each girder was simply supported on two steel rollers placed on two pedestals in the loading frame and subjected to a vertical load, either static or fatigue, applied at the middle of the top chord using a 1.0 MN (225 kip) capacity hydraulic actuator. A bearing plate was placed on the girder top surface underneath the actuator shaft. In order to restrain any out-of-plane displacement, the top chord was laterally supported at mid-span and at 1 m (3.28 ft) spacing on each side of the mid-span. The bottom chord was laterally supported only at mid-span. The lateral supports at mid-span were connected to the testing frame, whereas all other lateral supports were connected to the strong floor. All lateral supports were adjustable. Aluminium plates were glued to the side of the chord at the location of the lateral supports to reduce friction and to allow free movement in the vertical plane while out-of-plane displacements were restrained. Figure 10 depicts a schematic view of the test setup of the 6-panel girder. The test setup for all other girders was similar. Figure 11 shows the 8-panel girder during the test.

INSTRUMENTATION

A load cell was placed between the actuator shaft and the bearing plate on top of the girder to measure the applied load. Laser transducers (LTs) were used underneath the bottom chord to measure deflection of the girder specimens during the tests. Three LTs were used to measure the deflection of the 2- and 4-panel specimens, while 5 and 7 LTs were used to measure the deflection of the 6- and 8-panel specimens, respectively. The LTs were placed under the vertical truss elements for all specimens except the 2-panel specimens, where one LT was placed in the middle and two others were placed at the mid-length of the panels.

A total of seven mechanical transducers (MTs) were used to measure the elongation of the two CFFT diagonal members in the middle of the girders and the shortening of the three vertical members: the one at the middle and the two at the ends. Each double-headed bar in the two middle diagonal members was instrumented with a strain gauge located near each head. The bottom longitudinal reinforcing bars in the top and bottom chords were also instrumented with strain gauges at mid span of the girder. Strain gauges were also used in the four girders tested under static loading at the outer surface of the FRP tubes of the middle and end vertical elements to measure their longitudinal and circumferential strains.

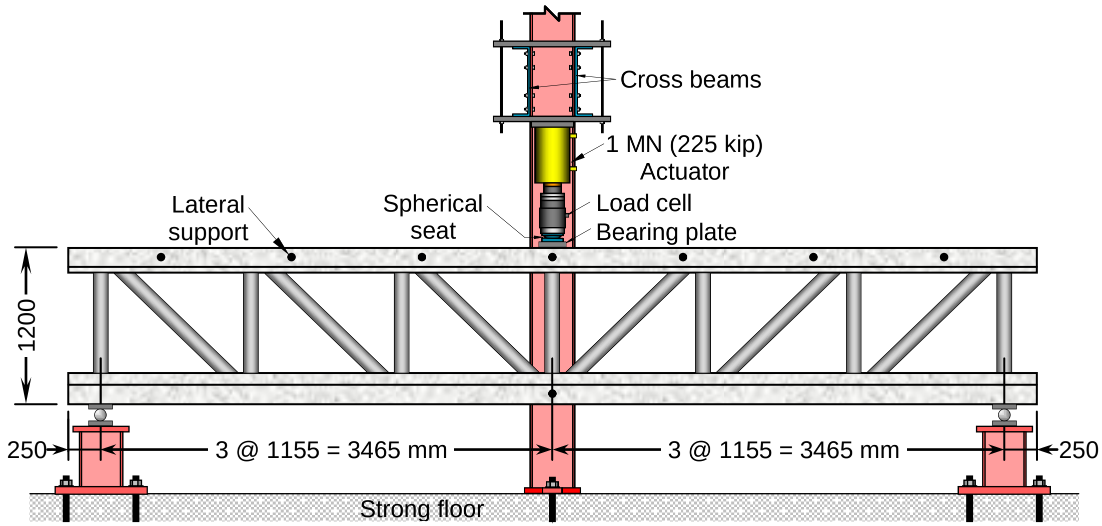


Fig. 10 Test setup of a 6-panel girder (1 in. = 25.4 mm)

