CENTER FOR TRANSPORTATION INFRASTRUCTURE AND SAFETY

Accelerated Construction of Bridge 14802301, Greene County, Missouri, with Prefabricated Stay-in-Place Glass Fiber Reinforced Polymer Reinforcement

by

Fabio Matta

Antonio Nanni

Thomas E. Ringelstetter

Lawrence C. Bank

A National University Transportation Center
at Missouri University of Science & Technology

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16. Abstract

In the two-year project reported herein, a new prefabricated stay-in-place (SIP) glass fiber reinforced polymer (GFRP) reinforcing system for the accelerated construction of bridge decks was developed and implemented in the reconstruction of Bridge 1482301 in Greene County, Missouri, USA. The owner opted for the replacement of the 70-year old bridge due to the severe corrosion-induced deterioration of the superstructure. A significant joint effort was required for the concept development, design, analysis, detailing, full-scale laboratory validation, development of project special provisions, and construction planning and execution. Related tasks were completed in coordination between Missouri S&T and the University of Wisconsin-Madison, FRP reinforcement manufacturers, the Greene County Highway Department (owner), the engineer-of-record, and the designated contractor.

The project demonstrated a technology that combines the superior durability of internal FRP reinforcement for concrete, and the substantial constructability advantages that derive from the use of light-weight advanced composite systems engineered in an innovative, integrated stay-in-place form.

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ACCELERATED CONSTRUCTION OF BRIDGE 14802301, GREENE COUNTY, MISSOURI, WITH PREFABRICATED STAY-IN-PLACE GLASS FIBER REINFORCED POLYMER REINFORCEMENT

EXECUTIVE SUMMARY

In the two-year project reported herein, a new prefabricated stay-in-place (SIP) glass fiber reinforced polymer (GFRP) reinforcing system for the accelerated construction of bridge decks was developed and implemented in the reconstruction of Bridge 1482301 in Greene County, Missouri, USA. The owner opted for the replacement of the 70-year old bridge due to the severe corrosion-induced deterioration of the superstructure. A significant joint effort was required for the concept development, design, analysis, detailing, full-scale laboratory validation, development of project special provisions, and construction planning and execution. Related tasks were completed in coordination between Missouri S&T and the University of Wisconsin-Madison, FRP reinforcement manufacturers, the Greene County Highway Department (owner), the engineer-of-record, and the designated contractor.

Prototype SIP reinforcement panels were prefabricated using off-the-shelf pultruded GFRP profiles arranged in a two-layer grating, where load transfer is achieved by mechanically constraining the core concrete within the three-dimensional grating. The SIP formwork consists of GFRP plates that are epoxy-bonded to the bottom layer of reinforcement.

The use of large-size and light-weight (23.7 kg/m^2) panels was aimed at allowing easy and rapid installation with single picks of a crane, thereby eliminating time-consuming and labor-intensive setting and removal of plywood forms and tying of bars on-site. The steel-free deck system was complemented by a newly designed open-post concrete railing reinforced with light-weight, prefabricated cages made of pultruded GFRP bars.

The design of post-deck connection and deck were validated through laboratory testing of full-scale overhang subassemblies and deck panel specimens. The results substantiated the design outcomes, and confirmed significant safety margins at failure with respect to applicable code-mandated strength requirements.

Construction was planned with the contractor to minimize time. The job was completed in November 2005 in five days, instead of the typical 2-3 weeks for similar steel reinforced concrete bridges built by the same contractor. A total of 18 panels were installed by six workers in six hours during the first day, covering the total length of 44 m of the slab-on-girder bridge. The second day was dedicated to mounting the railing post cages into the deck panels, preparing the formwork for expansion joints, chamfers and drip edges, and setting the finishing machine. The deck was cast and finished in the third work day. In the morning of the fourth and fifth days, the railing beam cages were mounted on top of the post cages prior to forming and casting.

The project demonstrated a technology that combines the superior durability of internal FRP reinforcement for concrete, and the substantial constructability advantages that derive from the use of light-weight advanced composite systems engineered in an innovative, integrated stay-in-place form.

ACKNOWLEDGMENTS

The financial support of the National UTC for Transportation Infrastructure and Safety, and of the Federal Highway Administration through the Innovative Bridge Research and Construction Program, is acknowledged. The assistance of Strongwell Corp. and Hughes Brothers Inc., industry members of the NSF Industry/University Cooperative Research Center for "Repair of Buildings and Bridges with Composites" (RB^2C), is also acknowledged. Special thanks are due to the Greene County Highway Department, MoDOT, Great River Engineering, Hartman Construction, and Master Contractors LLC, for their precious support throughout the project.

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1. INTRODUCTION

During the last decade, increasing investments have been made to support the research and development of innovative technologies for accelerated bridge construction, primarily under the sponsorship of the Federal Highway Administration (FHWA), the American Society of State Highway and Transportation Officials (AASHTO Technology Implementation Group), and the Transportation Research Board (TRB Task Force on Accelerating Innovation in the Highway Industry). Emphasis has been placed on improving safety and minimizing traffic disruption while enhancing quality and durability. The issue stems from the urgent need of upgrading and maintaining a significant portion of the bridge inventory while facing inevitable budget restrictions. Redecking operations are rather frequent, since corrosion of steel reinforcement is a major instrument of degradation in reinforced concrete (RC) decks and safety appurtenances. In the case of off-system bridges, cost-benefit analysis, contractors' know-how and equipment availability typically result in the adoption of either partial or full-depth cast-in-place (CIP) technologies. The most popular solution limits the use of prefabricated elements to standardized partial-depth precast prestressed concrete panels as structural stay-in-place (SIP) forms between the girders, with CIP concrete topping, as opposed to traditional removable plywood forms. SIP steel metal deck forms, with a full-depth CIP configuration that eliminates the problem of reflective cracks, are in some circumstances less attractive due to three drawbacks: a) safety concerns due to risks of accidental damage of relatively thin metal sheets, resulting in local buckling problems under wet concrete load; b) corrosion issues under aggressive environments; c) efficient inspection of the underside of the deck is complicated.

In the project presented herein, an innovative prefabricated glass Fiber Reinforced Polymer (FRP) SIP reinforcement has been selected to construct the replacement deck of Bridge 14802301 in Greene County, MO. Corrosion resistant FRP reinforcement gratings and SIP form plates are integrated into very large-size modular panels. The structural form takes advantage of FRP composites tailorability and lightweight to provide improved constructability, resulting in enhanced construction speed and safety. Prefabricated GFRP bar cages were used for the open-post reinforced concrete (RC) railing to complement the deck reinforcement, thereby developing a truly steel-free deck and railing system.

2. BRIDGE DESCRIPTION

The owner of Bridge 14802301 in Greene County, Missouri, USA, opted for the replacement of the original 70-year old slab-on-girder superstructure because of its precarious conditions, in addition to the increased load requirements. Figure 1(a) and Figure 1(b) show the extensive degradation that afflicted the RC deck, the top flanges of the steel girders, and the connections between the deck and the safety barrier.

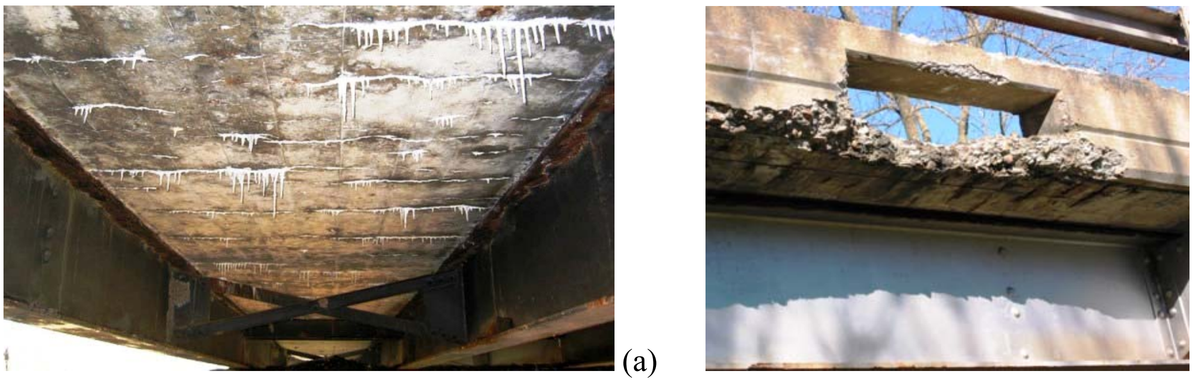
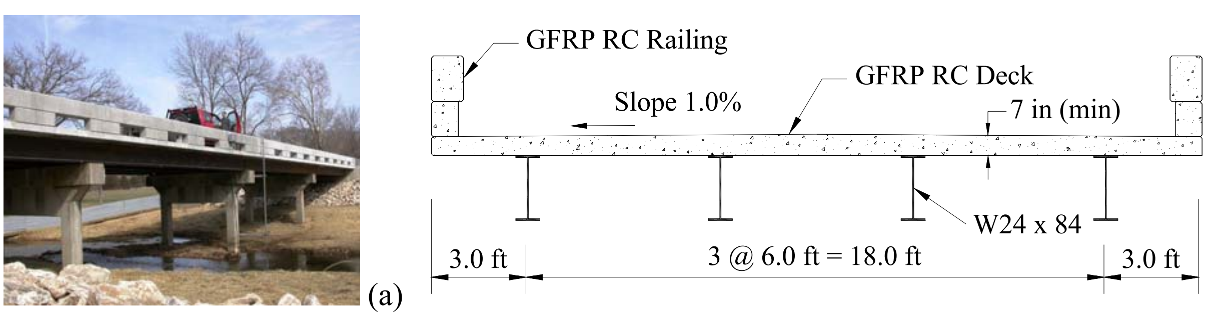

(a) (b)

Figure 1 – Old Bridge 14802301, Greene County, MO: degradation of concrete deck and steel girders (a), and of safety appurtenance (b).

