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EXPERIMENTAL EVALUATION OF SHEAR BEHAVIOR IN FRP STRENGTHENED CONCRETE BRIDGE GIRDER

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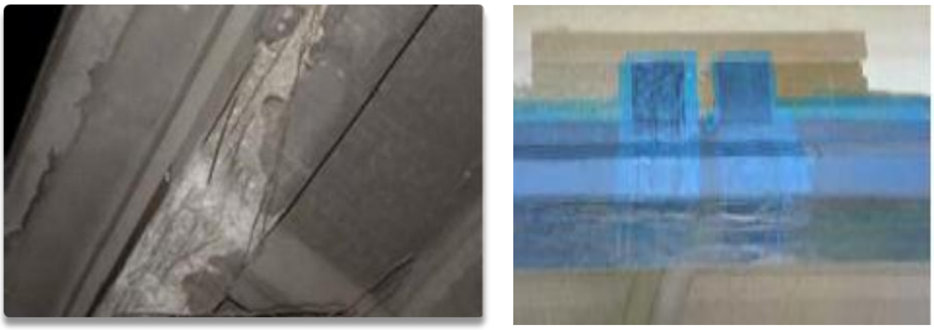
ABSTRACT: External Fiber Reinforced Polymer (FRP) wrapping is an increasingly popular technique for the rehabilitation/strengthening and confinement of damaged concrete bridge girders. There is a general lack of knowledge on the full-scale experimental evaluation of FRP wrapped girders. This is especially true for the shear behavior of concrete girders with vertical FRP application. In this study, a full-scale standard Texas bridge I-girder with typical prestressed and non-prestressed steel was experimentally evaluated for three-point shear load capacity, and concrete cracking. A common unidirectional carbon FRP (CFRP) in the form of U-wraps was utilized. Load was applied near one support to induce maximum shear force in the girder. The U-wrapped FRP increased the shear capacity of the girder by about 11%. Typical flexural-shear cracking was observed. The crack initiation and propagation was delayed by the presence of CFRP layers. The girder behaved in an elastic manner during the shear loading phase. Difficulty with FRP U-wrap application in the field may decrease the quality of the FRP-concrete bond and shear contribution. Use of additional mechanical or other types of anchoring for the FRP U-wrap could be useful in delaying any premature FRP debonding.

KEYWORDS: Concrete bridge girders; Flexural strengthening; FRP wrapping; I-Girders; U-wrap.

1 INTRODUCTION

The infrastructure report card for U.S.A. states that over 11% of the nation’s 607,380 bridges are structurally deficient and an estimated $20.5 billion is required annually to upgrade the nation’s deficient bridges by the year 2028 [1]. However, the current annual expenditure for bridge investments is only $12.8 billion and an additional $8 billion is required annually to upgrade the nation’s deficient bridges. Feasible bridge retrofitting and rehabilitation is, therefore, a viable option for upgrading deficient bridges, address budget constraints and reduce construction times. Fiber Reinforced Polymer (FRP) strengthening is one such method that can increase the life of a bridge and reduce the cost for replacement. This has been a popular, economic and convenient method for restoring and enhancing the strength and stiffness of damaged concrete bridges since 1999 [2], [3].

A total of 24 highway departments are currently using FRP laminate application as a bridge retrofitting technique. The Texas Department of Transportation (TxDOT) is currently maintaining over 30,000 bridges, most of which include precast prestressed concrete girders and cast in place concrete piers. Each year, a number of these girders are damaged due to vehicle impact, rebar corrosion and structural deterioration. Over height vehicles collision due to low clearance of older bridges or increase of roadway overlay thickness is the primary cause of the first type of damage [4]. The damage may be minor, or could be severe that includes spalling of concrete and damage or breakage of prestressing strands, as seen in Figure 1a. Therefore, for severe cases, the damaged girders are repaired and strengthened as soon as possible to maintain the safety and load carrying capacity of the girders. Among other repair methods, TxDOT has been using external strengthening of damaged concrete bridge girders with fiber reinforced polymer (FRP) fabrics since 1999. More than 30 such bridge strengthening has been performed in the state to date. The FRP strengthening is typically applied after any damaged prestressing strands are spliced and re-tensioned, and the spalled concrete replaced with new repair concrete (Figure 1b). Besides providing additional strength, the FRP strengthening adds confinement to the concrete, prevent significant spalling of concrete in case of future damage at the same location, and increase the durability.



