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Performance of a Hybrid FRP-Reinforced Bridge Truss Girder System – Experimental Assessment

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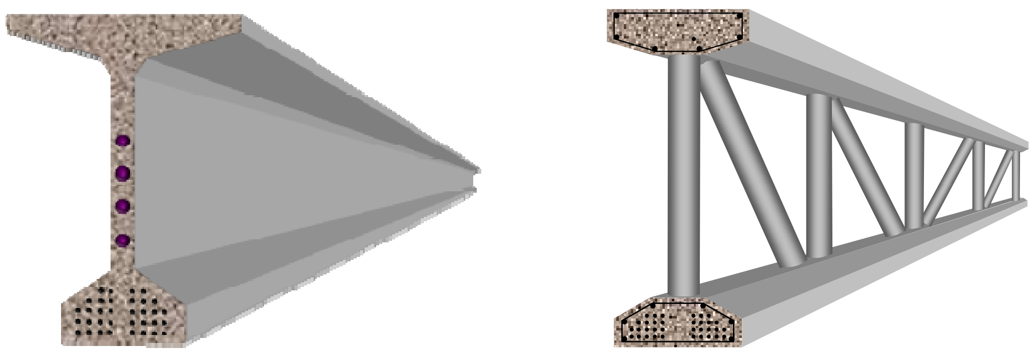
ABSTRACT: An experimental investigation into the performance of a novel precast hybrid FRP-concrete bridge truss girder system under static loading is presented. The girders consist of pretensioned top and bottom concrete chords connected by vertical and diagonal truss members made of concrete-filled fibre-reinforced polymer (FRP) tubes. Under gravity loads, the diagonal members are predominantly in tension and the verticals are mainly in compression. The truss members are connected to the chords by means of long double-headed glass FRP bars. The chords are also reinforced with glass FRP longitudinal bars and transverse stirrups. Four large-scale truss girders were fabricated, and tested under monotonic loading up to failure. All girders had identical cross-section dimensions with 1.32 m overall depth. Two of the girders were 2.83 m in length and consisted of 2 truss panels. The remaining two were 9.82 m long and consisted of 8 truss panels. One 2-panel and one 8-panel girders were reinforced with FRP bars and stirrups. The other two were reinforced with steel and tested as control specimens for comparison purposes. The tests showed excellent performance of the truss girders in terms of strength and stiffness.

1 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; a perspective view of a standard precast concrete I-girder is shown in Figure 1a. While these precast 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. Pretensioning the bottom flange produces initial camber at erection. The camber increases gradually with time due to creep of concrete. The eccentric pretensioning force in the bottom flange can produce tensile stresses of magnitude sufficient to cause cracking at the top of the girder's end sections. The time-dependent effects of creep and shrinkage of concrete and relaxation of prestressing steel can be significant. The girder web thickness can be too small to accommodate continuity post-tensioning. The internal post-tensioning ducts reduce the shear resistance of thin webs. Furthermore, the thin webs can lead to slenderness of deep I-girders, causing stability problems during handling and erection.

An innovative corrosion-resistant system has been recently developed as an alternative to the conventional slab-on-precast I-girder bridges (El-Badry, 2007). The system consists of precast prestressed concrete truss girders and cast-*in-situ* or precast concrete deck. Each truss girder consists of pretensioned top and bottom concrete chords connected by precast vertical and diagonal web members made of concrete-filled FRP tubes (CFFT). A perspective view of a typical truss girder is shown in Figure 1b. Under gravity loads, the vertical members are mainly in compression and the diagonals are predominantly in tension. The truss members are connected to the top and bottom chords by means of long double-headed glass FRP (GFRP) bars. The chords are also reinforced with GFRP longitudinal bars and transverse stirrups. The top and bottom chords provide the flexural capacity, whereas the truss web members resist the shear forces. The deck slab is connected to the girders top chords by means of double-headed GFRP studs. The girders may be post-tensioned by external tendons after erection to balance the slab weight, to provide continuity in multi-span bridges, and to resist the service loads on the bridge.

Advantages of the new system include reduced self-weight and enhanced durability. A self-weight comparison with the standard I-sections indicates a reduction ranging from 22.6 to 36.6 per cent, depending on the girder depth, can be achieved by using the truss system (El-Badry et al., 2014). The substantial reduction of the amount of concrete in the open web alleviates the time-dependent effects of creep and shrinkage. The light weight allows for longer spans or for smaller amounts of prestressing and load on the supports, resulting in reduced size of the substructure or number of the supporting piers in multi-span bridges and, hence, reduction of the initial cost. The FRP tubes serve as stay-in-place formwork and confine the concrete in compression, thus increasing its strength and ductility. The enhancement of durability through the use of FRP can reduce the maintenance cost and extend the bridge's useful life.