The new superstructure, depicted in Figure 2, has four spans of 37.0 ft exterior and 35.0 ft interior, for a total length of 144.0 ft. The girders are piecewise continuous over two spans, with a closed expansion joint at the central support. The cross section comprises four W24×84 steel girders spaced at 6.0 ft on-center and acting non-compositely with a 7.0 in. thick deck, and open-post concrete railings with post and opening width of 4.0 ft. The deck and clear roadway width are 24.0 ft and 22.0 ft, respectively.



(a) (b)

Figure 2 – New Bridge 14802301: photograph (a) and schematic of cross section (b).

A total of thirteen regularly spaced steel diaphragms (bolted 18.0×4.0 in. C-sections with thickness of 5/16 in.) are used as transverse stiffeners.

3. PREFABRICATED GFRP REINFORCEMENT

3.1 Stay-in-Place Deck Panels

The SIP deck panels are prefabricated by assembling off-the-shelf pultruded E-glass/vinylester components, typically used in floor grating applications in corrosive environments, into a three-dimensional grating (GridForm) made of two (top and bottom) layers, as depicted in Figure 3. The GFRP material was modified to meet the prescriptive specifications summarized in Table 1, and detailed in Appendix A. The main load-carrying elements are 1.5 in. I-bars spaced at 4.0 in. on-center, which run continuously in the direction perpendicular to traffic (transverse). Both shape and spacing of the I-bars have been devised to allow ease of walking over the three-dimensional assembly. Three-part cross rods, spaced at 4.0 in. on-center and running through pre-drilled holes in the I-bars web in the direction parallel to traffic (longitudinal), provide shrinkage and temperature reinforcement, enhance the in-plane rigidity of each reinforcing layer, and constrain the core concrete to ensure mechanical compatibility with the structural I-bars. Top and bottom reinforcing layers are integrated using two-part vertical connectors that space them at 4.0 in. on-center. The two components that form the connectors are shaped to be epoxy-bonded to the I-bars and then fastened together. The integrated formwork consists of 1/8 in. thick and 4.0 ft long plates that are epoxy-bonded to the I-bars in the bottom layer.

The system concept, detailing and construction procedure have been addressed to improve constructability by introducing original solutions when needed, and constantly seeking input from practitioners. Each panel has a width of 23 ft–2 in., a typical length of 8.0 ft [Figure 3(a)], and a weight of about 900 lb (4.85 lb/ft^2). The width corresponds to that of the bridge deck minus 5 in. per side in order to allow a traditional drip edge notch to be formed on-site. The use of large-size and light-weight panels allows easy placement of the SIP reinforcement on the bridge girders with single picks of a crane at four anchorage points. Hence, both time-consuming and labor-intensive setting/removing of plywood forms and tying of bars are eliminated. Adjacent panels are connected in a non-mechanical fashion by means of 1.0 ft long overlaps, which are formed by offsetting the top and bottom grating layers [Figure 3(a)], thereby preserving a degree of continuity in the longitudinal direction [Figure 4(a)]. 1/8 in. thick GFRP strips are inserted to cover the plate-to-plate butt joints, thus preventing concrete leaking during casting [Figure 4(b)].

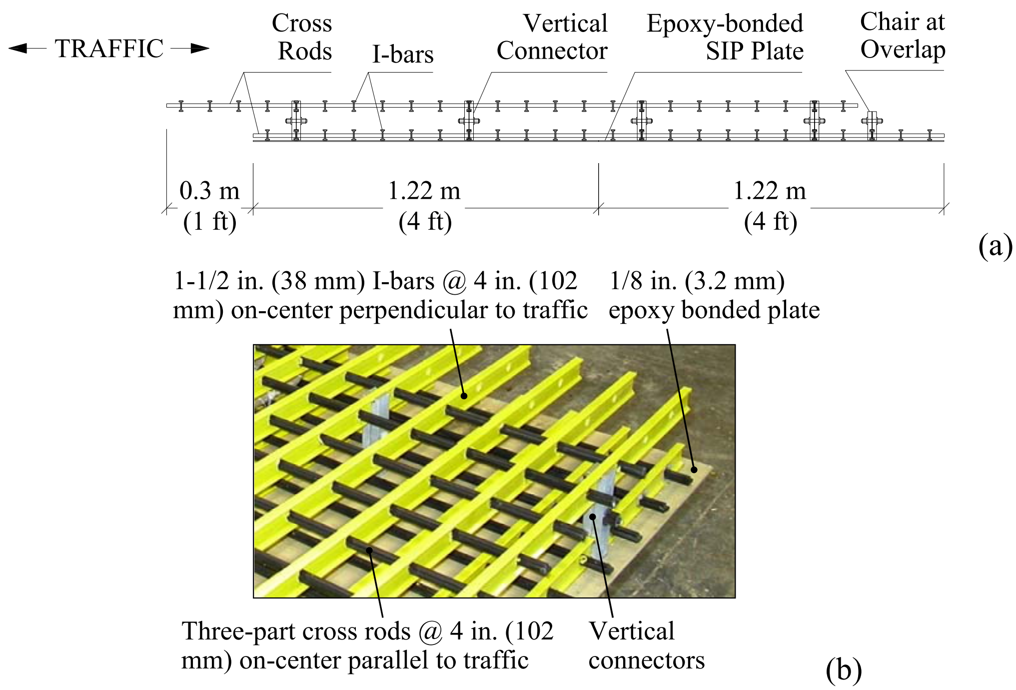


Figure 3 – GridForm reinforcement panels: longitudinal section (a) and close-up (b).

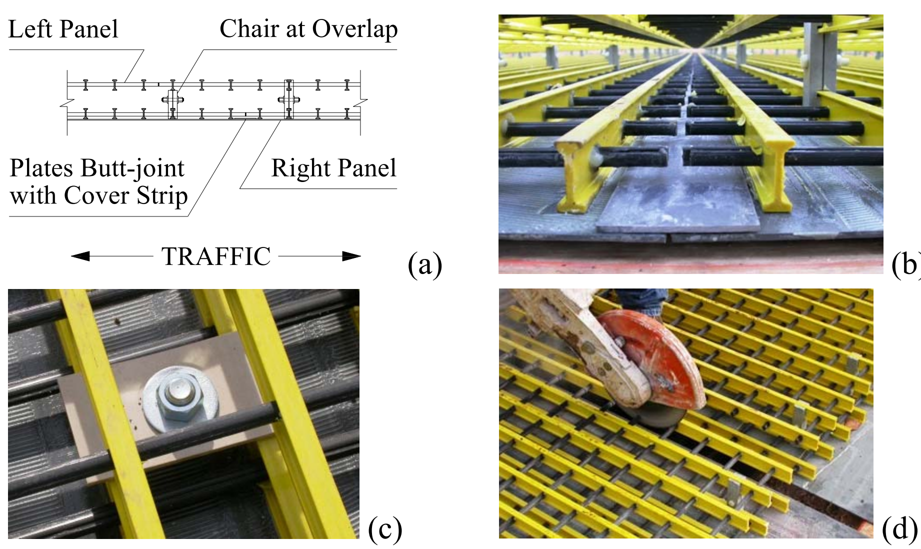


Figure 4 – GridForm detailing: panel-to-panel overlap connection (a, b); anchoring to girder (c); and cutting of end panels at expansion joint (d).

Table 1 – Limiting mechanical properties for GFRP I-bars in SIP panels (see Appendix A).

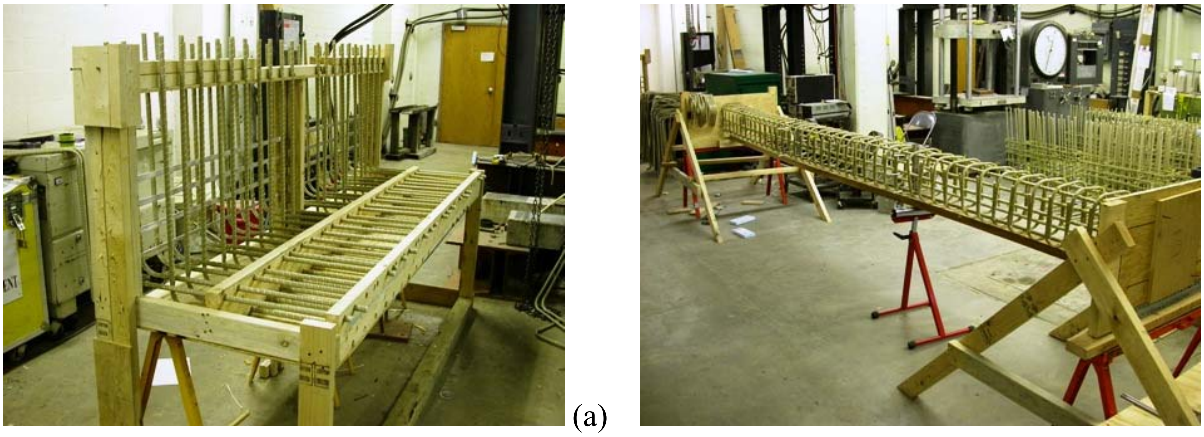
Material Property	**GV2 Material**
Longitudinal tensile strength	80.0 ksi
Transverse tensile strength	4.0 ksi
Longitudinal compressive strength	60.0 ksi
Transverse compressive strength	10.0 ksi
Longitudinal tensile modulus	4.5 msi
Longitudinal compressive modulus	4.0 msi
In-plane shear stiffness	0.40 msi
Major (longitudinal) Poisson's ratio	0.25

When using steel girders, each SIP unit is anchored to the top flanges via stainless steel threaded bolts at every 8.0 ft, keeping the bottom reinforcing layer in place with 1/4 in. thick GFRP washers [Figure 4(c)]. The holes in the SIP plate are drilled on site. When composite action is sought between girders and deck, the panels can be supplied with pre-drilled holes with longitudinal and transverse spacing of 4.0 in. on-center to accommodate welded shear studs. No cambering of the panels is required to match the roadway crown, which is formed using the finishing machine. The length and layout of the end panels are designed to fit the actual bridge length and accommodate the expansion joints. Since GFRP is easy to saw-cut, adjustments can be readily made on site [Figure 4(d)].