(a). Damaged *(b).* Repaired

Figure 1. Damaged and repaired prestressed concrete girder

Several design guides, standards and manufacture's guidelines are available worldwide for the design and analysis of FRP strengthening systems for concrete structures. Some of these provisions are based on theoretical models, while others are based on experimental work. In the U.S.A., the primary design/analysis source is the guidelines published by Committee 440 of the American Concrete Institute. The FRP strengthening design procedure for TxDOT projects is based on ACI 440 provisions [5]. In ACI 440, the design recommendations are based on limit state method and strength/serviceability requirements. Additional load factors are applied to the contribution of the FRP reinforcement. These factors were determined based on statistical evaluation of variability in mechanical properties, predicted versus full-scale test results, and field evaluations. These provisions were developed based on theoretical analysis and testing on small scale concrete samples. The following failure modes are considered in the shear strength determination by the ACI approach: (1) crushing of concrete before yielding of the reinforcing steel; (2) yielding of shear steel followed by rupture of the FRP laminate; (3) yielding of the shear steel followed by concrete crushing; (4) shear/tension delamination of the concrete cover; and (5) debonding of the FRP from the concrete substrate. Although Mode 2 is preferred in order to fully utilize the strengths of both the prestressing steel and the FRP laminate, laboratory testing to date has only shown this mode as a possibility for beams with proper detailing. In most cases, Mode 4 has controlled the failures.

Only a limited number of prior studies on FRP strengthened full scale concrete bridge girders were located herein. An experimental study by Elsafty and Graeff [6] involved half scale AASTHO Type II girders and composite decking that was flexurally strengthened with two or three layers of carbon FRP (CFRP). CFRP strips were also used as transverse U-wraps for shear strengthening that extended along the girder web. Eight strengthened girders were tested under a four-point static loading, five with two layers of flexural CFRP strengthening and the remaining three with three layers of flexural CFRP. Most of the girders failed by the CFRP rupture mode, which is the preferred mode of failure according to the design guidelines. Debonding problems did not allow successful achievement of the full strength of FRP.

Rosenboom [7] studied two different loading conditions on 30 prestressed bridge C-channel girders: extreme loading simulated by a monotonic load to failure, and fatigue loading designed to simulate service loads. An analytical model was presented which predicts the flexural behavior of the system assuming certain failure modes. Twenty one girders were part of a FRP strengthening study, five part of a repair study, and four part of a FRP bond study. A majority of the specimens failed in concrete crushing and other in intermediate crack debonding and rupture of CFRP. The flexural capacity of the girders increased by as much as 73% with the use of externally bonded CFRP sheets. Transverse CFRP U-wraps delayed debonding failures, and they increased the tensile strain in the CFRP at intermediate crack debonding failure by as much as 22%.

Another study by Ekenel and Myers [8] examined crack repair in concrete beams by epoxy injection and CFRP strengthening for increased stiffness. The flexural capacity increased due to the CFRP strengthening. Crack injection provided an increase in stiffness in the linear region of the load–displacement curves for all beams without CFRP strengthening.

Ghosh and Karbhari [9] studied the damage progression in the bridge deck slabs and girders under simulated truck loads. The ACI 440 based debonding strains were found to be un-conservative; it was suggested that a fracture based approach be used instead. The use of anchors was found to be critical for FRP shear strengthening of girders.

An experimental study looked at a damaged prestressed bridge girder repaired with CFRP [10]. The longitudinal CFRP sheets restored a portion of the flexural strength lost due to damage. Transverse CFRP sheets assisted in the development of the longitudinal CFRP sheets and prevented debonding, and significant deflection reduction (20%) was noticed.

Mohanamurthy and Yazdani [11] reviewed and compared flexural strength predictions in FRP strengthened concrete bridge girders from relevant design guidelines and standards, both from U.S.A. and abroad. FRP rupture was considered as the preferred failure mode in validating both theoretical and experimental analysis. The design guidelines were found to be quite conservative.