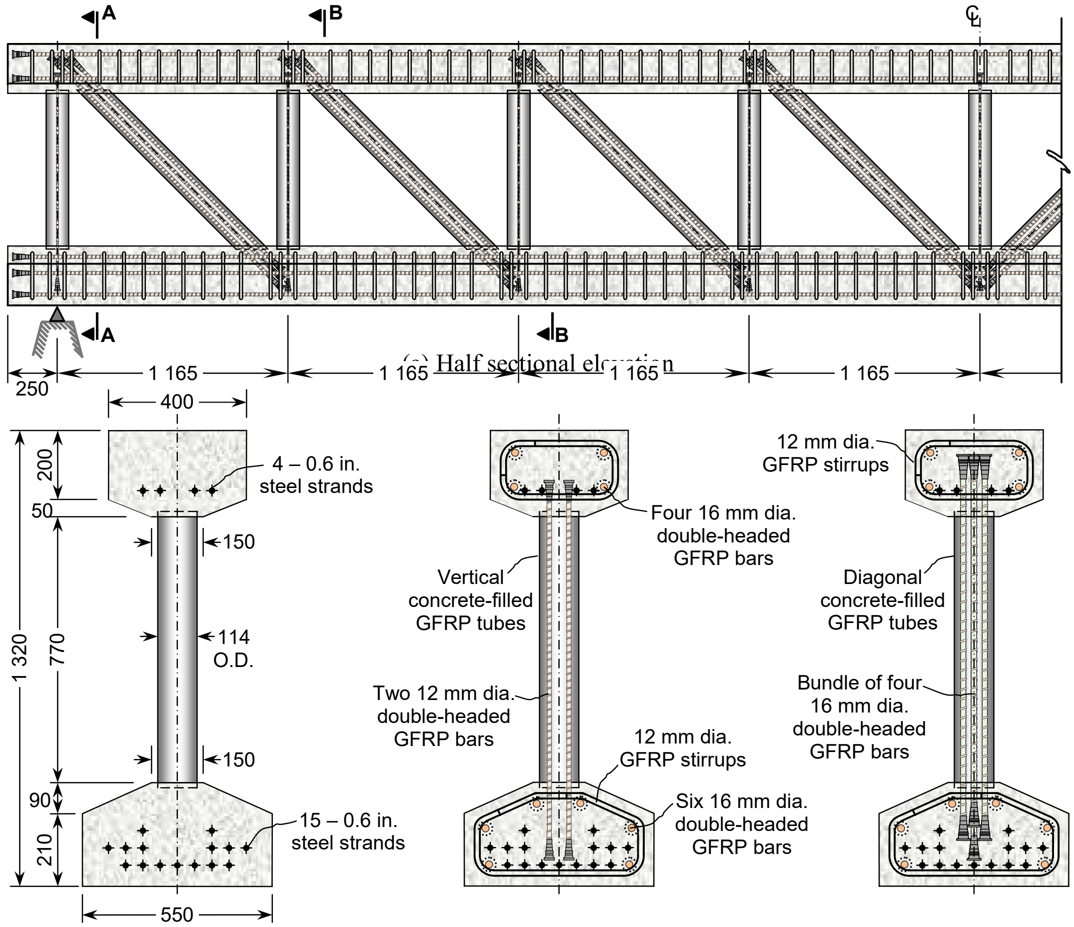
(a) Standard Precast Concrete I-Girder (b) Hybrid FRP-Concrete Truss Girder

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

A comprehensive research program is in progress at the University of Calgary aiming at investigating performance of the hybrid bridge system and its components under various loading conditions. Static and fatigue loading tests have been conducted 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 members to the chords have been tested. Details and results of these tests have been reported by El-Badry et al. (2013) and Hadizadeh Harandi (2015). The following sections present another part of the program concerned with testing large-scale 2- and 8-panel truss girders under static loading up to failure.

2 EXPERIMENTAL PROGRAM

The experimental program consisted of fabricating and testing four large-scale truss girders under monotonic static loading up to failure. Two of the girders were 2.83 m long and comprised 2 truss panels. The remaining two girders were 9.82 m long and consisted of 8 truss panels. All girders had identical cross-section dimensions with 1.32 m overall depth. One 2-panel and one 8-panel girder were reinforced with FRP bars and stirrups. The other two girders were reinforced with steel and tested as control specimens for comparison purposes. The short 2-panel girders were tested to examine the truss shear resistance, whereas the long 8-panel girders were tested to evaluate the flexural resistance. The following sections report and discuss the details and results of the tests.