3.2 Railing Bar Cages

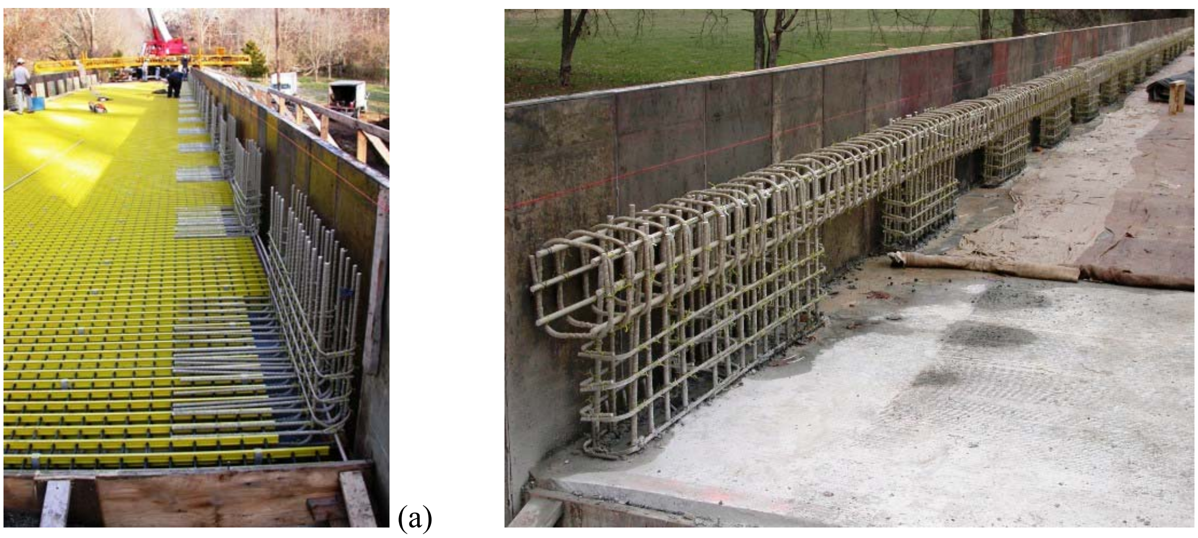
Prefabricated, light-weight cages made of pultruded E-glass/vinylester bars were used to construct the open-post railings (Figure 5 and Figure 6). The tensile properties of the bars used are reported in Table 2, and detailed in Appendix A.

Cut-out pockets within the overhang reinforcement facilitate insertion of the post cages at the correct spacing [Figure 6(a)], while the longitudinal beam cages are installed upon casting of the deck and spliced using additional GFRP bars.



(a) (b)

Figure 5 – Shop preassembly of railing cages: posts (a) and longitudinal beams (b).



(a) (b)

Figure 6 – Open-post railing construction: Mounting of post cages in panel cut-out pockets (a) and of beam cages upon casting of deck (b).

Table 2 – Limiting full-section properties of GFRP bars for railing cages (see Appendix A).

Bar Size	**Nominal Diameter**	**Tensile Strength**	**Longitudinal Tensile Modulus**
#3	0.375 in.	110 ksi	5.92 msi
#5	0.625 in.	95 ksi	5.92 msi

4. STRUCTURAL DESIGN

The GFRP RC deck and railings were designed following the ACI 440 guidelines (ACI 440 2003), and in compliance with applicable criteria mandated by the AASHTO Standard Specifications (AASHTO 2002). Concrete with nominal cylinder compressive strength f'_c = 4000 psi was assumed in the calculations.

4.1 Bridge Deck

Load Factor Design – The Load Factor Design (LFD) is applied assuming a 1.0 ft wide, 24.0 ft long and 7.0 in. thick deck strip on simple supports spaced 6.0 ft apart, which are representative of the steel girders (AASHTO 2002). Design positive and negative bending moments are obtained by multiplying their nominal values by the dead and live load factors, β_D and β_L, and by the impact factor, I, i.e.,

$$\omega_{LFD} = \gamma\left[\beta_D D + \beta_L L\left(1+I\right)\right] = 1.3\left[D + 1.67\left(1.3L\right)\right]$$

where I = 0.3 is taken as the suggested maximum value (AASHTO 2002). The nominal live load positive and negative moment per ft produced by HS20-44 truck loading, in case of main reinforcement perpendicular to traffic, are calculated as

$$M_{L+} = M_{L-} = 0.8\left(\frac{S+2}{32}\right)P_{20} = 0.8\left(\frac{5.67+2}{32}\right)16 = 3.07 \text{ kips-ft/ft}$$

where 0.8 = continuity factor to be applied for both positive and negative moment calculations for the case of slab continuous over three or more supports; S = effective span length, in ft, defined as the distance between the edges of the I-girder top flange plus one-half of the top-flange width; and P_{20} = 16 kips for HS40-44 loading. The unfactored live load negative moment per ft at the overhang is computed as

$$M_{L-,o} = \frac{P_{20}}{0.8X + 3.75}X = \frac{16}{0.53+3.75}0.67 = 2.50 \text{ kips-ft/ft}$$

wherein $E = 0.8X + 3.75$ is the effective slab width, X = 0.67 ft the distance of the load, applied at a distance of 1 ft from the inner face of the parapet, from the point of support, which is assumed at the flange edge of the exterior steel girder.

Table 3 summarizes the LFD ultimate bending moments between the girders and at the overhang, along with the design positive and negative moment capacity of the slab as per ACI 440 (2003). Detailed calculation of the design moments, $\phi_f M_n$, are given in Appendix B.

Table 3 – Results of LFD analysis of bending moment in deck (see Appendix B).

Deck area	Parameter	$M_{DL,deck}$ (kip-ft/ft)	$M_{DL,railing}$ (kip-ft/ft)	$M_{L,HS20\text{-}44}$ (kip-ft/ft)	M_{LFD} (kip-ft/ft)	$\phi_f M_n$ (kip-ft/ft)
Between girders	Positive moment	0.16	0.14 (constant)	3.07	9.05	14.16
	Negative moment	–0.24		–3.07	–8.79	–9.98
Overhang	Negative moment	–0.41	–0.72	–2.50	–8.52	–9.98

The deck resistance against post loadings at a section flush with the exterior girder web is checked based on an effective length of resisting slab given by

$$E = 0.8X + 5 = 0.8 \times 2.25 + 5 = 6.8 \text{ ft}$$

(AASHTO 2002).

Service Load Design – Service design positive and negative bending moments are obtained by multiplying their nominal values by the dead and live load factors, β_D and β_L, and by the impact factor, I, i.e.,

$$\omega_{SLD,I} = \gamma\left[\beta_D D + \beta_L \left(L + I\right)\right] = D + 1.3L$$

The maximum deck deflection between the girders produced by the design live load is computed as 0.034 in., *i.e.*, $\Delta i_{LL+I} = S\ /\ 1972$. As a reference, the applicable AASHTO LRFD Bridge Design Specifications (AASHTO 1998) recommend limiting such ratio to S / 300. The maximum short term and long term deflections under service loads are computed as 0.038 in and 0.047 in, respectively.

The maximum crack widths under positive and negative moment are determined as 0.018 in. and 0.033 in., respectively, with the recommended limiting value being 0.028 in (ACI 2003). In the latter case, the excess width was considered acceptable on the sole basis of aesthetic considerations. Detailed calculations are reported in Appendix B.

4.2 Open-Post Railings

The geometry and reinforcement layout of the open post railings and the post-deck connections are detailed in Figure 7 and Figure 8, and result in a modified GFRP RC version of the required Modified Kansas Corral Rail (MKCR). The original MKCR profile, which performed adequately under crash testing by preventing vehicle snagging and rollover, was improved by increasing the height of the rail beam from 14 in. to 17 in., for a total height of 30 in., to further reduce the risk of rollover.

Two layers of bent No. 5 GFRP bars were used to connect the post to the deck panels, and a shear key was included at the construction joint. A rigorous procedure was followed for the structural design to resist the required 10 kip transverse load applied at the mid-height of the 17 in. high rail beam face (AASHTO 2002). Failure may occur due to concrete crushing or FRP reinforcement rupture in flexure at the weakest connected section, insufficient anchorage of the post or development length of the deck reinforcement, or diagonal tension cracking at the corner. In the last three cases, the design fails to fully utilize the reinforcement, and may yet be preferred due to constructability and cost considerations, provided that the strength requirements are met.

The design in Figure 8 requires a check against diagonal tension failure at the corner joint at the post-deck connection. As detailed in Figure 9, the transverse load F_p applied to the post produces a compression force C_p in the post section at the connection, which is transferred to the deck via the formation of a diagonal compression strut of length l_{dc}. In addition, the shear force F_p is transferred to the deck as an axial force $-F_p$ and a bending moment $0.5F_pt_d$, which adds to F_pH_e to produce the resultant moment in the deck M_d that generates the force couple C_d and $F_{f,d}$. Diagonal cracking may occur prior to flexural failure in the deck as the concrete modulus of rupture f_r is reached along the diagonal strut, thereby disabling the main load transfer mechanism.