A single span reinforced concrete T-beam bridge in South Troy, Rensselaer County, New York, built in 1932, was strengthened for shear with externally bonded FRP laminates in November, 1999 [12]. Under service live loads after the FRP was installed, main rebar stresses were moderately reduced, concrete stresses (flexural and shear) moderately increased, and transverse live load distribution to the beams slightly improved. Although the FRP participated in load carrying, strain compatibility was not satisfied at some locations, attributed to the level of precision in strain measurements and/or a lack of full bond development at the time of testing. As expected, after the FRP installation, the neutral axis migrated downwards, but the effective flange width remained almost unchanged for all truck load positions.

The John Hart Bridge in Prince George, British Columbia, the Maryland Bridge in Winnipeg, Manitoba, and the Langevin Bridge in Calgary, Canada, have been strengthened with externally boned CFRP. The ease of handling and placement of the CFRP materials resulted in reduced construction time, when compared with other conventional repair techniques.

It is apparent from the above review that prior full-scale experimental investigation of external FRP strengthened concrete bridge girders is limited. Most of the prior studies dealt with flexural FRP strengthening. The behavior of such girders with FRP shear strengthening needs further investigation, and full scale experimental evaluation is the most appropriate and realistic approach for this purpose. The current paper presents an experimental study that involved the shear testing of a full scale TxDOT standard bridge girder with representative span lengths, and with FRP U-wrap strengthening. Test results, such as maximum load capacity, deflection and strains were analyzed herein, and compared with codal provisions. The comparison sheds important light on the FRP laminate performance on the full-scale representative bridge girder and the validity of the ACI 440 and AASHTO LRFD provisions for the design of the FRP strengthening system for such girders. It was found that the AASHTO [13] provisions were almost identical to the ACI 440 provisions. As this study involved a full scale bridge girder, the strengthened FRP system is close to the actual systems in the field.

2 TESTING PROGRAM

A standard TxDOT TX-28 girder was used in this experimental program. This girder is generally utilized in medium to short span applications. The section properties are given in Table 1 and the section shown in Figure 2. The Girder was 10.1 m long and assumed to be simply supported. The girder steel (both flexural and shear) was designed using the Load and Resistance Factor (LRFD) design standards from TxDOT [14]. The TxDOT provisions are mostly similar to the AASHTO LRFD Bridge Design provisions [15]. A software published by TxDOT, PGSuper, was utilized for the design purposes [16]. The TxDOT customized version of PGSuper is a windows-based software for the design, analysis, and load rating of multi-span precast-prestressed bridge girders in accordance with the AASHTO LRFD specifications and TxDOT design guidelines. The basic Tx28 girder was designed herein with the PGSuper software. The design included the prestressing steel, stirrups and other secondary reinforcement. PGSuper currently does not include the design of FRP strengthening for bridge girders. Therefore, ACI 440 provisions were utilized herein to calculate the shear capacities that FRP application provided to the basic TX28 girder with the steel arrangement found from PGSuper.

Table 1. Tx28 girder section properties

Area, m^2	Strong axis moment of inertia, m^4	Weak axis moment of inertia, m^4	Weight, kN/m
0.37	0.021	0.017	9.19

