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Experimental Evaluation of FRP Strengthened Concrete Bridge Girders for Flexure

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***Abstract*—External Fiber Reinforced Polymer (FRP) fabric strengthening is an increasingly popular technique for strengthening and confinement of damaged concrete bridge girders. There is a general lack of knowledge on the full-scale performance of FRP strengthened girders. In this study, three full-scale girders were experimentally evaluated for flexural load capacity, failure mode, cracking and ductility. A unidirectional carbon FRP with one and two layers application, and U-wrap anchoring, was employed. Flexural FRP was found to increase the first cracking load by 7-14%, the ultimate load by 6-7% and the ductility by up to 25%. FRP U-wrap anchoring was instrumental in delaying cracking, increasing the ultimate load, delay FRP debonding and increase ductility. FRP debonding at the epoxy layer was the controlling failure mode, as opposed to the FRP rupture or debonding in the concrete substrate. Tested girders failed to reach the high predicted FRP debonding and rupture strains, possibly due to inadequate surface preparation using industry accepted practice.**

***Keywords*— FRP stregnthening; concrete bridge girders; FRP wrapping; flexural strengthening; U-wrap; experimental evaluation**

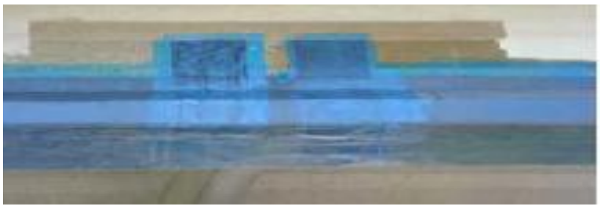
I. INTRODUCTION

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 (ASCE [1]). 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 (Yang [2], Ganga [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 (Miller [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 Fig. 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 (Fig. 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.

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 (ACI [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 flexural strength determination: (1) crushing of concrete before yielding of the reinforcing steel; (2) yielding of tension steel followed by rupture of the FRP laminate; (3) yielding of the tension 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.

(a) *Damaged.*



(b) *FRP strengthened.*

Fig. 1. *Damaged and FRP strengthened pre-stressed concrete bridge girders.*

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 (Klaiber [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 (Hag-Elsafi [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. 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 that prior studies mainly focused on field investigations, analytical investigations or experimental work dealing with failure modes and the role of U-wrapping. Comparison with applicable code provisions, the effect of concrete surface preparation and the effect of number of FRP layers vs. the presence/absence of U-wrapping were not investigated. The current paper presents an experimental study that addresses these issues through the flexural testing of three full scale TxDOT standard bridge girders with representative span lengths, and with FRP strengthening. Test results, such as maximum load capacity, failure mode and strain, are compared with calculations based on the ACI 440 provisions. The comparison sheds important light on the FRP laminate performance on full-scale representative bridge girders (including FRP laminate layer numbers and U-wrapping), the effect of surface preparation, the expected mode of failure in flexural strengthening cases and the validity of the ACI 440 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 full scale bridge girders, the strengthened FRP system is close to the actual systems in the field.

II. MATERIALS AND METHODS

A total of three standard TxDOT TX-28 girders were used in this experimental program. The girders are generally utilized in medium to short span applications. The section properties are given in Table I and the section shown in Fig. 2. Each 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 (TxDOT [14]). The TxDOT provisions are mostly similar to the AASHTO LRFD Bridge Design provisions (AASHTO [15]). A software published by TxDOT, PGSuper, was utilized for the design purposes (PGSuper [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 bending capacities that FRP application provided to the basic TX28 girder with the steel arrangement found from PGSuper.