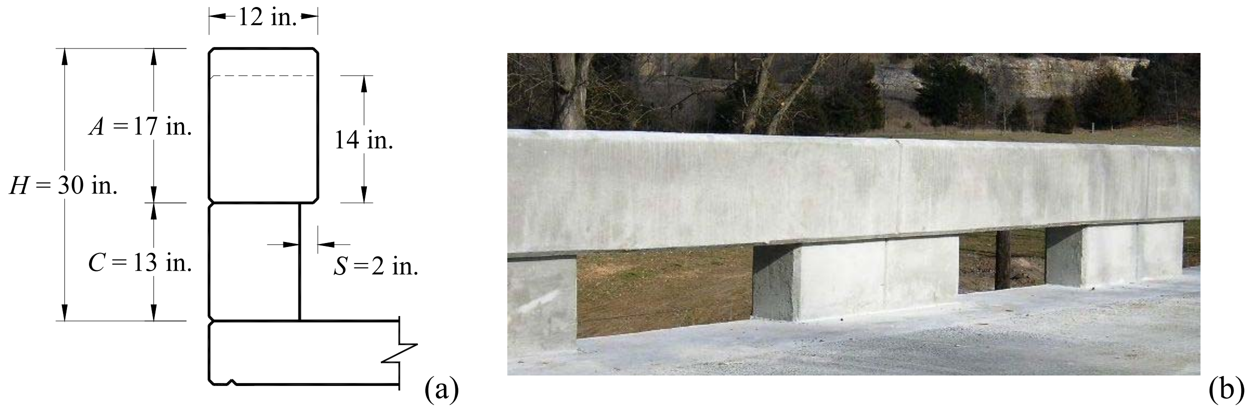


Figure 7 – Geometry of GFRP RC MKCR: thru-section profile (a); and photograph of railing with post and gap opening length of 4 ft (b).

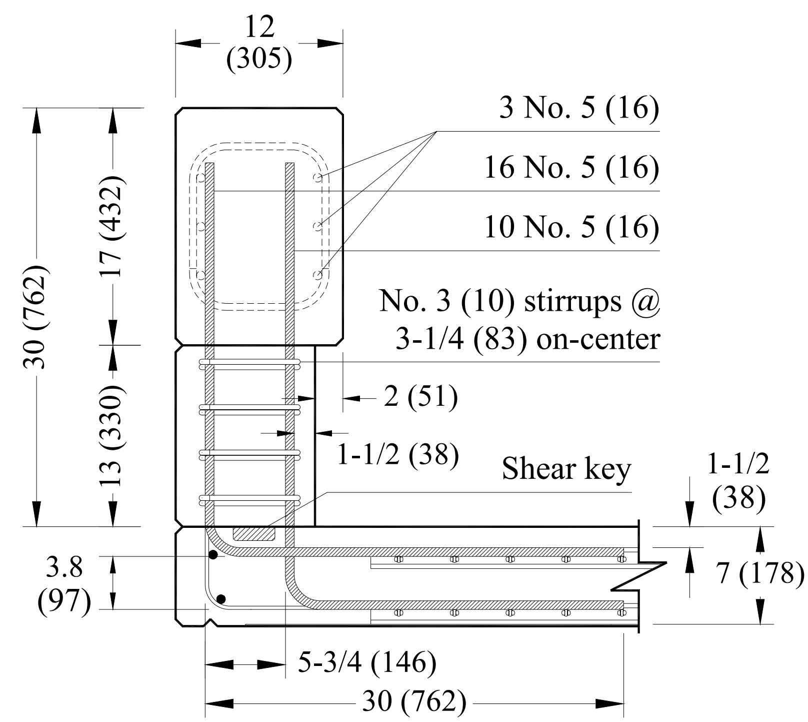


Figure 8 – Reinforcement layout of post-deck connection. Dimensions in in. (mm).

The accuracy of analytical results based on the theory of elasticity, where a parabolic distribution of the tensile stress along l_{dc} is assumed, has been demonstrated with respect to experimental results (Nilsson and Losberg 1976). The original closed-form procedure was modified and rendered in an iterative fashion to explicitly account for the effect of the shear force F_p in addition to the bending moment M_d, and is summarized in the flow chart in Figure 10. The tensile force T acting perpendicular to the diagonal strut is computed neglecting any strength contribution of the slab portions adjacent to the connection, and assuming $f_r = 7.5\sqrt{f_c'}$ (psi). Figure 9(c) shows the free body diagram of the corner joint with the resultant internal forces. Convergence is achieved for a nominal strength $F_{n,p}$ of 11.9 kip, which corresponds to 30% of the nominal flexural capacity of the deck section in Table 3. The design strength is computed as $\phi_{dt}F_{n,p}$ = 10.1 kip by assuming a reduction factor for diagonal tension ϕ_{dt} = 0.85, thus meeting the minimum 10 kip requirement mandated by AASHTO (2002). Detailed calculations are provided in Appendix B.

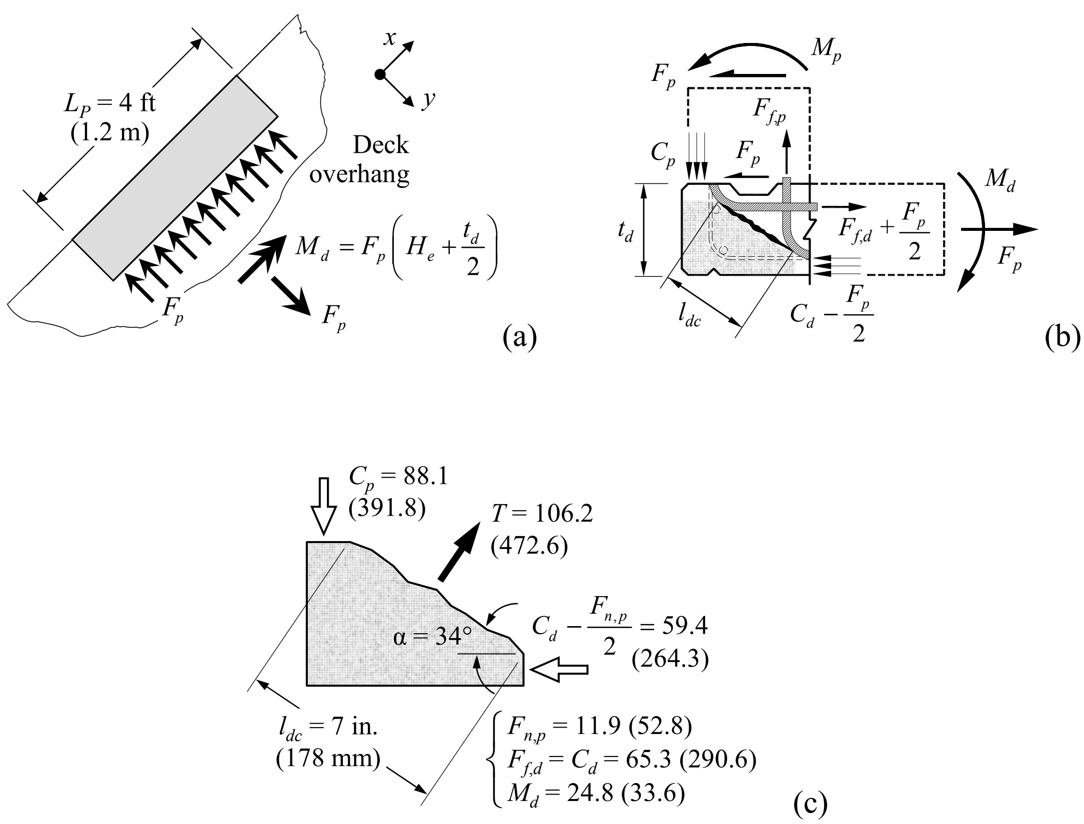


Figure 9 – Design of post-deck connection: applied force and reactions in deck (a); internal forces at connection (b); and free body diagram of corner joint (c). Forces and moment in kip (kN) and kip-ft (kN-m).

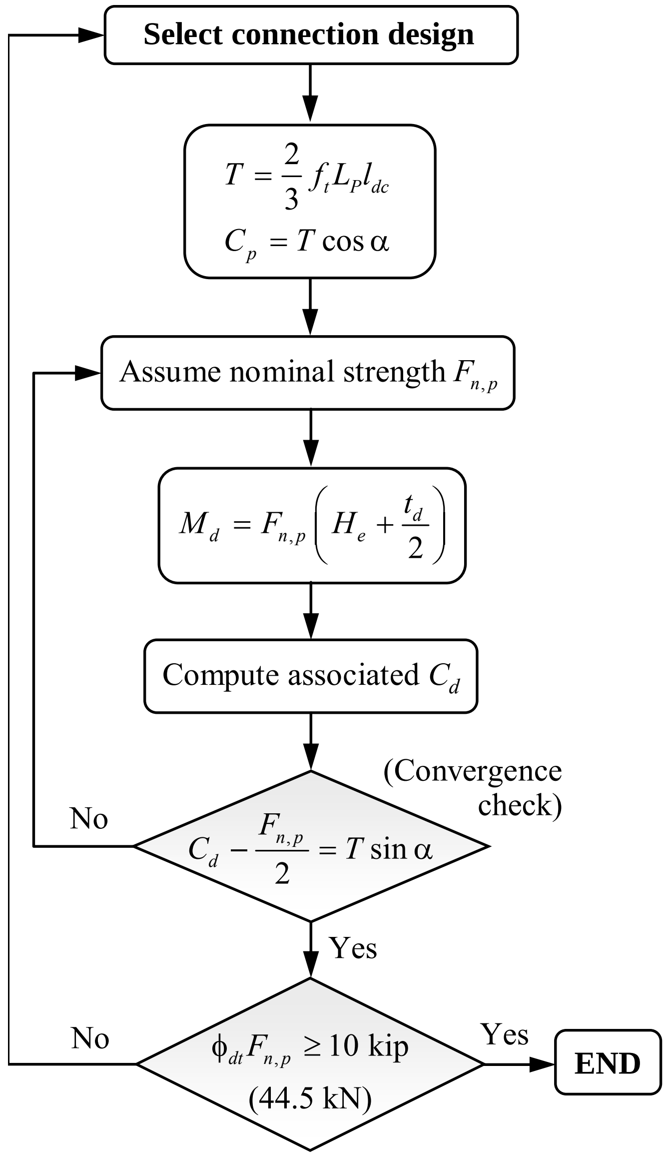


Figure 10 – Flow chart for post-deck connection design controlled by diagonal tension failure at corner.