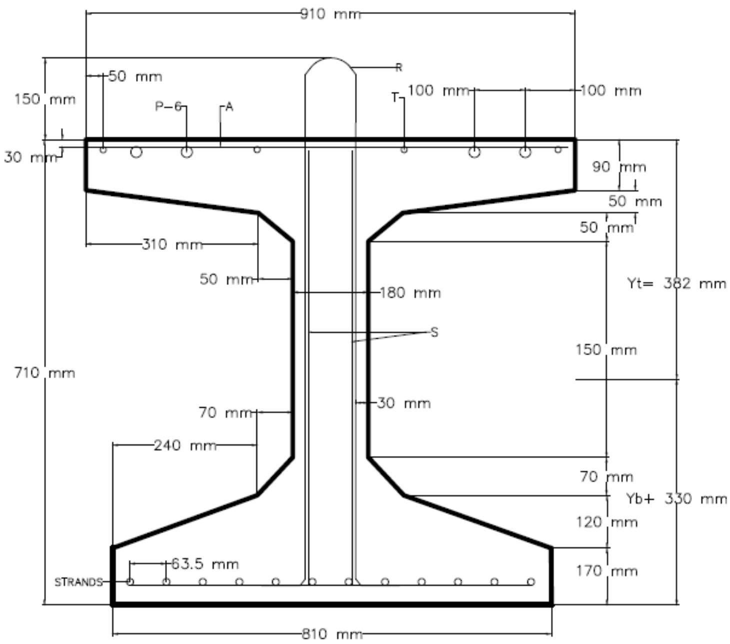
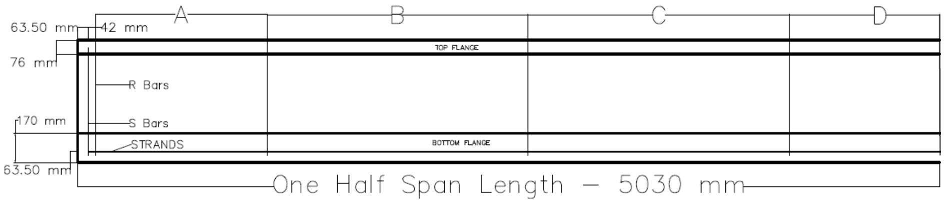


Figure 2. TX28 Girder Cross Section

The assumed design concrete compressive strength for the girders, as used by the precast manufacturer, was 34.5 MPa. This value was used in designing all steel (mild and prestressed) for the girders, according to PGSuper. However, the actual concrete strength at the girder testing time (at seven months after girder casting) was found to be 79.3 MPa from concrete cylinder testing, and this value was used in this study to calculate the three point load capacity of the girders. The prestressing steel was low-lax 12.5 mm diameter strands with 1862 MPa ultimate tensile strength. All secondary steel (including U.S. #3 stirrups) was assumed to have a yield strength of 410 MPa. The girder steel design is shown in Figures 2 and 3. The girder was produced at a precast yard at Waco, Texas, using steam curing, and then transported to the structural laboratory at UT Arlington for testing.



A – 13 Spacing @ 76.20 mm
B – 15 Spacing @ 101.20 mm
C – 10 Spacing @ 152.40 mm
D – 4 Spacing @ 203.20 mm

Figure 3. Girder elevation

3 CFRP AND EXPOXY

The TxDOT [14] guidelines for FRP strengthening of concrete members contain an approved list of FRP and epoxy for bridge projects. A common unidirectional 609 mm wide CFRP from this list, produced by a well know international manufacturer, was utilized in this study. This fabric is laminated using a compatible epoxy from the same manufacturer to form the composite laminate used to strengthen the concrete elements. The epoxy was a two-component, moisture-tolerant, high strength, high modulus type, approved. The relevant properties of the selected CFRP and epoxy are presented in Table 2.

Table 2. Properties of CFRP matrix and epoxy

Cured Laminate Properties for CFRP	Design Values
Tensile Strength	724 MPa
Modulus of elasticity	53567 MPa
Elongation at break	1%
Thickness	0.5 mm
Properties of Epoxy	
Tensile Strength	72.4 MPa
Tensile Modulus	31716 MPa
Elongation at break	4.8%
Flexural Strength	123.4 MPa

The CFRP applying scheme is shown in Figure 4. Two adjacent layers of vertical CFRP U-wraps were applied near each end of the girder as shown, corresponding to the maximum shear force region in the girder for a three-point loading set-up. The load was applied at 0.91 m at one end of the girder for inducing maximum shear force in the girder.