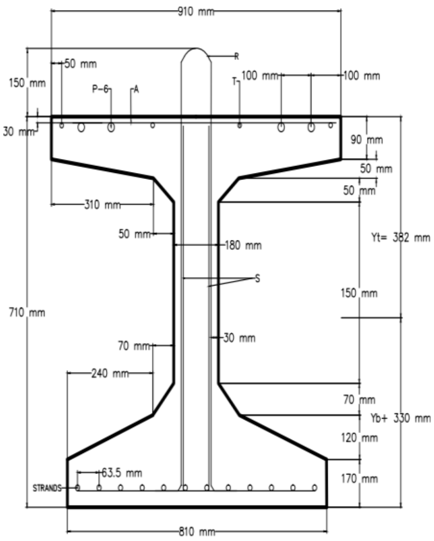


Fig. 2. *Tx28 girder cross section.*

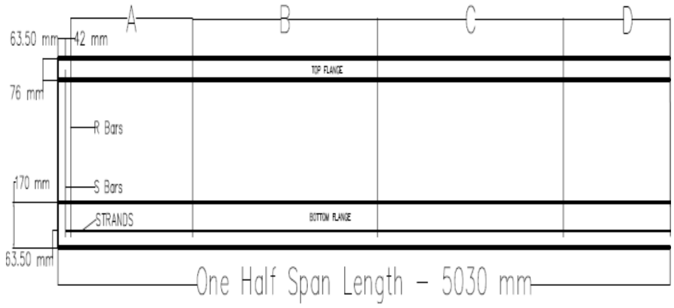
The assumed design concrete compressive strength for the girders, as used by the precast manufacturer, was 58.6 MPa. The corresponding concrete mix design is presented in Table II. This value was used in designing all steel (mild and prestressed) for the girders, according to PGSuper. However, the actual concrete strength (found from the average of three 150 by 300 mm cylinder testing) 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 (Table VI). 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 Figs. 2 and 3. The girders were produced at a precast yard at Waco, Texas, using steam curing, and then transported to the structural laboratory at UT Arlington for testing.

TABLE I. 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

TABLE II. CONCRETE MIX PROPORTIONS FOR THE GIRDERS PER CUBIC METER OF CONCRETE

Components	Weight
Type III cement	195.60 kg
Fly Ash-Class F	65.20 kg
Coarse aggregates	582.97 kg
Fine aggregates	489.67 kg
Admixture: Water reducing admixture Set retarding concrete admixture	 1.30 kg 0.086 kg
Water	78.37 kg



A – 13 Spacing @ 76 mm.

B – 15 Spacing @ 101 mm.

C – 10 Spacing @ 152 mm.

D – 4 Spacing @ 203 mm.

Fig. 3. *Girder elevation.*

A. CFRP and Epoxy

The TxDOT [17] guidelines for FRP strengthening of concrete members contain an approved list of FRP and epoxy for state 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 is a two-component, moisture-tolerant, high strength, high modulus type, approved. The relevant properties of the selected CFRP and epoxy are presented in Table III.

TABLE III. PROPERTIES OF CFRP AND EPOXY.

Cured Laminate Properties for CFRP	**Design Values**
Tensile Strength	724 MPa
Modulus of elasticity	56500 MPa
Elongation at break	1%
Thickness	0.51 mm
Properties of Epoxy	
Tensile Strength	55 MPa
Tensile Modulus	1724 MPa
Elongation at break	3%
Flexural Strength	79 MPa

B. 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 herein 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.

The bond capacity of FRP is developed over a minimum critical length. To develop the effective FRP stress at a section, the available anchorage length of FRP should exceed the value of 80 mm given by the relevant equation in ACI 440.2R [5]. For FRP systems, a lap splice should be made by overlapping the fibers along their length. The required overlap, or lap-splice length, depends on the tensile strength and thickness of the FRP material system and on the bond strength between adjacent layers of FRP laminates. A 150 mm overlapping extension was provided for the second FRP layer in girder GF2.

Thereafter, the saturated CFRP was installed on the girder surface (Fig. 7). 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.

To determine the effect of clamping on flexural CFRP debonding, U-wraps were applied on both ends of the flexural layers and also near the center of girder GF1. These U-Wraps provided additional shear capacities along with the clamping action. No additional anchoring on the U-wrap (e.g. mechanical) is currently required by TxDOT and they were not provided.