Fig. 11 Eight-panel truss girder during the test

STATIC AND FATIGUE LOADING ROUTINES

Girder specimens G1 to G4 were tested under static loading increasing monotonically from zero to failure. The six girders G5 to G10 were tested under fatigue with different upper and lower load limits. Girder G10 was tested under a variable upper load limit and a constant lower load limit of 40 kN (9 kip). The lower limit represents the superimposed dead load on a bridge, while the variable load range represents the moving live load. The variable upper load limit was applied over eight periods of 250,000 cycles each. After completion of each period, the upper load limit was increased by 43.75 kN (9.83 kip) increment, which is equal to 50 percent of the maximum wheel load of CL-W Truck of the Canadian Highway Bridge Design Code, CAN/CSA-S6-06[6].

Table 5 lists the specified upper and lower load limits, as well as the load range for each of the six fatigue specimens. Typically, each fatigue girder was loaded first by a static load increasing monotonically from zero to the specified upper load limit in order to produce cracking in the top and bottom concrete chords. This was followed by three static loading cycles between the specified lower and upper load limits. The static cycles were followed by fatigue load cycles applied at a rate of 3 Hz. After fatigue damage of each girder, a static load was applied monotonically from zero to complete failure in order to investigate the residual load-carrying capacity of the girders. In all tests, the static loads were applied in a displacement-controlled mode, while the fatigue cycles were applied in a load-controlled mode.

Table 5 Fatigue loading

Girder Designation	Lower Load Limit, kN (kip)	Upper Load Limit, kN (kip)	Load Range, kN (kip)
G5-2D-F550-200	350 (78.7)	550 (124)	200 (45.0)
G6-2D-F500-200	300 (67.4)	500 (112)	200 (45.0)
G7-2D-F450-150	300 (67.4)	450 (101)	150 (33.7)
G8-2D-F430-140	290 (65.2)	430 (96.7)	140 (31.5)
G9-2D-F500-200	300 (67.4)	500 (112)	200 (45.0)
G10-2D-FV-V	40 (9)	$40 + i\times 87.5$* ($9 + i\times 19.7$)	$i\times 87.5$ ($i\times 19.7$)