AASHTO (2002) also requires that the railing beam be designed for a moment produced by a concentrated load of 10 kip applied at the mid section of the opening, and given as 10 kip × [L_O (ft) / 6] = 6.7 kip-ft using a 4 ft opening length. The beam design includes three No. 5 tension bars per side (Figure 8) with an effective depth d of 10.2 in. in the rectangular section, thus providing a nominal and design moment capacity $M_{n,b}$ and $\phi_f M_{n,b}$ of 52.0 kip-ft and 26.0 kip-ft, respectively. The shear reinforcement consists of No. 3 double-C GFRP stirrups spaced at 4 in. (102 mm) on-center, which provide a design shear strength of 25.1 kip (111.6 kN). The beam design allows to withstand the maximum moment produced by the design load, and to transfer it to the adjacent posts.

5. LABORATORY VALIDATION

5.1 GFRP RC Deck

Structural adequacy of the deck design was validated through full-scale testing of one-way beam and slab panel specimens, respectively (Ringelstetter et al. 2006), to assess flexural strength, on which design is based, and punching shear strength, which is typically the dominant failure mode for RC bridge decks. A 7 ft×8 ft×7 in. GridForm RC slab, simply supported on-center along the shorter sides, was tested under static patch load. A concrete design mix with 3/4 in. maximum aggregate size, and 28-day compressive strength of 4000 psi, was utilized. The actual average strength was 4350 psi, as determined through cylinder compression tests per ASTM C39.

A maximum midspan deflection of 0.332 in. was measured during casting, corresponding to 0.175 in. in case of a continuous three-span configuration. The value is significantly below the required limit of 0.25 in. for conventional SIP formwork (AASHTO 1998).

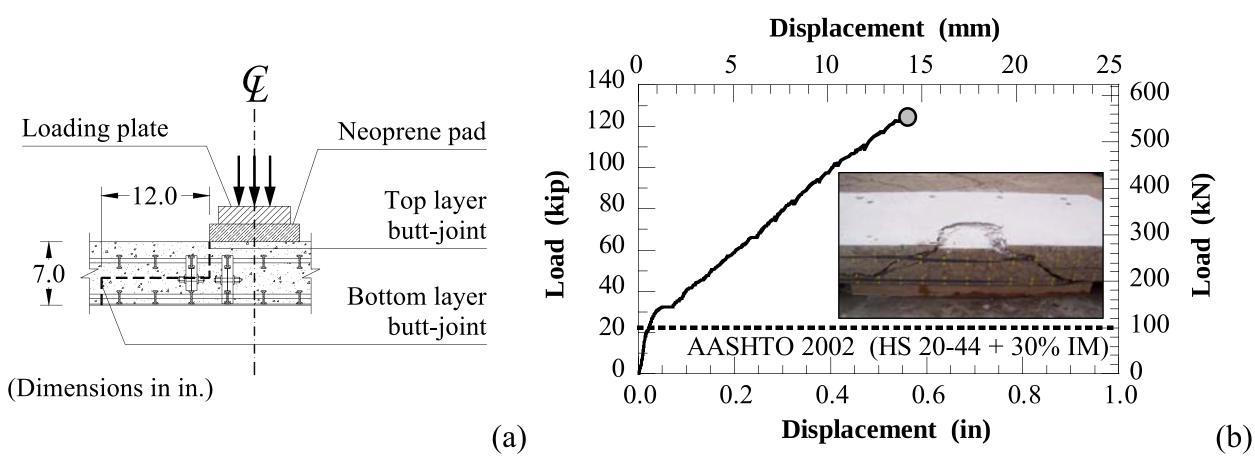


Figure 11 – Punching shear test on deck panel: lap splice section (a) and load-center displacement response (b).

A lap splice was positioned flush with the edge of the neoprene loading pad [Figure 11(a)] to assess the response at the weakest reinforcement area. The load-center displacement response is plotted in Figure 11(b). The punching shear strength was 124.9 kip, that is, about six times the 21.0 kip load given by the 16.0 kip HS20-44 truck wheel service load increased by a 30% impact factor (IM).

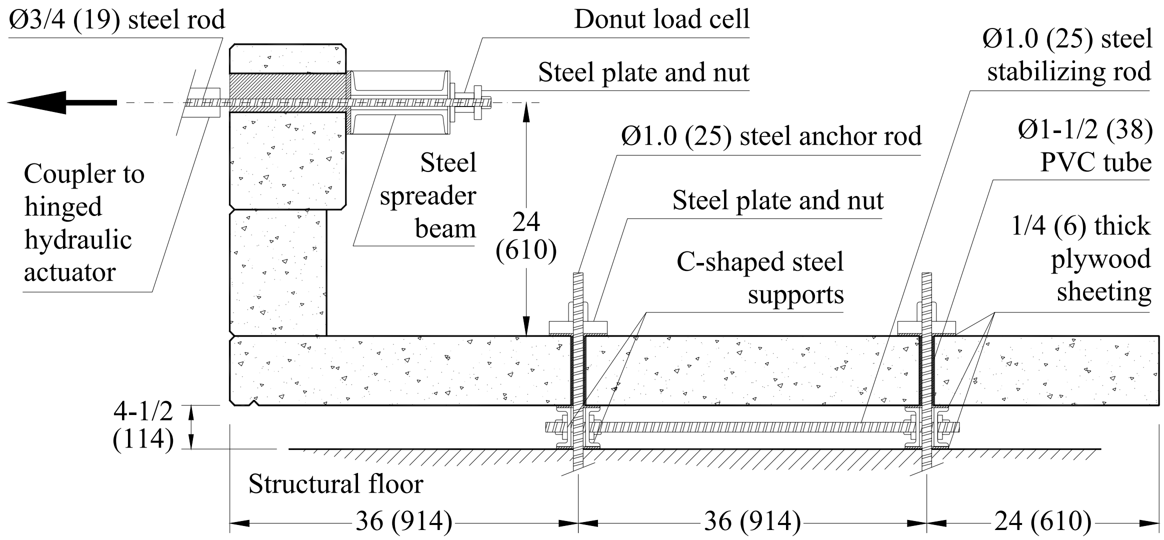
5.2 GFRP RC Railing

The transverse strength of the post-deck connection was assessed by testing a full-scale overhang subassembly under quasi-static transverse load. The setup included a 8.0×8.0 ft slab supported on steel beams and tightened to the laboratory strong floor using two rows of three Ø1.0 in. steel threaded rods each spaced at 3.0 ft on-center (Figure 12). The load was applied at a height of 2.0 ft from the slab surface using a steel spreader beam, which was engaged by a steel plate and threaded rod assembly that was connected to the hinged fitted-end of a hydraulic jack.

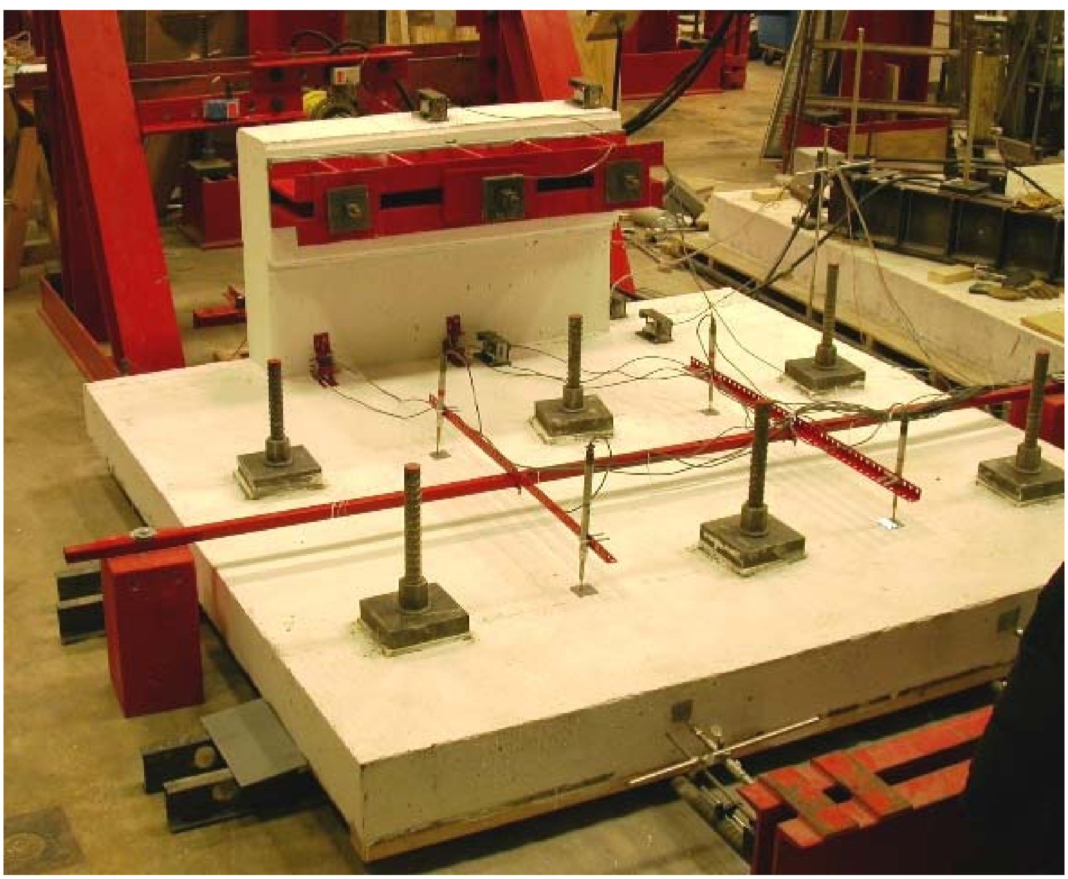
The specimen, which replicated the reinforcement layout in Figure 8, was constructed using normal weight concrete with average cylinder compressive strength f'_c of 4975 psi in the slab, and of 8422 psi in the post, respectively.