(b) Side view (c) Section A-A (d) Section B-B

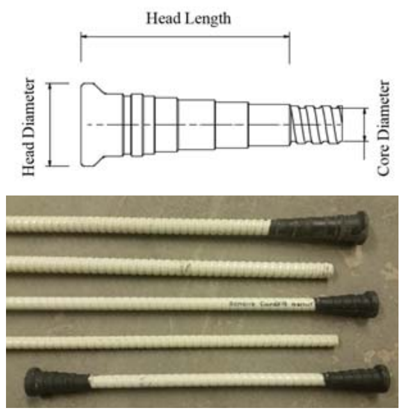
Figure 2. Dimensions and reinforcing details of the FRP-reinforced 8-panel girder.

2.1 Description and reinforcing details of the truss girders

All the four girders had the same cross-sectional dimensions with an overall depth of 1.32 m. The truss panel length was taken equal to 1.165 m allowing for a 45-degree angle of the diagonal CFFT members and span length of 2.33 m and 9.32 m, leading to span-to-depth ratio of 1.76 and 7.07 for the 2- and 8-panel girders, respectively. The girders were denoted G*i*-*j*Rd; where "G" refers to girder, i = 1, 2, 3 or 4 is the girder number, j = 2 or 8 is the number of truss panels in the girder, R indicates the type of reinforcement used, either "S" for steel or "F" for FRP, and d = 12 or 16 mm, respectively, is the nominal diameter of the steel or GFRP headed bars used in the diagonal CFFT truss members. Figure 2 presents the dimensions and reinforcing details of the 8-panel truss girder G4-8F16. Similar figures can be shown for the other three girders. In the FRP-reinforced girders, the chords were reinforced with 12 mm dia. GFRP stirrups and 16 mm dia. longitudinal GFRP double-headed bars arranged as shown in Figures 2a and c. In the steel-reinforced girders, the chords were reinforced with 10 mm dia. stirrups and 15 mm dia. longitudinal hooked bars. A minimum concrete cover of 30 mm to the stirrups was used. The bottom chords of the 2- and 8-panel girders, respectively, were pretensioned with six and fifteen 0.6 in. (15 mm) 7-wire low relaxation steel strands stressed to 75% of their ultimate tensile strength. Four strands were provided in the top chord to eliminate or control the tensile stresses and cracking due to local bending of the chord (Figure 2b).

Filament-wound GFRP tubes with 4 in. nominal diameter and approximately 70% circumferential and 30% longitudinal fibres were used for the truss web members. The tube inner diameter and wall thickness were 110 mm and 1.9 mm, respectively. The diagonal CFFT members were connected to the top and bottom chords by a bundle of four ½ in. (12.7 mm) dia. steel or 16 mm dia. GFRP long double-headed bars with the heads embedded in the chords and the stems extending through the member length (Figures 2a, c, and d). The verticals were connected to the chords by two ½ in. dia. steel or 12 mm dia. GFRP headed bars. Figure 3 shows the types of reinforcement used in the truss girders.

(a) Steel deformed headed bar



(b) GFRP reinforcing cage (c) GFRP bars with or without anchor head

Figure 3. Types of reinforcement used in the truss girders.

2.2 Material properties

2.2.1 Concrete

The target compressive strength of concrete used in all girders was 70 MPa at 28 days. The strength at release of pretensioning was specified at 45 MPa. 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 was used. The concrete 28-day compressive and tensile strengths as well as the compressive strength measured on the day of testing the girders are listed in Table 1. The values shown are the average of testing 3 cylinders.

Table 1. Concrete strength

Girder designation	Compressive strength at 28 days (MPa)	Tensile strength at 28 days (MPa)	Compressive strength on day of testing (MPa)
G1-2S12	67.0	4.8	71.3
G2-8S12	75.1	5.2	77.1
G3-2F16	71.4	5.3	73.2
G4-8F16	81.5	5.7	83.7

2.2.2 Glass FRP tubes

Filament-wound GFRP tubes with approximately 70% circumferential fibres and 30% longitudinal fibres were used for the vertical and diagonal truss members. The inner diameter and wall thickness were, respectively, 110 mm and 1.9 mm. The mechanical properties in the longitudinal and circumferential directions of the tubes were provided by the manufacturer in accordance with the ASTM test methods D2105 and D1599, respectively, and are summarized in Table 2.