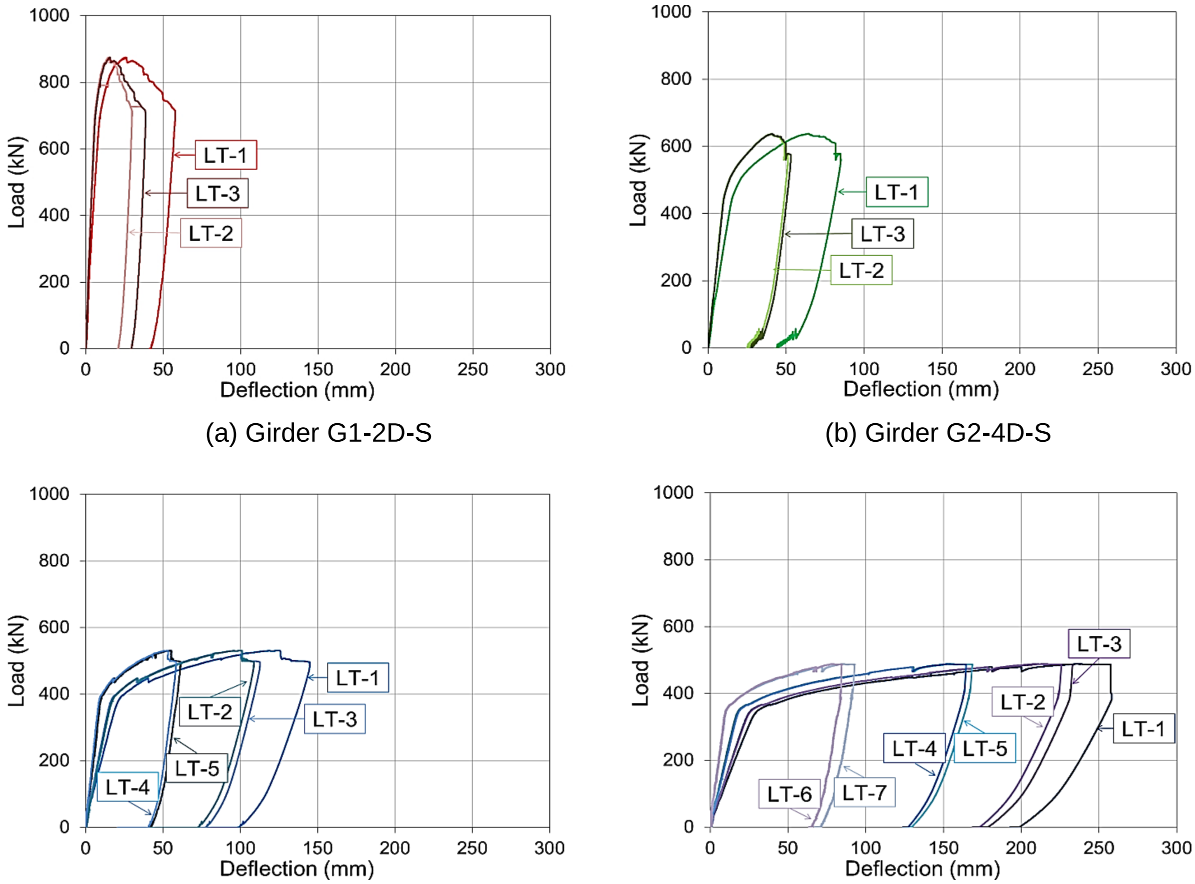
* The 87.5 kN (19.7 kip) is the maximum wheel load of the CL-W Truck of CAN/CSA-S6-06, and i = 1.0, 1.5, . . ., 4.5.

TEST RESULTS AND DISCUSSION

BEHAVIOR UNDER STATIC LOADING

The load-deflection response of girder G1 to G4 under static loading is shown in Fig. 12. The load-deflection curves are shown for all locations of laser transducers in each girder. Figure 13 compares the load versus mid-span deflection response of the four girders. In the 8-panel girder, the mid-span deflections measured and calculated at 75% of the load at yielding were 24 and 16 mm (0.96 and 0.63 in.), respectively. It should be mentioned that loading the 8-panel girder was stopped at 200 mm (8 in.) deflection when the actuator reached its maximum stroke. The girder was then unloaded and two thick steel plates were placed underneath the spherical seat. The girder was then reloaded to failure. The effect of unloading and reloading on the girder's behavior is shown by the dotted lines in Fig. 13. The figure clearly shows the effect of the span-to-depth ratio, *R*, on the load-deflection response of the girders.

Table 6 lists the critical loads: P_{cr}, $P_{y,first}$, $P_{y,last}$, and P_u, for the four girders; where P_{cr} is the load at first cracking of the chords, $P_{y,first}$ is the load at first yielding of the headed bars in one of the diagonal CFFT members, $P_{y,last}$ is the load at last yielding of the headed bars, and P_u is the ultimate load. The table indicates that there is no significant increase in the applied load after yielding of the headed bars within the CFFT diagonal members. These results show that the overall load carrying capacity of the truss girder is governed primarily by the double-headed bars reinforcing the diagonal members, as long as the capacity of the other components – the vertical CFFT members and the concrete chords – is sufficient to allow the headed bars to reach yielding.