The maximum transverse displacement measured at the mid section at the top of the post is plotted with respect to the applied load in Figure 13. Deck and post-deck interface cracking developed between 6.2 kip and 7.5 kip, when a marked stiffness reduction could be observed together with increasing crack widths. Hairline cracks also developed in the slab without affecting the overall stiffness. Diagonal tension failure [Figure 9(b)] occurred at a load of 12.3 kip [Figure 13(b)], thus in good agreement with the analytical prediction of 12.2 kip, which satisfies the AASHTO (2002) requirement scaled from 10.0 kip to 9.0 kip to account for the height of the load line increased from the required 21.5 in. to 24.0 in.

From a structural standpoint, the design was adopted since: the code-mandated requirements were met using nominal 4000 psi concrete, commonly adopted for bridge decks and railings; the transverse strength could be accurately predicted; upon failure, the post remained attached to the slab and could still carry load up to 6.3 kip while undergoing large deformations (in excess of the 6.0 in. stroke of the actuator).

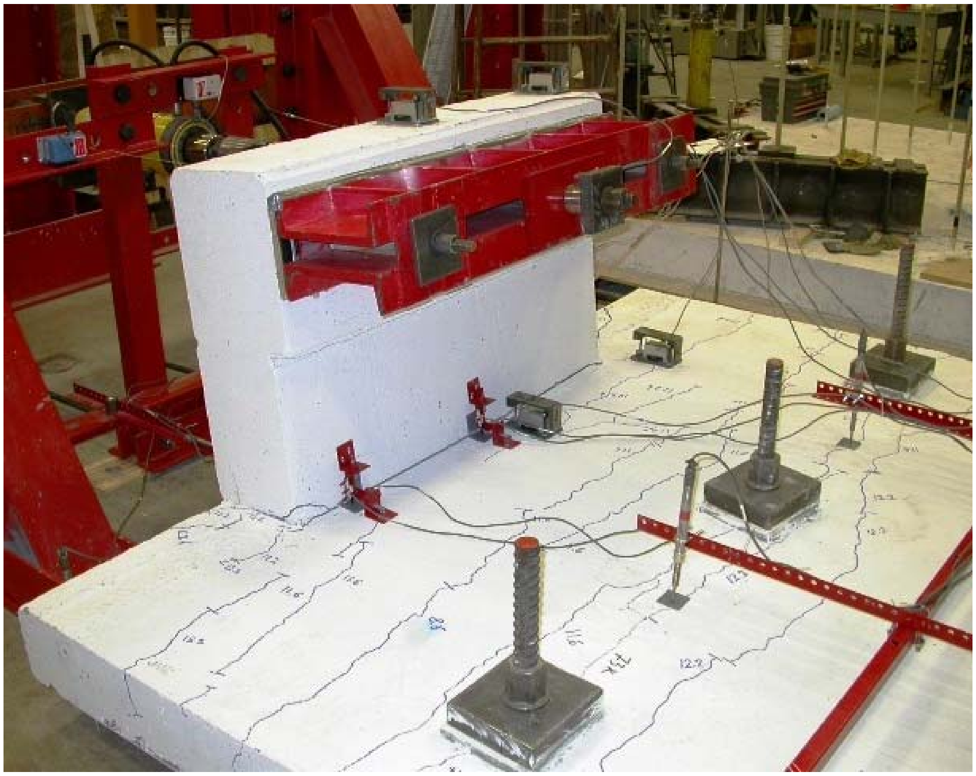


(a)

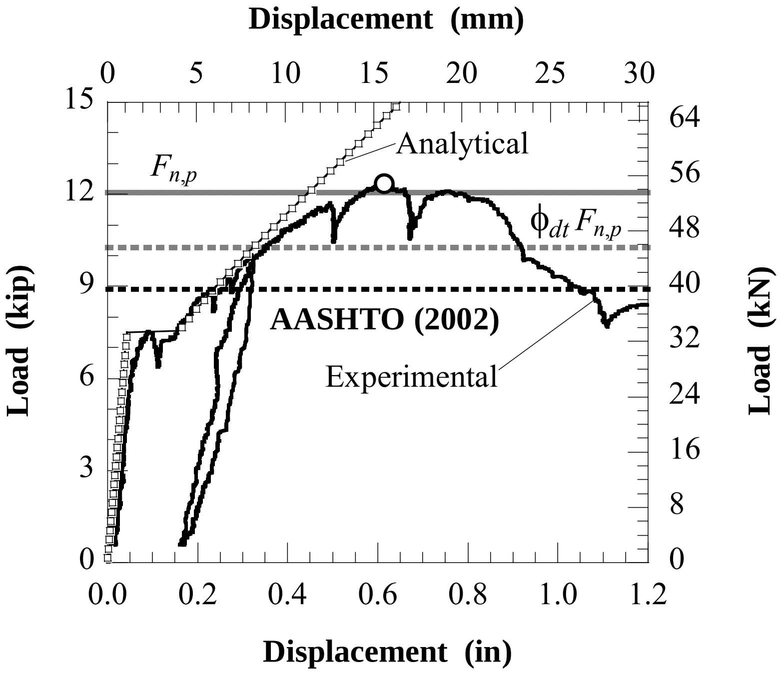


(b)

Figure 12 – Post-deck connection test setup: schematic (a) and photograph (b). Dimensions in in. (mm).



(a)



(b)

Figure 13 – Proof test of post-deck connection: photograph (a) and load-maximum transverse displacement response (b).

6. FIELD IMPLEMENTATION

The reconstruction of Bridge 1482301 was completed in November 2005. The field implementation project was developed as a joint effort between academia, government, and industry parties, including the Missouri University of Science and Technology, the University of Wisconsin-Madison, the Greene County Highway Department (bridge owner), Great River Engineering (engineer of record), Hartman Construction (contractor), Master Contractors LLC (responsible for the assemblage and installation of the MKCR GFRP bar cages), Strongwell Corp. (manufacturer of the GFRP deck panels), and Hughes Brothers, Inc. (manufacturer of the GFRP bars for the MKCR reinforcement), and with the assistance of the Missouri Department of Transportation (MoDOT).

The construction operations were planned with the contractor parties to minimize the amount of time and work. Construction of the deck and railing from installation of the SIP panels to casting of the open-post railing is documented in Figure 14. The job was completed in five days, instead of the typical 2-3 weeks needed for similar steel RC bridges built by the same contractor:

Day 1 – All 18 deck panels were set in place and anchored to the steel girders by six workers in a total of six hours.

Day 2 – The prefabricated post cages were inserted into the cut-out pockets in the SIP panels; the deck details were formed in a traditional manner using plywood, including expansion joints, chamfers, and drip edges; finally, the finishing machine was set.

Day 3 – The deck was cast and finished in a total of two and a half hours. The roadway crown was formed using the deck finishing machine, which allowed to avoid either impractical cambering of the FRP panels, or time-consuming preparation of haunches.

Day 4 – The prefabricated longitudinal beam cages of the open-post railings were mounted, and the railings formed. Since the temperature was below the minimum of 35ºF required by the Greene County Highway Department to proceed with casting, the operation was delayed.

Day 5 – Casting of the railing was completed.

The bridge was successfully load tested in July 2006 and has been in service for over two years.