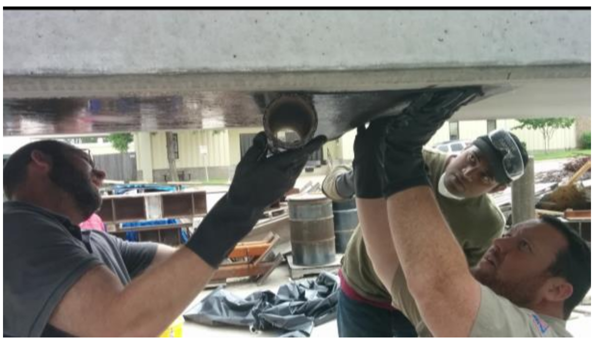


Fig. 7. *Saturated CFRP application.*

C. Instrumentation

A composite linear strain gage, suitable for measuring strains in FRP composites, was used on the test girders. 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. For girders GC and GF1, a total of seven gages were installed, one each at the junction of the girder web and flange at the top and bottom, and three at the bottom surface, as shown in Figs. 5 and 6. Following symbols shown in Table IV, gages FT, FB, BT and BM were used for the compression and tension strains and to verify the loading/strain symmetry. Gages B1, B2 and B3 were used to record the tensile strains on the FRP surface. Two more gages were added at the bottom for girder GF2 to get additional strain information for the 2nd CFRP layer (Fig. 6). A strain gage scanner was used to collect strain data with each element connected as a quarter bridge. Four Linear Variable Differential Transducers (LVDT), clamped to wooden planks at the bottom of each girder, were used to obtain vertical displacements, as shown in (Fig. 8). Two LVDTs at the center of the girder were used to check the maximum displacements of the girders and to check the symmetry of load application. The LVDTs near the supports were installed to verify the support symmetry.

D. Condition Assessment 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 (ASTM D7522 [18]). 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 (Fig. 9a). 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 girders with FRP. Only one sample showed adequate ASTM acceptability with failure in the concrete substrate, indicating good workmanship and bond quality (Fig. 9b). The other two samples had mixed failure modes involving partial concrete, FRP and epoxy failures.

TABLE IV. DESIGNATION OF STRAIN GAGES.

FT – Front Top	BT – Back Top
FB – Front Bottom	BB – Back Bottom
B1, B2, B3, B4, B5 – On the FRP surface at the bottom of the girder	

Fig. 8. *LVDT placement.*

(a) *Pull off testing.*

(b) *Failed samples showing good bond.*

Fig. 9. *Pull-off testing.*

E. 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 (Fig. 10). To simulate simple supports, a steel roller and plate assembly was used at each end (Fig. 11). A load cell with 2669 kN capacity was used, and hydraulic pump was used to apply load through the load cell. The machine was handled manually 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 failure. Failure was defined as the step at which the loading could not be increased anymore with additional loading steps, accompanied by rapid deflection and cracking increase. All test data, such as load, strain and deflection were automatically collected through a data acquisition system.

Fig. 10. *Central load application, girder CF2.*

One control girder (without any CFRP application) and two girders with different CFRP application schemes were used in this study. The designation and symbol used herein for each girder are shown in Table V, and the corresponding CFRP schemes in Figs. 4-6. The flexural wrapping was applied at the mid-span region (for GF1 and GF2) corresponding to the maximum bending moment region in the girders for a three-point loading set-up. The load was applied at mid-span. For GF1, CFRP U-wrapping was applied over the flexural CFRP to provide clamping action and delay debonding. This is discussed in more detail later in this paper.