(a) Girder G1-2D-S
(b) Girder G2-4D-S
(c) Girder G3-6D-S
(d) Girder G4-8D-S

Fig. 12 Load-deflection response of the truss girders under static load
(1 in. = 25.4 mm; 1 kip = 4.448 kN)

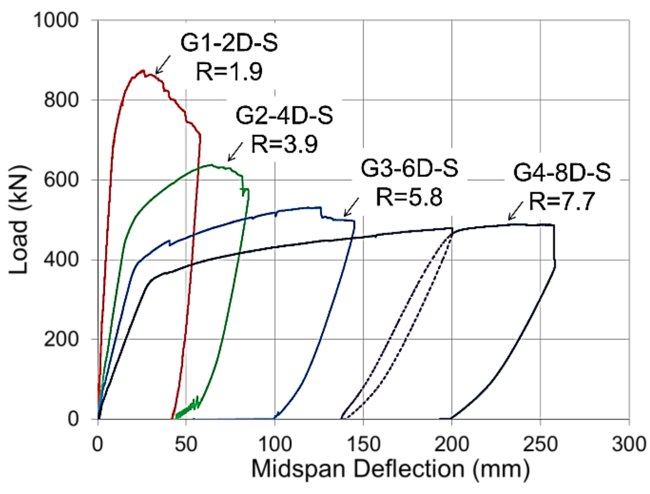


Fig. 13 Effect of the span-to-depth ratio, R, on the load-deflection response
(1 in. = 25.4 mm; 1 kip = 4.448 kN)

Table 6 Critical loads on the truss girder specimens

Girder Designation	Span-to-Depth Ratio, R	At First Cracking P_{cr}, kN (kip)	At First Yielding of Headed Bars, $P_{y,\,first}$, kN (kip)	At Last Yielding of Headed Bars, $P_{y,\,last}$, kN (kip)	Ultimate Load, P_u, kN (kip)
G1-2D-S	1.925	300 (67.4)	670 (151)	860 (193)	875 (197)
G2-4D-S	3.850	225 (50.6)	495 (111)	635 (143)	638 (143)
G3-6D-S	5.775	215 (48.3)	425 (95.5)	510 (115)	531 (119)
G4-8D-S	7.700	185 (41.6)	400 (89.9)	450 (101)	488 (110)

Failure Mode under Static Loading

After yielding of all the critical headed bars, the ultimate failure mechanism and ductility of the truss girders became dependent on the structural properties of the FRP tube in the vertical members and the performance of the top and bottom concrete chords. In the shortest girder, G1-2D-S, ultimate failure occurred by crushing of concrete of the top and bottom chords, following yielding of the longitudinal flexural bars within the chords, indicating ductile behavior of the girder (Fig. 14). No sign of significant rupture of the FRP tubes of the vertical CFFT members was observed during testing of this girder.

(a) Crushing of Top Chord (b) Crushing of Bottom Chord

Fig. 14 Failure of girder G1-2D-S: Crushing of the concrete chords after yielding of the headed bars and the flexural reinforcement

In the longest girder, G4-8D-S, in addition to crushing of the concrete in the top and bottom chords, following yielding of the headed bars in the diagonal members and the longitudinal flexural bars in the chords, bending of the vertical CFFT member was observed over the support, where the rotation of the top and bottom chords was largest (Fig. 15a). The calculated rotational stiffness of the vertical CFFT member relative to that of the top chord is 1:159.

Also, local rupture at the bottom of the FRP tube of the second vertical member from the support was observed at failure (Fig. 15b). The circumferential strain measured at near the ends of the vertical members increased dramatically after yielding of all the headed bars in the diagonal members. Frame analysis of the girder revealed that, of all the vertical members, the second member from the support carried the largest axial compressive force.