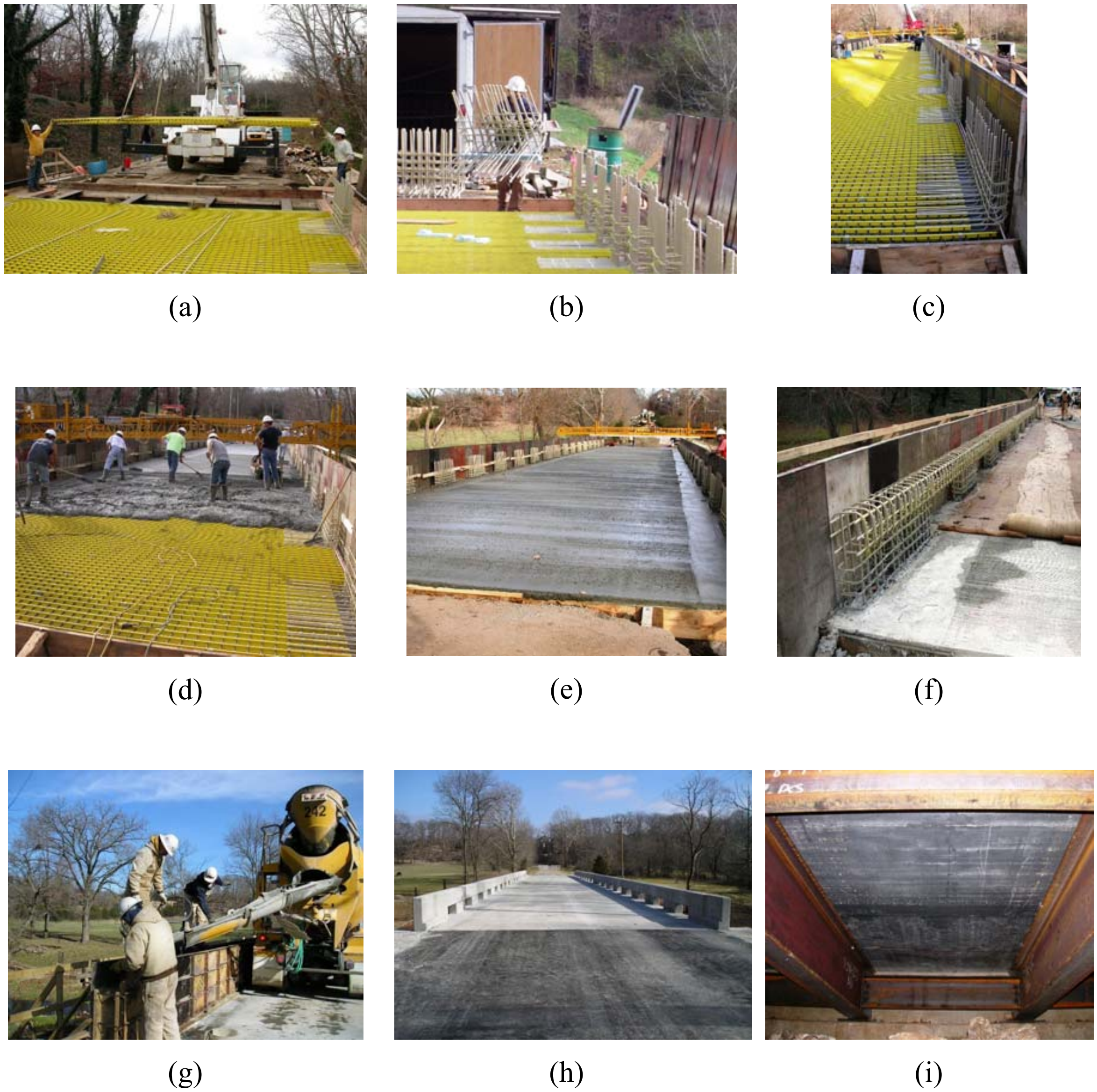


Figure 14 – Bridge redecking operations: installation of SIP panels (a); mounting of post cages (b-c); deck casting (d) and finishing (e); mounting of railing beam cages (f); casting of railing (g); and finished superstructure (h-i).

7. REFERENCES

ACI Committee 440, “Guide for the design and construction of concrete reinforced with FRP bars,” ACI 440.1R-03, American Concrete Institute, Farmington Hills, MI, 2003, 42 pp.

American Association of State Highway and Transportation Officials (AASHTO), “Standard specifications for highway bridges,” 17th edition, AASHTO, Washington, D.C., 2002.

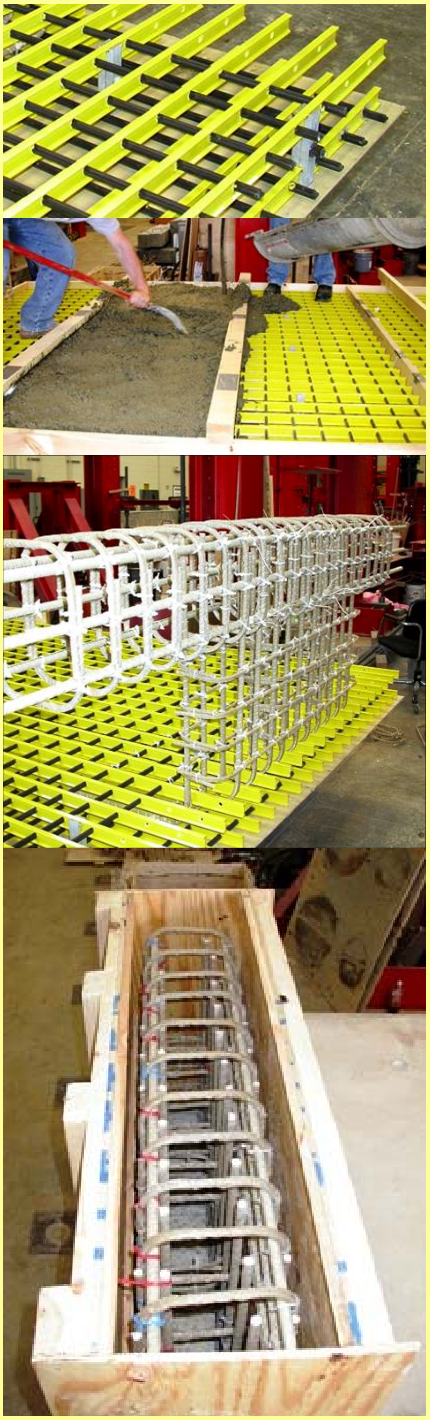
American Association of State Highway and Transportation Officials (AASHTO), “Load and resistance factor design (LRFD) bridge design specifications,” 2nd edition, AASHTO, Washington, D.C., 1998.

Nilsson, I. H. E., and Losberg, A., “Reinforced concrete corners and joints subjected to bending moment,” *Journal of the Structural Division*, V. 102, No. ST6, Jun. 1976, pp. 1229-1254.

Ringelstetter, T.E., Bank, L.C., Oliva, M.G., Russell, J.S., Matta, F., and Nanni, A., “Cost-Effective, Structural Stay-in-Place Formwork System of Fiber-Reinforced Polymer for Accelerated and Durable Bridge Deck Construction,” Transportation Research Record, 1976, 2006, pp. 183-189.

APPENDIX A – PROJECT SPECIAL PROVISIONS

BHO-B039(24) BRIDGE 14802301, GREENE COUNTY, MO



PLAN SUBMITTAL TO THE MISSOURI DEPARTMENT OF TRANSPORTATION

Glass Fiber Reinforced Polymer (GFRP) Reinforcement for Concrete Deck and Rail System: Material and Construction Specifications

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LIST OF FIGURES

LIST OF TABLES

GLASS FIBER REINFORCED POLYMER (GFRP) REINFORCING BARS

A. Materials and Manufacturing

A.1. Description

GFRP Reinforcing Bars shall consist of furnishing and placing fiber reinforced polymer bars as shown on the plans and required by the contract. All GFRP Reinforcing Bars will be supplied by Greene County, MO.

A.2. Classification of Constituent Materials and Manufacturing

Fibers - Any commercial grade E-glass is permitted. The fiber may be in the form unidirectional rovings or tows of any size or weight, or can be in the form of stitched, woven, braided or non-woven fabrics, or mats of any size or weight. Fiber sizings and coupling agents shall be appropriate for the resin system used. The manufacturer of the fiber itself and the manufacturer of any fabrics or mats must be reported.

A.3. Resins

Any commercial grade vinylester thermosetting polymer resin is permitted. A vinylester resin is defined as a thermosetting reaction product of an epoxy resin with an unsaturated acid, usually methacrylic acid, which is then diluted with a reactive monomer, usually styrene (ASTM C904). The base polymer in the resin system may not contain any polyester. Blending of vinylester resins is permitted. The manufacturer of the polymer resin must be reported. Styrene may be added to the polymer resin during processing. The amount of styrene, as a weight percentage of the polymer resin, added during processing shall be reported. Added styrene shall be less than 10% by weight of resin (pph resin).

A.4. Fillers

Commercial grade inorganic fillers such as kaolin clay, calcium carbonate, and alumina trihydrate are permitted and shall not exceed 20% by weight of the polymer resin constituent. The type and manufacturer of the inorganic filler must be reported. Commercial grade additives and process-aids, such as, release agents, low-profile shrink additives, initiators, promoters, hardeners, catalysts, pigments, fire-retardants, and ultra-violet inhibitors are permitted as appropriate for the processing method. Shrink additives shall be less than 10% by weight of the polymer resin. Commercial grade inorganic or organic non-woven surfacing mats or veils are permitted.

A.5. Manufacturing Process

FRP materials must be produced using the pultrusion manufacturing process or by a process approved by the Engineer-of-Record (Engineer). All FRP material parts provided to the job site must be produced using the same pultrusion die and in the same production lot. Manufacturer shall report upon request the maximum internal pultrusion die temperature measured by thermocouple. Manufacturer shall report the date of production and the lot size.

A.5.3. Straight Bars

Straight bars are cut to a specified length from longer stock lengths in a fabricator’s shop or at the manufacturing plant.

A.5.4. Bent Bars

Bending FRP rebars made of thermoset resin should be carried out before the resin is fully cured. After the bars have cured, bending or alteration is not possible due to the inflexibility or rigid nature of a cured FRP bar. Because thermoset polymers are highly cross-linked, heating the bar is not allowed as it would lead to a decomposition of the resin, thus a loss of strength in the FRP.

The strength of bent bars varies greatly for the same type of fiber, depending on the bending technique and type of resin used. The strength of the bent portion should be determined based on tests performed in accordance with recommended methods cited in the literature. Bars in which the resin has not yet fully cured can be bent, but only according to the manufacturer’s specifications and with a gradual transition, avoiding sharp angles that damage the fibers.

A.6. Fiber Architecture

Three classes of fiber architecture are permitted for the FRP material. The division into the three classes depends on the total fiber volume fraction (expressed as a percent of the total material volume) and the total volume of continuous longitudinal fiber (expressed as a percent of the total fiber volume) along the longitudinal axis of the laminate (also called the 0 degree axis.) Laminates cut from a three-dimensional part must have the same longitudinal axis.

A.6.1. Class 1 FRP Material

The material must have a total fiber volume fraction of 55% or greater and must have a total longitudinal fiber volume (relative to the total fiber volume) of 95% or greater.