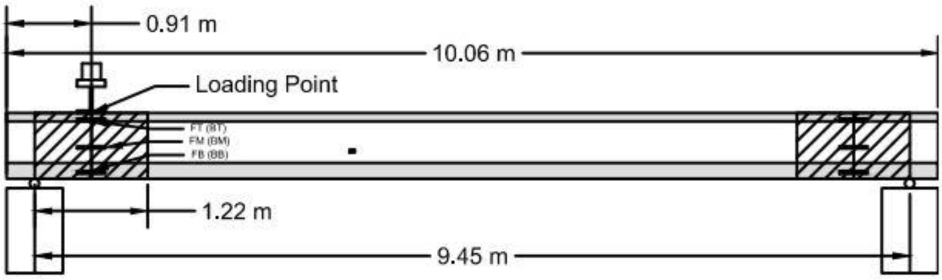


Figure 4. Girder CFRP layout, strain gage layout and load application

4 CFRP APPLICATION

Proper surface preparation in the form of roughening and cleaning is essential for adequate bonding and performance of the CFRP-epoxy-concrete matrix. In order to ensure adequate surface preparation, a well-known national company for FRP application on concrete, and used frequently by TxDOT for bridge projects, was hired for the surface preparation and FRP application. The crew members roughened all girder surfaces that would have FRP application with a hand-held grinder, as per the CFRP manufacturer specifications. The surfaces were then cleaned using compressed air and brushes to remove the dust. The girder surfaces were protected from moisture intrusion by covering with tarpaulin wraps. Following the manufacturer's specification, the two-part epoxy was mixed with a hand held mixer. A fine silica was added to the mix as specified, which helps in filling the concrete surface pores. After epoxy mixing, rollers were used to apply the epoxy on the concrete surface and also on the pre-cut lengths of the CFRP. The described methodology is typically used in field application of FRP on bridge projects.

Thereafter, the saturated CFRP was installed on the girder surface. Special care with hand pressure was taken to smooth out any voids under the CFRP and ensuring a good installation. A minimum 48 hours curing time was allowed before load testing. No additional mechanical or other types of anchoring on the U-wrap (e.g. mechanical) currently required by TxDOT and they were not provided.

5 INSTRUMENTATION

A composite linear strain gage, suitable for measuring strains in FRP composites, was used on the test girder. Each gage was 2 mm in length, 0.9 mm in width and had 120 ohm resistance. The strain gages were installed according to the manufacturer's specifications, using an epoxy adhesive. Following symbols shown in Table 3, gages FT, FM, FB, BT, BM, BB were used for the compression, tension and shear strains and to verify the loading/strain symmetry. A strain gage scanner was used to collect strain data with each element connected as a quarter bridge. Four Two Linear Variable Differential Transducers (LVDT) were used to monitor the vertical displacements: one at each support, one at 6.5' from the end of the girder and one directly below the loading point.

Table 3. Designation of strain gages

FT – Bottom of top flange, front	BT – Bottom of top flange, back
FB – Top of bottom flange, front	BB – Top of bottom flange, back
FM – Mid web, front	BM – Mid web, back

6 BOND BEHAVIOR OF FRP – CONCRETE INTERFACE

The strengthening of concrete structures by using FRP system depends mainly on the interface bond between the FRP sheets and concrete surface. The bond quality is affected by any lack of proper surface preparation, epoxy mixing, epoxy application and the FRP application onto the concrete surface. Inadequate bonding results in inadequate stress transfer and resulting insufficient composite action. One popular method to spot check the bond quality is the ASTM Pull-Off Testing [17]. In this test, a 50 mm circular dolly is glued to the CFRP surface. After 24 hours of curing, the dolly is pulled off from the surface with a tension tester (Figure 5a). If the failure surface is within the concrete substrate, a good bond quality is indicated. A failure in the CFRP or epoxy indicates inadequate bond quality. In the current study, three random pull off tests were performed on the CFRP. All samples showed adequate ASTM acceptability with failure in the concrete substrate, indicating good workmanship and bond quality (Figure 5b).

(a). Tester application

(b). Failed samples showing good bond

Figure 5. Pull-off testing

7 LOAD TESTING

Two 0.91 m high concrete pedestals were used as support blocks for the girder load testing. A steel W-shape and plates were used to apply the point load uniformly along the width of the girder (Figure 6). To simulate simple supports, a steel roller and plate assembly was used at each end. The point load was applied at 0.6 m from one support center, or 0.91 m from the girder end to induce maximum shear force on the section (Figure 4). A load cell with 2669 kN capacity was used, while the steel frame supporting the load cell was capable of carrying a load of 1780 kN. A hydraulic pump was used to manually apply load through the load cell with a loading rate of 22 kN per step and one step per two seconds until the cracking load was observed. Cracks were monitored regularly and marked on the girder surfaces. Once the cracking load was reached, the loading rate was reduced to 9 kN per step until the load cell capacity was reached. The testing was stopped when the load reached a value of 1690 kN for safety. All test output, such as load, strain and deflection were automatically collected through a data acquisition system.