(a) Bending of Vertical CFFT Member (b) Local Rupture of Vertical FRP Tube

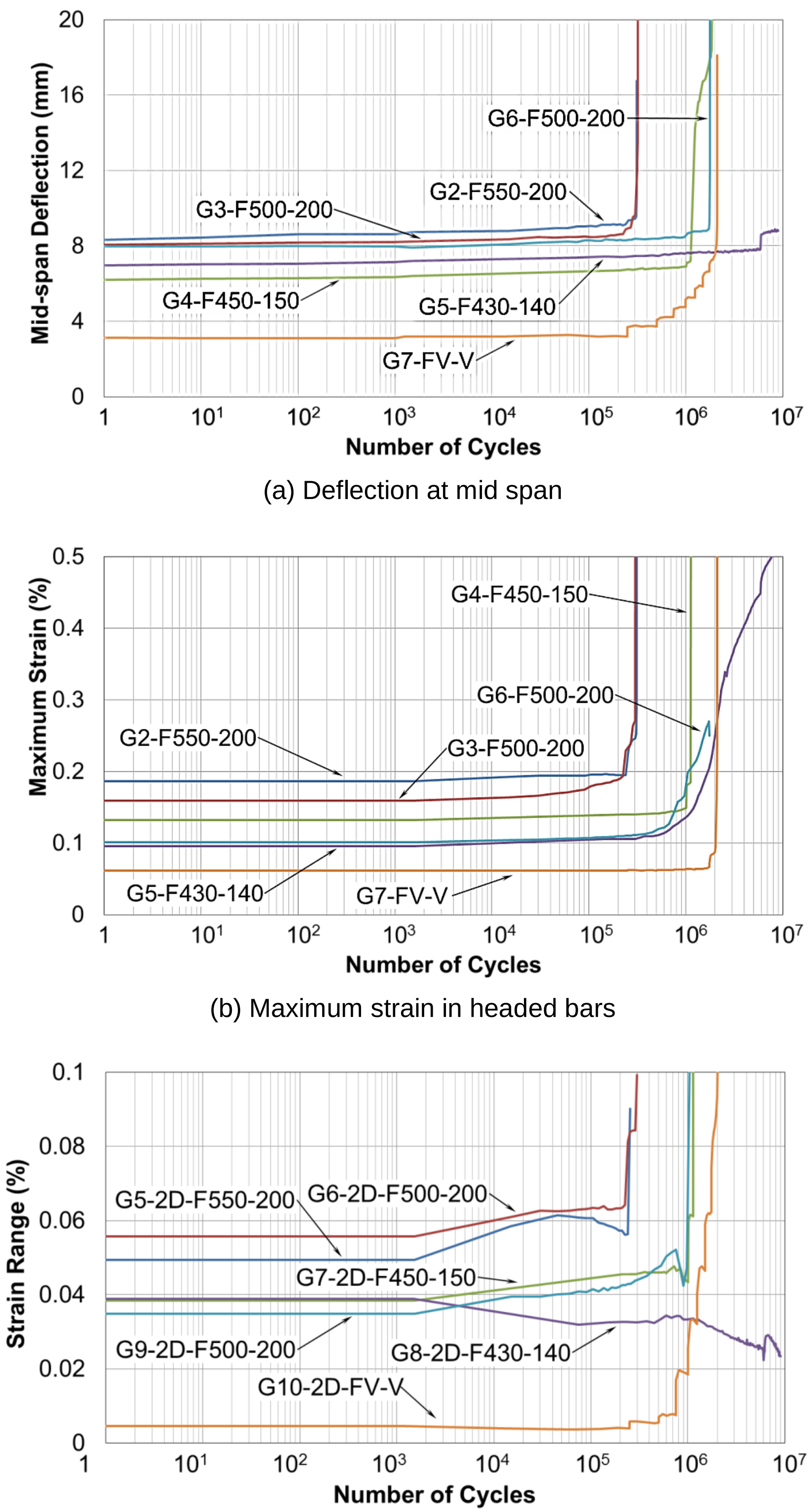
Fig. 15 Failure of girder G4-8D-S

BEHAVIOR UNDER FATIGUE LOADING

Figure 16a shows variation of the mid-span deflection with the number of fatigue cycles in girders G5 to G10 for different upper loads and load ranges. The figure shows almost constant deflection throughout the fatigue tests prior to failure. The most critical parameters that affect the fatigue life of a structure are the upper stress level and the stress range. For each girder, Fig. 16b shows variation of the maximum strain induced in the most stressed headed bar with the number of load cycles. The largest strain range induced in the headed bars versus the number of cycles is plotted in Fig. 16c for each girder.