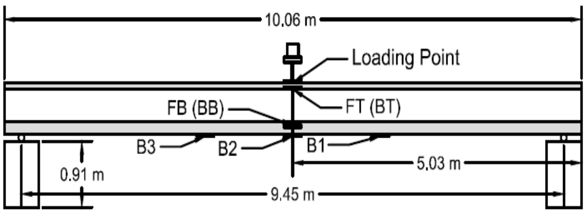


Fig. 4. *Control girder (GC) configuration.*

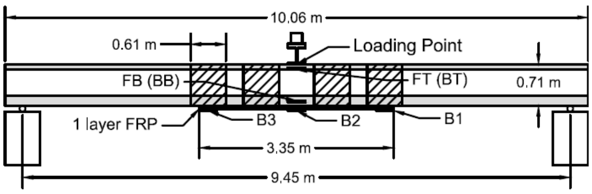


Fig. 5. *Girder GF1 configuration.*

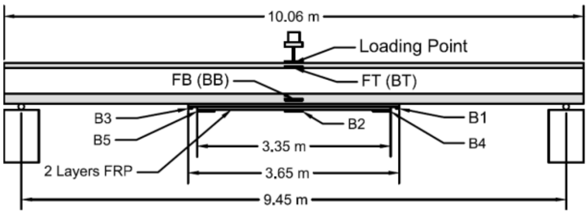


Fig. 6. *Girder GF2.*

TABLE V. GIRDER DESIGNATIONS.

Designation	Girder Description
GC	Control without CFRP
GF1	With one flexural CFRP layer, with U-wraps
GF2	With two flexural CFRP layers, no U-wraps

RESULTS AND DISCUSSION

A. *Crack Initiation*

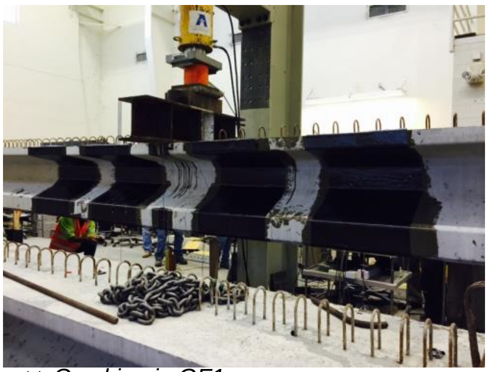
Fig. 12 shows photographs of crack patterns in the girders. As expected, flexural cracks initiated near mid-span and gradually progressed along the depth and outward from the mid-span with increasing loads. Some cracks changed from vertical flexural type to inclined flexure-shear type at some distance from mid-span. As shown in Table VI, the first cracking loads in girders GF1 and GF2 were about 14% and 7% more than in GC, respectively, showing the beneficial effect of flexural FRP in delaying crack development. However, cracks initiated earlier in girder GF2 than in girder GF1. Clearly, the U-wrapping in girder GF1 was the key in delaying crack development, even though GF2 had two layers of FRP. The first cracking loads and the subsequent change in girder stiffnesses are clearly observed in the load-deflection plots in Fig. 13. The maximum vertical deflections (average of the two central LVDTs) at the mid-span were plotted herein. The measured deflections from the two LVDTs were close, showing that the girders were symmetrically loaded in the transverse direction, and no twisting took place during testing.

TABLE VI. SUMMARY RESULTS.