Table 2. Mechanical properties of GFRP tubes

Tensile strength (MPa)		Compressive strength (MPa)	Tensile modulus (GPa)		Compressive modulus (GPa)	Poisson's ratio
Long.	Circum.	Longitudinal	Long.	Circum.	Longitudinal	
240	480	240	20.6	29	20.6	0.16-0.26

2.2.3 Reinforcement

Steel deformed double-headed bars of ½ in. dia. (Figure 3a) were used in connecting the CFFT truss members to the concrete chords of girders G1 and G2. Glass FRP stirrups of 12 mm dia. and double-headed bars of 12 mm and 16 mm dia. were used for reinforcing the concrete chords and for connecting the truss web CFFT members to the chords of girders G3 and G4 (Figures 2 and 3). Properties of this reinforcement as provided by the manufacturer are given in Table 3. The tensile properties of the steel reinforcement, and those of steel strands used for pretensioning the concrete chords (Figure 2b) are given also in Table 3.

Table 3. Dimensions and mechanical properties of reinforcement

Reinforcing type	Core diameter (mm)	Area (mm^2)	Head length (mm)	Head diameter (mm)	Short-term tensile strength (MPa)	Long-term tensile strength (MPa)	Long-term design strength (MPa)	Modulus of elasticity (GPa)
ϕ 12 GFRP headed bar	12	113	60	30	> 1000	580	445	60
ϕ 16 GFRP headed bar	16	201	100	40	> 1000	580	445	60
GFRP stirrup	11.6	106	NA*	NA	> 700	250	190	55
ϕ ½ in. steel headed bar	12.7	127	NA	40	503**	620	620	184
ϕ 15 steel bar	15.9	200	NA	NA	424**	704	704	187
ϕ 10 steel stirrup	11.3	100	NA	NA	485**	730	730	210
ϕ 0.6 in. steel strand	15.2	140	NA	NA	1792**	1943	1943	193

* NA = Not Applicable ** Yield or Proof Strength

2.3 Fabrication of the truss girders

The 2-panel girders were fabricated in the University's Structures Laboratory, whereas the 8-panel girders were fabricated in a local precast plant in Calgary and then shipped to the Structures Laboratory for testing. Figure 4 shows the fabrication process of the 2-panel girder G3-2F-16. The vertical and diagonal CFFTs were produced prior to the chords (Figure 4a). The GFRP tubes were cut to length and fixed in special wooden frames (Figure 4a). Two double-headed bars spaced at 68 mm centre-to-centre were fixed in position along the diameter of the vertical tube. Four double-headed bars were bundled and inserted in the diagonal tube with the heads protruding from the tube ends. The wooden frames were provided with templates to fix the headed bars in place and ensure that the centroidal axis of the bundle coincides with that of the tube. Figure 4b shows the formwork with the CFFT members and the reinforcing cages of the chords in place in preparation for pretensioning and casting the chords. As shown in Figure 4b, for ease of fabrication, the chords were cast in a 90-degree rotated position with the CFFT truss members placed in a horizontal plane. Figure 4c shows the girder after casting and removal from the form.

(a) Vertical and diagonal CFFT members after casting

(b) Chords ready for casting (c) Girder after casting

Figure 4. Fabrication of the 2-panel truss girder G3-2F16.

2.4 Test setup and instrumentation

Each girder was simply supported on two rollers in the loading frame. The girder was subjected to a vertical static load applied at the middle of the top chord using a 1.5 MN capacity hydraulic actuator. A bearing plate was placed on the girder top surface underneath the actuator shaft. The top chord was laterally supported at the location of the load and near the supports in order to restrain any out-of-plane displacement. Figure 5 shows the 8-panel girder G4-8F16 in preparation for testing.

Figure 5. Test setup of the 8-panel girder.

A load cell was placed between the actuator shaft and the bearing plate on top of the girder to measure the applied load. For the 2-panel girders, 3 laser transducers were placed below the bottom chord to measure deflection at mid span and at mid-length of each panel. For the 8-panel girder, 7 transducers were used to measure deflection at mid-span and at the location of each vertical member of the truss. Frame analysis of the 8-panel girder using the computer software CPF (El-Badry and Ghali, 1990) revealed that the 2nd and 7th diagonal members carried the highest tensile forces in the truss, and the 2nd, 5th (i.e. central) and 8th vertical members carried the highest compressive forces. Therefore, each double-headed bar in the two diagonals of the 2-panel girders, and the 2nd and 7th diagonals of the 8-panel girders, was instrumented with a strain gauge located at the interface of the member with the top and bottom chords. Mechanical transducers were used to measure elongation of the two critical diagonals and shortening of the three critical verticals in each girder. Strain gauges were also used at the outer surface of the FRP tubes of the three critical vertical members to measure their longitudinal and circumferential strains. Also, the bottom longitudinal reinforcing bars in the concrete chords were instrumented with strain gauges at mid span of each girder.