Table 7 gives the initial stress limits and the corresponding stress range in the most stressed headed bars in the diagonal truss members as well as the maximum number of fatigue cycles reached at the end of the test. Except in girder specimen G8-F430-140, typical failure due to fatigue occurred at the middle truss joint in the bottom chord by facture of the headed bars in one of the diagonal elements at the weld with the head. This was accompanied by a significant increase in deflection (Fig. 16a) and damage in the top concrete fibers of the bottom chord around the vertical CFFT. Girder specimen G8-F430-140, which was subjected to the lowest upper load, survived 9.0 million cycles without any failure and the test was terminated.

The Canadian Highway Bridge Design Code, CSA-S6-06[6] limits the change in stress under repeated service loads to 65 MPa (9.4 ksi) for anchorages and connections and to 125 MPa (18 ksi) in reinforcing and prestressing steel. The code also specifies 2 million cycles as a minimum acceptable fatigue life of a highway bridge. Table 7 indicates that girders G7, G9, and G10 which experienced maximum stress in the headed bars ranging from 55% to 75% of the yield strength and stress range in the bars close to or slightly higher or lower than the code specified limit survived close to or slightly higher than 2 million fatigue cycles.



(a) Deflection at mid span

(b) Maximum strain in headed bars

(c) Strain range in headed bars

Fig. 16 Variation of deflection at mid-span and maximum strain and strain range in the headed bars with the number of fatigue cycles (1 in. = 25.4 mm)

Table 7 Fatigue stress limits and maximum number of cycles

Girder Designation	Maximum Stress, MPa (ksi)	Stress Range, MPa (ksi)	Maximum Number of Cycles
G5-F550-200	390 (56.6)	133 (19.3)	311,853
G6-F500-200	319 (46.2)	122 (17.7)	321,129
G7-F450-150	348 (50.5)	126 (18.3)	1,875,940
G8-F430-140	259 (37.6)	78 (11.3)	9,000,000
G9-F500-200	310 (45.0)	116 (16.8)	1,811,076
G10-FV-V	257 (37.3)	149 (21.6)	2,106,418

Note: Test on Girder G8 was terminated after 9,000,000 cycles without failure.