			% Increase in Load Capacity (compared to GC)
			N/A
			7.2
GF2	449	633	5.7

(a) *First crack in GC (at 418 kN load).*

(b) *Cracking at failure, GC.*



(c) *Cracking in GF1.*

Fig. 12. *Cracking in girders.*

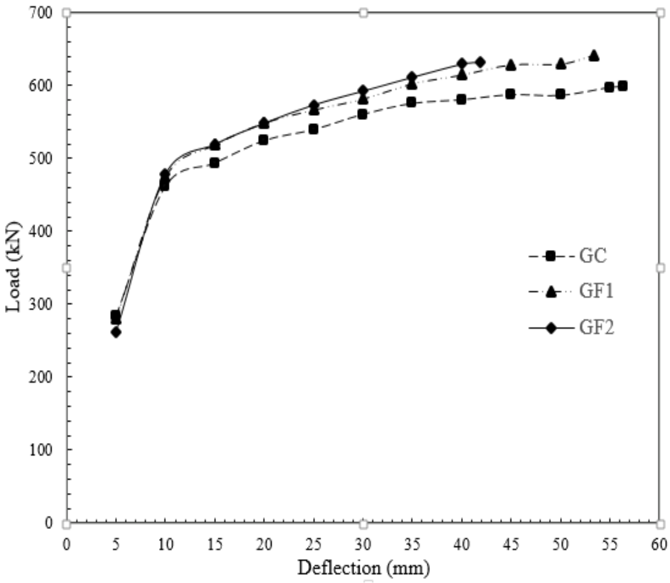


Fig. 13. *Load vs. maximum deflections.*

B. *Failure Mode and Capacity*

With increased loading beyond first cracking, the number of cracks and the crack widths increased. As the girders approached failure, deflections increased at a high rate with minimal load increments. The load dropped with increased deflection at the failure load, signifying the girder failure. In girder GC, failure was initiated by prestress yielding, while the FRP layers debonded at the epoxy interface in a sudden manner with a loud noise in GF1 and GF2 (Table VI). FRP-epoxy debonding photographs are presented in Fig. 14, showing that the FRP clearly debonded at the epoxy interface.

Table VI shows that the actual load capacities in the FRP girders were greater than that for the non-FRP control girder by 6 - 7%. In reality, girder GF2 had a slight drop in load capacity as compared to GF1.

Five possible failure modes are described in ACI 440, as mentioned previously. The governing failure mode is found from strain limit comparisons for a particular concrete strength, section, steel configuration span length and loading. Following appropriate ACI 440 [5] equations (Section 10.1.1, ACI 440, Eq. 10-2), the FRP debonding strain (Mode 5) for the girders herein was found to be 0.0174 and 0.0359 for GF1 and GF2, respectively. The composite FRP laminate tensile rupture strain is 0.01 from the manufacturer's information (Table III). Modes 1 - 3 from above did not control the failure. So, according to the theoretical formulation, the governing failure mode should be FRP rupture at the strain of 0.01 which is lower than the debonding strain. However, the actual observed FRP failure was not any of the Modes 1-5, but failure of the FRP-epoxy interface. Photographs of observed girder debonding are presented in Fig. 14. It should be noted that, although FRP U-wrapping is mentioned as a possible clamping mechanism to delay tension FRP debonding in the current ACI 440, the actual provisions do not recognize any increased debonding failure strain from such clamping.

The maximum strains in Table VII show that the compressive strains at the top flange (gages BT, BB) were well below the generally assumed concrete crushing strain of 0.003. The maximum tensile FRP strains (0.0056 to 0.0065) occurred at mid-span, as expected (for gage B2). However, these values are quite less than the expected failure strain of 0.01 (FRP rupture).

It is apparent that the expected FRP failure modes could not be achieved due to possible inadequate surface preparation and/or FRP/epoxy storage, mixing or application. The theoretical provisions are based on a perfect FRP-epoxy bonding with that would shift the failure away from the interface to either FRP rupture or debonding within the concrete matrix. One explanation could be any lack of adequate quality of the FRP installation, in the form of air bubbles under FRP and inadequate surface preparation. However, the installation was done by a well-known professional company using techniques that are typically used in actual concrete bridge FRP installation. No apparent defects in the installed FRP were observed upon inspection of the FRP surface.

C. *Girder Ductility*

It is evident from Fig. 14 that girder GF2 had less ductility than girder GF1, with a maximum deflection about 15 mm less than GF1. Clearly, the U-wrap clamping in GF1 was beneficial in delaying the FRP failure. It may also be noted that the P-Δ plots for GF1 and GF2 are in close proximity to each other. Although girder GF2 shows a small increased loading at various deflections, it is not appreciable. The extra layer of FRP did not significantly contribute to the flexural capacity, as also verified from Table VI values. As noted, girder GC had the highest mid-span deflection, followed by GF1 and GF2, respectively.