Figure 6. Load application

8 STRENGTH ANALYSIS

Hand calculation was used herein to determine the expected theoretical shear capacity of the girder. The analysis was based on the shear steel, concrete and CFRP strengths, and also on the cross-sectional properties, steel layout (both prestressed and shear) and the CFRP layout, as discussed previously. AASHTO LRFD Specifications were used herein for the strength calculation [13]. The

AASHTO provisions currently do not cover FRP strengthening design provisions. Therefore, provisions from the ACI 440 guidelines were used to calculate the shear contribution from the CFRP [5]. The basic shear strength equation is as follows:

$$V_n = V_c + V_s + V_p + \psi V_f \quad (1)$$

where:

V_n = nominal total sectional shear capacity
V_c = concrete shear capacity (AASHTO Eq. 5.8.3.3-3) = 658 kN
V_s = steel shear capacity (AASHTO Eq. 5.8.3.3-4) = 814 kN
V_p = shear contribution of any harped prestressing strands = 0
V_f = CFRP shear capacity (ACI Eq. 11-3) = 163 kN
ψ=a reduction factor for three-sided CFRP U-wrap (0.85 from ACI 440)

The AASHTO LRFD equations used herein are based on the Simplified Shear Design Approach. Because the girder used all straight prestressing strands, there was no shear contribution from them. Because of the controlled conditions under which the girder was constructed and tested, no load factors or resistance factors were applied. Manufacturer's data (Table 2) for the selected CFRP laminate was used in the calculation of the CFRP contribution.

9 RESULTS AND DISCUSSION

Based on Eq. 1, the various shear strength contributions were calculated as follows: Vn = 1610 kN, Vc = 658 kN, Vs = 814 kN, Vp = 0, and Vf = 163 kN. It is noted that the shear strengthening with CFRP U-wrapping is expected to increase the shear capacity of the girder by about 11%. The CFRP contribution could be useful in cases where: (1) the steel stirrups are insufficient in providing the needed shear resistance, (2) the concrete contribution is limited due to low concrete strength and/or small cross-section, and (3) increasing the steel stirrup size or decreasing the stirrup spacing is impractical due to design constraints or increased steel congestion. The results from the girder shear testing are presented in Table 4. The capacity of the loading mechanism was 1780 kN which was also close to the maximum shear force applied near the support. It is apparent that, although the girder theoretical shear capacity was about 10% less than the maximum applied shear, the girder did not fail and the CFRP layers were intact without any apparent debonding. The shear behavior of the test girder and the associated CFRP behavior were well demonstrated during the load testing.

Table 4. Shear capacity and deflection results

Sectional shear capacity near support (kN)	Maximum load applied (kN)	Maximum Deflection (mm)
1610	1690	18.8

9.1 Crack initiation and propagation

Figure 7 shows photographs of crack patterns in the girder at the maximum applied load. Cracks first appeared in the CFRP at 454 kN load. With increased loading beyond first cracking, the number of cracks and the crack widths increased. Some cracks changed from vertical flexural type to inclined flexure-shear type at some distance from mid-span. It was difficult to locate cracks under the CFRP layers near the load application area at the support. Mots cracks demonstrated a flexure-shear type behavior.

Figure 7. Cracking in girder

9.2 Crack initiation and propagation

Figure 8 shows the load versus the maximum vertical deflection in the test girder. The deflection readings were from the LVDT placed at the location of the expected maximum girder deflection, based on mechanics of shear behavior (1.98 m from the loaded support). The curve shows an almost linear load-deflection relationship until the end of testing. It is obvious that the test girder was not close to failure at the end of testing. None of the five possible failure modes described by ACI 440 was demonstrated in the testing program. There was no apparent FRP or concrete cover delamination, or concrete crushing or prestress yielding. Apparently, the test girder had significantly greater overall shear capacity than what was predicted by the design codes. The maximum strains in Table 5 show that the compressive strains at the top flange (gages FT, BT) were well below the generally assumed concrete crushing strain of 0.003.