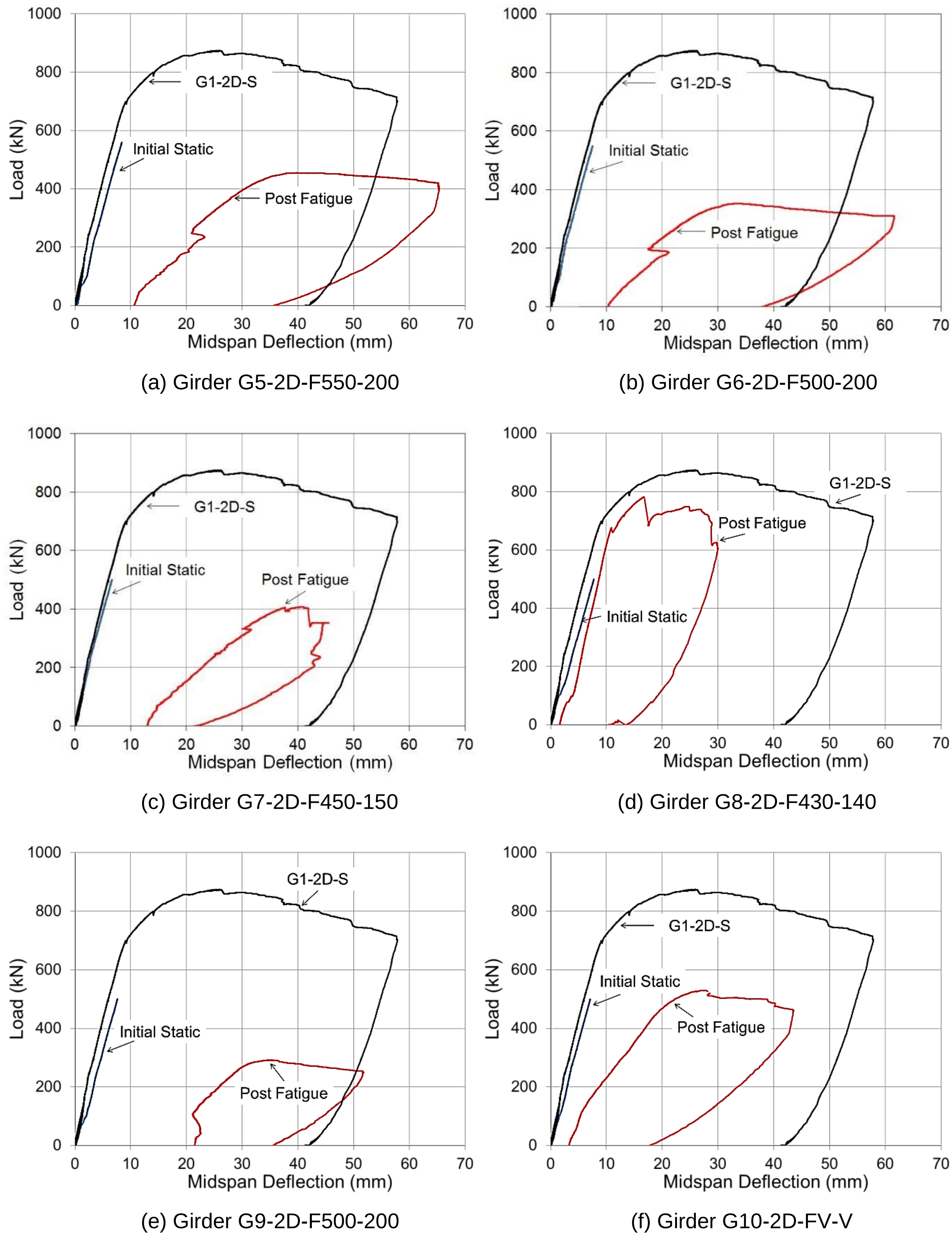
POST-FATIGUE BEHAVIOR

The post-fatigue load-deflection diagrams obtained under monotonic static loading to complete failure of girders G5 to G10 are presented in Fig. 17. The load-deflection curve for girder G1-2D-S tested under static loading applied monotonically from zero to failure is also plotted for comparison. As can be seen in Fig. 17, with the exception of girder G8, all the fatigue specimens experienced significant permanent deflection following the cyclic loading. Also, significant degradation of stiffness took place due to the fatigue loading. Table 8 lists the permanent deflection values measured at the end of the fatigue tests. Also listed are the stiffness values for the fatigue girder specimens before and after the cyclic loading. It can also be seen from Fig. 17 that, in comparison with girder G1-2D-S, all the fatigue specimens experienced reduction in their ultimate load carrying capacity. The reduction in the load capacity varied from 40% to 67% of the 875 kN (197 kip) capacity of girder G1-2D-S. Girder G10 which was tested under variable upper load experienced the smallest reduction in stiffness and load capacity after fatigue loading.