Fig. 15 presents the load vs. flexural strain plots for the girders. Results from the central strain gages B2 were plotted herein. The strains in the graph shows increment along with the load applied. Once the FRP failed, there was rapid decrease in the strain with the increase in load. Among the FRP strengthened specimens, GF1 showed the higher strain than in GF2 before FRP failure. The reason for this high strain in GF1 is that U-wraps for anchorage were applied on it, which slowed the FRP failure.

(a) *Girder GF1*.

(b) *Girder GF2.*

Fig. 14. *FRP debonding at failure.*

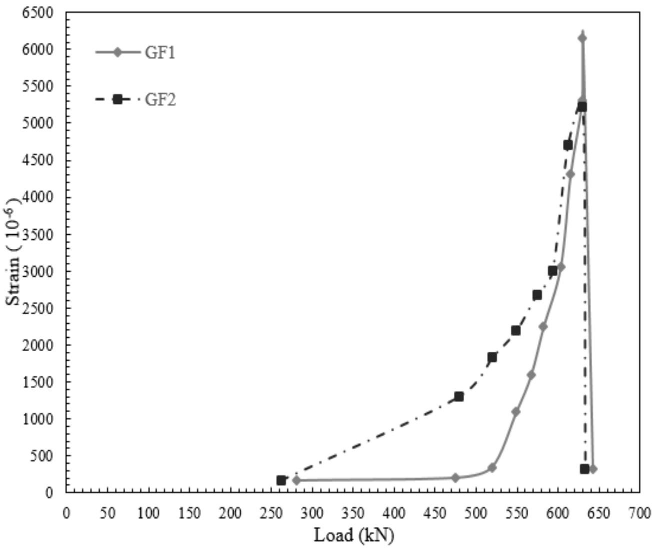


Fig. 15. *Load vs. strain, center gage B2.*

TABLE VII. MAXIMUM STRAINS AT FAILURE.

Strain Gage	Microstrain*	
	Girder GF1	***Girder GF2***
FT	-666 (C)	-440 (C)
BT	-639 (C)	-451 (C)
B1	+4648 (T)	+533 (T)
B2	+6517 (T)	+5595 (T)
B3	+1033 (T)	+287 (T)
B4	N/A	+766 (T)
B5	N/A	+348 (T)

** C: compression; T: Tension.*

IV. CONCLUSIONS

The following conclusions may be made based on the findings from the current study:

1. Adequate experimental investigation of the flexural behavior of FRP strengthened full-scale prestressed concrete bridge girders in order to determine the expected strength gain and the FRP failure mode is lacking.
2. It was found that flexural FRP strengthening was helpful for enhancing the performance of Tx28 concrete bridge girders. The benefits include: 7-14% increase in first cracking load, 6-7% increase in ultimate load, and up to 25% increase in ductility.
3. Flexural cracking in FRP strengthened girders initiate and progress similarly to those in un-strengthened control girders. The crack initiation is slightly delayed in the FRP girders with additional stiffness, with the cracking load about 7-14% larger for the non-FRP girders.
4. U-wrapping with transverse FRP that anchors the flexural FRP is clearly beneficial in delaying crack initiation and propagation, increased load carrying capacity, delaying FRP failure and increased ductility. U-wrapping is more beneficial than increasing the number of flexural FRP layers from one to two.
5. Flexural FRP failure at the epoxy layer was the observed failure mode in both FRP application investigated herein. However, according to theoretical provisions, the failure should have been governed by flexural FRP rupture. The maximum achieved FRP tensile strains were quite less than that expected for the FRP rupture strain. Clearly, the concrete surface preparation and/or the FRP-epoxy storage, mixing and applications were inadequate. However, these tasks were performed by a very well- known national company using techniques that are prevalent in actual bridge applications.
6. 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 evidently a need to emphasis the minimum FRP application quality specified by guidelines and manufacturers. This will prevent the premature FRP-epoxy interface debonding, allowing for other relevant FRP failure modes to take place with increased load capacity and ductility.
7. The relevant guidelines do not currently address the beneficial anchoring effect of FRP U-wrapping on the behavior of flexurally strengthened concrete bridge girders. U-wrapping is not a factor in the design procedures from the guidelines.

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