3 TEST RESULTS AND DISCUSSION

Each girder was tested under a point load applied in a displacement-controlled mode and increased monotonically from zero to failure. Figure 6a compares the load versus mid-span deflection curves of girders G1 to G4. As can be seen, the two 2-panel girders G1 and G3 exhibited close values of their respective ultimate loads and maximum deflections. The ultimate loads of the 8-panel steel-reinforced girder G2 and FRP-reinforced girder G4 were 616 kN and 978 kN, respectively. The maximum deflection of the two girders was very close to 145 mm. Each sudden drop in the load of girder G2-8S12 was due to fracture of the steel double-headed bars at a connection of a diagonal truss member to the top or bottom chord. The drop in load of girder G4-8F16 was due to crushing of the top chord concrete in compression (Figure 10c).

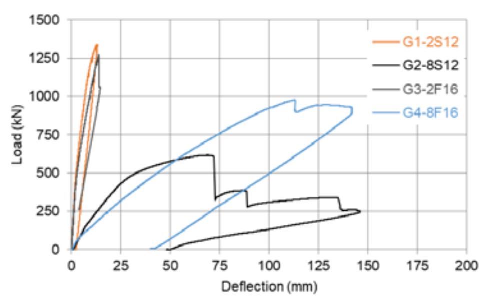


Figure 6. Load-deflection response of all girders.

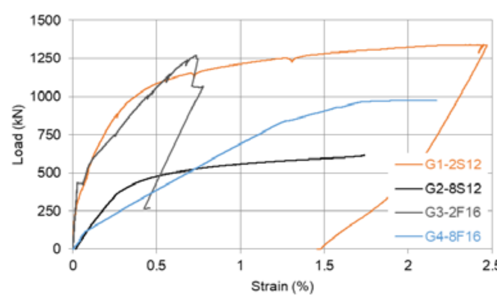


Figure 7. Load vs. maximum strain in headed bars.

Figure 7 compares the maximum strain in the double-headed bars connecting the diagonals to the concrete chords. The yield strain of the ½ in. steel double-headed bars is 0.35 per cent. From Table 3, the allowable design stress of the GFRP bars as recommended by the manufacture is 445 MPa (Schöck, 2015). The corresponding strain is 0.74 percent. From Figure 7, the loads at yielding of the steel headed bars in girders G1 and G2 are 73.5, 68.0 per cent of the ultimate loads, respectively. For the GFRP headed bars in girders G3 and G4, the loads at the allowable stress are 98.0, 54.4 per cent of the ultimate loads, respectively.

Figure 8 presents variation of the maximum shortening of the vertical and elongation of the diagonal truss members. In the 2-panel girders G1 and G3, the central vertical member experienced a much greater shortening than the two end verticals, whereas in the 8-panel girders G2 and G4, the

8th vertical exhibited the greatest shertening. At the same load level, shortening of the verticals reinforced with GFRP headed bars was greater than that of the steel-reinforced verticals. This is attributed to the lower stiffness of the GFRP bars. The ultimate failure of girders G1 and G3 occurred in the central vertical member due to crushing of the concrete core followed by rupture of the GFRP tube. This explaines the significant shortening at failure as shown in Figure 8. The diagonal members in the steel-reinforced girders G1 and G2 experienced significant elongation when yielding occurred in the steel double-headed bars.