Table 8 Effect of cyclic loading on girder post-fatigue stiffness and load carrying capacity

Girder Designation	Post-Fatigue Permanent Deflection (mm)	Initial Stiffness (kN/mm)	Post-Fatigue Stiffness (kN/mm)	Stiffness Degradation (%)	Post-Fatigue Ultimate Load (kN)	Reduction in Ultimate Load (%)
G5-F550-200	10.7	66.7	24.2	64	455	48
G6-F500-200	10.4	73.9	23.1	69	350	60
G7-F450-150	13.1	75.3	20.6	73	406	54
G8-F430-140	1.6	64.4	56.0	13	782	10
G9-F500-200	21.5	65.8	23.8	64	291	67
G10-FV-V	3.3	70.3	36.6	48	529	40

Note: 1 in. = 25.4; 1 kip = 4.448 kN



(a) Girder G5-2D-F550-200

(b) Girder G6-2D-F500-200

(c) Girder G7-2D-F450-150

(d) Girder G8-2D-F430-140

(e) Girder G9-2D-F500-200

(f) Girder G10-2D-FV-V

Fig. 17 Post-fatigue load-deflection response of girders G5 to G10 in comparison with that of girder G1-2D-S (1 in. = 25.4 mm; 1 kip = 4.448 kN)

SUMMARY AND CONCLUSIONS

A new bridge system consisting of a concrete deck on top of precast prestressed hybrid FRP-concrete truss girders has been developed. The web members of the truss are made of concrete-filled glass FRP tubes connected to prestressed concrete top and bottom chords by means of double-headed plain bars and dowels. The hybrid truss system is light in weight and durable, and hence, economical and cost effective. Fabrication of the truss girder is practical and efficient. Shipping, handling, and storage of the truss girders are economical and lead to saving in transportation cost. The results of testing four truss girders of different span-to-depth ratios under static loading and six two-panel truss girders under fatigue loading showed excellent performance in terms of ultimate strength, stiffness and fatigue life. The fatigue tests under different service load levels and amplitudes showed that the truss girders can satisfy the bridge code fatigue requirements. Post fatigue monotonic loading tests showed reasonable residual load-carrying capacity of the girders following fatigue failure.

ACKNOWLEDGEMENTS

The financial support received from the Natural Sciences and Engineering Research Council of Canada (NSERC), ISIS Canada, Cement Association of Canada, Alberta Innovation and Science, and Oriental Construction Co., Japan, is gratefully acknowledged. Decon, Ontario, donated the headed bar reinforcement. Pipe Specialties, Alberta, Canada provided the filament-wound FRP tubes. The authors are grateful to Armtec for the donation of materials and for fabrication and shipping the truss girder specimens.

REFERENCES

1. Elbadry, M., "An Innovative Hybrid FRP-Concrete System for Short- and Medium-Span Bridges," Proceedings of the *COBRAE 2007 Conference on "Benefits of Composites in Civil Engineering,"* University of Stuttgart, Stuttgart, Germany, March 28-30, 2007, 16 pp.
2. Elbadry, M., Schonknecht, K., and Abe, H., "Experimental Evaluation of Connections in Hybrid Precast Concrete Bridge Truss Girders," *Transportation Research Record, TRR*, Journal of the Transportation Research Board, TRB, V. 2332, No. 2, October 2013, pp. 64-73.
3. Hadizadeh Harandi, M., *Static and Fatigue Behaviour of Connections in Hybrid Concrete-FRP Bridge Truss Girders Using Glass FRP Double-Headed Bars*, M.Sc. Thesis, Department of Civil Engineering, University of Calgary, in progress.
4. Canadian Precast/Prestressed Concrete Institute (CPCI), *Design Manual for Precast and Prestressed Concrete*, 4th Edition, 2007.
5. Ghali, A. and Dilger, W.H., "Anchoring with Double-Head Studs," *Concrete International*, V. 20, No. 11, November 1998, pp. 21-24.
6. CAN/CSA-S6-06, *Canadian Highway Bridge Design Code*, Canadian Standards Association, Rexdale, Ontario, Canada, 2006.