The maximum tensile FRP strains (0.000006) occurred near the loaded support location, where the shear force is maximum, as expected (for gage B2). However, this value is many times less than the expected tensile failure strain of 0.001 for FRP rupture.

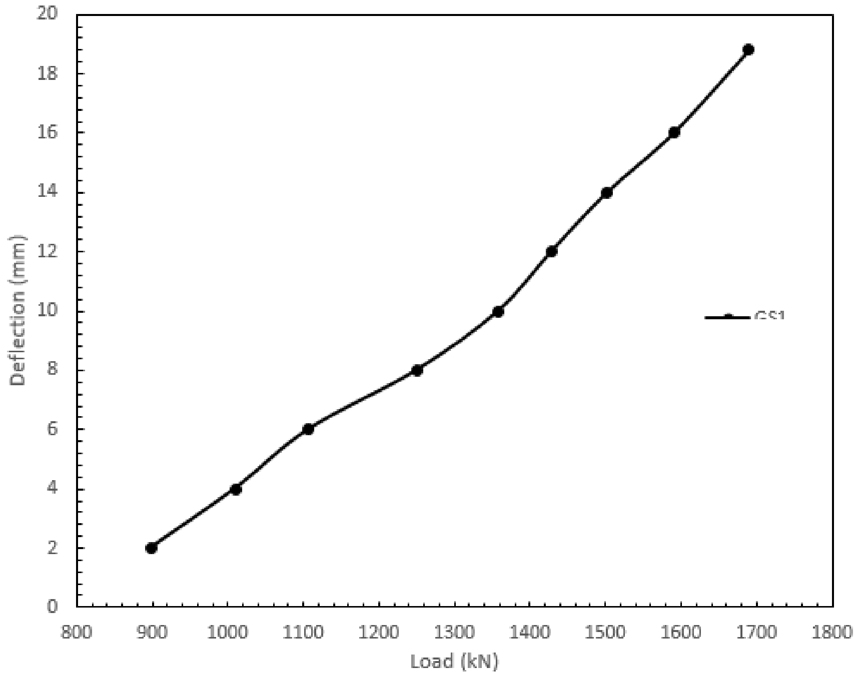


Figure 8. Load vs. maximum deflection plot

Table 5. Maximum strains from strain gages

Strain Gage	**Microstrain** (C: compression; T: Tension)
FT	-3.02 (C)
BT	-2.12 (C)
FM	5.77 (T)
BM	5.39 (T)
FB	6.07 (T)

10 CONCLUSIONS

The following conclusions may be made based on the findings from this investigation:

a) Adequate experimental investigation on the shear behavior of FRP strengthened full-scale prestressed concrete bridge girders in order to determine the expected strength gain and the FRP performance is lacking.
b) Shear FRP strengthening in the form of vertical U-wrapping is helpful for enhancing the performance of a concrete bridge girder. The benefits include about 11% increase in shear load capacity.
c) Shear cracking in FRP strengthened girders initiate as flexural cracks and subsequently change into diagonal tension cracks, as expected. It is difficult to locate concrete cracks that are covered by the FRP laminate during the experiment.
d) U-wrapping with transverse FRP is clearly beneficial in delaying crack initiation and propagation, increased load carrying capacity, delaying FRP failure and increased ductility.
e) The maximum achieved FRP tensile strains were quite less than that expected for the FRP rupture strain.
f) Field FRP applications in actual concrete bridges can be challenging with deteriorating concrete, overhead applications, hard to reach areas below the bridge deck and working around traffic flow. The current study involved FRP application in a laboratory setting with controlled circumstances, where the FRP application quality was expected to be better than the field application. There is a need to emphasis the minimum FRP application quality specified by guidelines and manufacturers.
g) The relevant guidelines do not currently address any beneficial effect of mechanical anchoring of FRP U-wrapping on concrete bridge girders. Mechanical anchoring of FRP U-wrapping is not a factor in the design procedures from the guidelines.

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