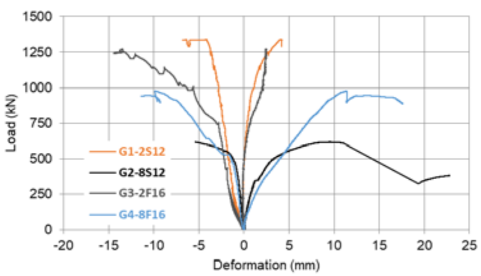


Figure 8. Deformations of the truss members

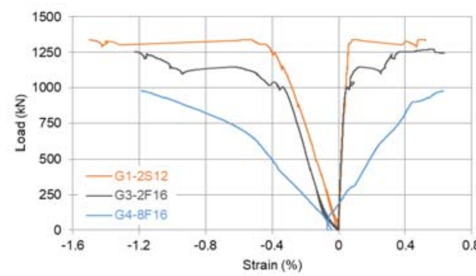


Figure 9. Maximum strains in the vertical tubes

Figure 9 shows variation of the axial and circumferential stains in the GFRP vertical tubes with the applied load. The strain gauges on the tube of the 8-panel girder G2 reinforced with steel malfunctioned during the test and did not yield reasonable results. The maximum strains shown in the figure for the 2-panel girders G1 and G3 occurred in the central tube, at the bottom in girder G1 and at mid-height of the tube in girder G3. The two curves shown for girder G4-8F16 are both for the vertical strains in the 2nd vertical tube in the truss. The top of the tube experienced compressive strain throughout the test, while the bottom was subjected to compression at low load levels, then to tension, indicating bending of the tube with the increase in load and deflection of the girder.

Failure of the 2-panel girders G1 and G3 occurred by crushing of the core concrete under compression and then rupture of the GFRP tube of the central vertical member and (Figure 10a). In girder G3, this was followed by crushing of concrete in compression in the top chord. The steel-reinforced 8-panel girder G2-8S12 failed by sequential fracture of the headed bars at the bottom connection of the 2nd diagonal, followed by the bars at the top connection of the 1st diagonal (Figure 10b), and then the bottom connection of the 3rd diagonal from the support. Failure of the GFRP-reinforced 8-panel girder took place by crushing of concrete in compression at mid-span in the top chord (Figure 10c). This was accompanied by a sudden small drop in the load as shown in Figure 6. No sign of fracture of the GFRP headed bars connecting the diagonal CFFT truss members to the top and bottom chords.

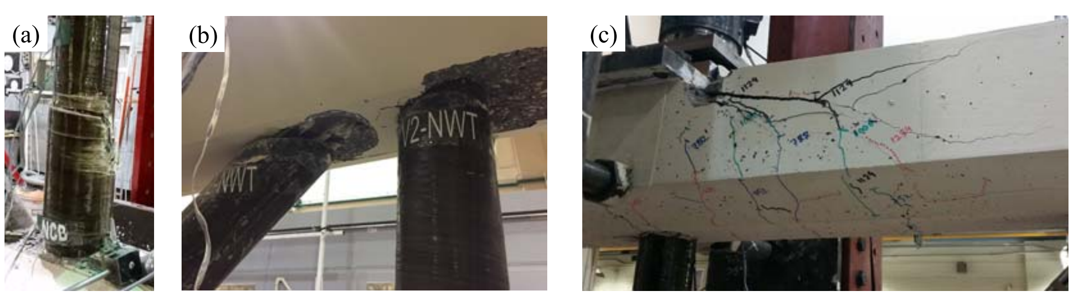


Figure 10. (a) Rupture of the vertial GFRP tube; (b) Fracture of the diagonal headed bars; (c) Crushing of concrete in the top chord

4 SUMMARY AND CONCLUSIONS

A novel bridge system consisting of a concrete deck on top of precast prestressed hybrid FRP-concrete truss girders has been developed. The vertical and diagonal truss members are made of concrete-filled GFRP tubes connected to concrete chords by means of GFRP double-headed bars. The hybrid system is light in weight and durable, and hence, economical and cost effective. Fabrication of the girder is practical and efficient. The results of testing four girders under static loading have been presented. Two of the girders comprised two truss panels and had low span-to-depth ratio that made them shear critical. The other two girders consisted of eight panels each and had higher span-to-depth ratio, and thus were critical in flexure. The following conclusions are drawn from the test results:

1. Despite the lower stiffness of the GFRP reinforcement than that of steel, the maximum load sustained by the GFRP-reinforced 2-panel girder was as high as that sustained by the steel-reinforced similar girder.
2. The ultimate load and failure of the 2-panel girder were governed by crushing of the concrete core, followed by rupture of the GFRP tube of the central vertical truss member.
3. The ultimate load of the steel-reinforced 8-panel girder was governed by yielding and strain hardening strength of the double-headed bars connecting the diagonal truss members to the top and bottom chords. Failure occurred by fracture of the headed bars at three connections.
4. The GFRP-reinforced 8-panel girder sustained a load 60 per cent than the steel-reinforced 8-panel girder. Failure occurred only by crushing of concrete in compression in the top chord.

5 ACKNOWLEDGEMENTS

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