A.6.2. Class 2 FRP Material

The material must have a total fiber volume fraction of 40% or greater and must have a total longitudinal fiber volume (relative to the total fiber volume) of 75% or greater.

A.6.3. Class 3 FRP Material

The material must have a total fiber volume fraction of 40% or greater and must have a total longitudinal fiber volume (relative to the total fiber volume) of 40% or greater.

Non-woven continuous filament mats (CFM) of the primary reinforcement type are included in the total fiber volume fraction count. Only continuous fibers in the longitudinal direction are included in the total longitudinal fiber volume.

A.7. Classification

The material is classified on the laminate level according to its fiber type, resin type and fiber architecture. Laminates having at least three-dimensional transversely isotropic symmetry or two-dimensional (in-plane) orthotropic symmetry are permitted. For in-plane orthotropy laminates must be balanced and symmetric. The classification is applied to every distinct laminate thickness and fiber architecture within the FRP part. The classification nomenclature is as follows: fiber type, polymer resin type, class (e.g., GV2 designates a glass/vinylester class 2 FRP material).

Manufacturer shall report items detailed above in a tabular form as shown in Table A-1 for the FRP materials produced.

Table A-1: Reporting requirements for constituent materials of GFRP bars

ITEM	TYPE	MANUFACTURER	SPECIAL REQUIREMENTS
Fiber	E-glass roving type	E-glass roving manufacturer	NA
	E-glass fabric type(s)	E-glass fabric manufacturer	NA
	E-glass mat type	E-glass mat manufacturer type	NA
Veil	Surface veil type	Surface veil manufacturer	NA
Resin	Vinylester type(s)	Vinylester manufacturer	NA
	Styrene type	Styrene manufacturer	pph (less than 10 pph resin)
Filler	Filler type	Filler manufacturer	pph (less than 20 pph resin)
Additives	Shrink additive type	Shrink additive manufacturer	pph (less than 10 pph resin)
Process	Pultusion die temperature	NA	NA
	Date of production	NA	NA
	Lot size	NA	NA

A.8. Physical and Mechanical Properties

A.8.1. Full-Section Testing

The manufacturer shall provide full-section longitudinal strength and stiffness properties for all sizes of GFRP bars specified in the plans. Full-section tests shall be conducted on as-produced lengths of GFRP rebar and require specialized end anchorages and gripping devices. A

minimum of three full-section tests is required for each size bar. Longitudinal tensile strength and stiffness of GFRP bars tested in full-section shall meet or exceed values shown in
Table A-2.

Table A-2: Limiting full-section properties for GFRP bars

BAR SIZE	NOMINAL DIAMETER	STRENGTH	STIFFNESS
#3	0.375 in (9.53 mm)	110 ksi (760 MPa)	5.92 Msi (40.8 GPa)
#4	0.500 in (12.7 mm)	100 ksi (690 MPa)	5.92 Msi (40.8 GPa)
#5	0.625 in (15.9 mm)	95 ksi (655 MPa)	5.92 Msi (40.8 GPa)
#6	0.750 in (19.1 mm)	90 ksi (620 MPa)	5.92 Msi (40.8 GPa)
#7	0.875 in (22.2 mm)	85 ksi (586 MPa)	5.92 Msi (40.8 GPa)
#8	1.000 in (25.4 mm)	80 ksi (550 MPa)	5.92 Msi (40.8 GPa)
#10	1.25 in (31.8 mm)	75 ksi (517 MPa)	5.92 Msi (40.8 GPa)

A.8.2. Coating for Bond to Concrete

FRP Rebars shall have a proprietary coating applied to their entire outside surface to ensure bond to the concrete. FRP rebars shall have a bond strength of not less than 1450 psi (10 MPa) when measured in a direct pull-out test.

A.8.3. Sealing of Cut-Ends

Manufacturer shall seal all cut-ends of the pultruded FRP rebars with an epoxy or vinylester resin prior to shipment.

A.9. Quality Assurance

A.9.1. GFRP Reinforcing Bars

Quality control should be carried out by lot testing of GFRP bars. The manufacturer should supply adequate lot or production run traceability. Tests conducted by the manufacturer or a third-party independent testing agency can be used.

All tests should be performed using the recommended test methods cited in the literature. Material characterization tests that include the items detailed in Table A-1 and in Table A-2 should be performed at least once before and after any change in manufacturing process, procedure, or materials.

The manufacturer should furnish upon request a certificate of conformance for any given lot of GFRP bars with a description of the test protocol. An authorized company representative shall sign, date and certify all test reports. Two copies of the certified test reports shall be provided at the time of material delivery. Reports and certifications shall be provided by the manufacturer to the Engineer for approval.

A.10. Referenced ASTM Methods

Standards of the American Society of Testing and Materials referred to in this paper are listed below. All standards appear in the current annual edition of ASTM standards published by the American Society of Testing and Materials, West Conshohocken, PA.

C904 - Standard Terminology Relating to Chemical-Resistant Nonmetallic Materials
D570 - Standard Test Method for Water Absorption of Plastics
D618 - Standard Practice for Conditioning Plastics for Testing
D638 - Standard Test Method for Tensile Properties of Plastics
D695 - Standard Test Method for Compressive Properties of Rigid Plastics
D696 - Standard Test method for Coefficient of Linear Thermal Expansion of Plastics between -30° and 30° with a Vitreous Silica Dilatometer
D2344 - Standard Test Method for Short-Beam Strength of Polymer Matrix Composite Materials and Their Laminates
D2583 - Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a Barcol Impressor
D2584 - Standard Test Method for Ignition Loss of Cured Reinforced Resins
D3039 - Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials
D3171 - Standard Test Method for Constituent Content of Composite Materials
D3410 - Standard Test Method for Compressive Properties of Polymer Matrix Composite Materials with Unsupported Gage Section by Shear Loading
D3418 - Standard Test Method for Transition Temperatures of Polymers By Differential Scanning Calorimetry
D3916 - Standard Test Method for Tensile Properties of Pultruded Glass-Fiber-Reinforced Plastic Rod
D3917 - Standard Specification for Dimensional Tolerance of Thermosetting Glass-Reinforced Plastic Pultruded Shapes
D4475 - Standard Test Method for Apparent Horizontal Shear Strength of Pultruded Reinforced Plastic Rods By The Short-Beam Method
D5083 - Standard Test Method for Tensile Properties of Reinforced Thermosetting Plastics Using Straight-Sided Specimens
E1356 - Standard Test Method for Assignment of the Glass Transition Temperatures by Differential Scanning Calorimetry or Differential Thermal Analysis

B. Construction Methods

B.1. Field Handling and Storage

Delivered FRP reinforcement to the job site must be unloaded using fabric slings anchored at 3rd points to avoid excessive deformation. During storage all FRP materials must be kept clean and protected from excessive exposure to moisture.

B.2. Cutting of FRP Materials

Cutting of any FRP materials must be done with the use of a toothless chop disk or diamond coated circular blade. All field cuts of the bar materials must be sealed with Concresive 1090 or a similar sealant approved by the Engineer.

B.3. Securing of FRP Reinforcement System

The FRP deck reinforcement system must be properly secured to ensure stabilization and prevention of wind uplift prior to concrete placement.

B.4. Ties

Only non-metallic ties, either plastic cable ties or coated wire can be used to tie down grid panel or reinforcement bars.

B.5. Reinforcing Bar Placement

The FRP reinforcing bars must be properly anchored against displacement before concrete placement, by tying up to and against the FRP grid panel with the use of plastic ties.

C. Method of Measurement

FRP Reinforcing Bar will be measured by the kilogram, and the quantity shall be the number of kilograms incorporated in the completed work in accordance with the requirements of the plans and specifications. The masses of the bars will be computed using a density of 125 lb/ft^3 or 2000 kg/m^3.

GLASS FIBER REINFORCED POLYMER (GFRP) REINFORCEMENT CAGE

A. General Description

A Fiber Reinforced Polymer (FRP) reinforcement cage with integral Glass FRP (GFRP) stay-in-place form (herein referred to as GridForm) shall be used in lieu of steel reinforcing bars and conventional formwork/falsework in the concrete bridge deck. A fiber reinforced polymer is a composite material consisting of fibers embedded in a polymer matrix (also referred to as a binder). Provisions for the FRP composite materials to be used in the GridForm are presented in Section F and Section G.

The GridForm panel shall be a three-dimensional (3-D) lattice of FRP pultruded members arranged in a rigid form that provides reinforcement to the concrete bridge deck in two directions near the top and bottom surfaces of the deck slab (perpendicular to the traffic flow shall be called the "primary" reinforcement direction for the GridForm panel, and parallel to the traffic flow shall be called the "secondary" reinforcement direction of the GridForm panel). The GridForm panel consists of two planar pultruded grids (called top and bottom grids) of FRP pultruded members arranged in an open lattice form (similar to a COTS pultruded grating) and connected together to form an integral 3-D structure, together with a FRP plate that serves as a bottom plate form to support the weight of wet concrete and construction live loads imposed during deck placement and curing.

The individual pultruded members in the primary direction, called "main" bars, of the GridForm shall be "I" shaped pultruded bars. The individual pultruded members in the secondary direction, called "cross-rods," of the GridForm shall be multi-part pultruded elements.

The individual pultruded bars in the top and bottom planar grids shall be connected together with interlocking cross-rods so as to provide for in-plane force transfer at their intersections to ensure anchorage in the surrounding concrete. Continuous force transfer along the bars is not required. A portion of the longitudinal fiber reinforcement (rovings) in the individual bars shall be continuous through the intersections. Only the net area of the main bar that contains the fiber reinforcement that is continuous through the intersections shall be used to determine the bar area. The planar grids of the GridForm may be connected using a proprietary cross-rod connecting system. The separate parts of the cross-rods shall be adhesively bonded to each other, and to the main bars to seal the main bars where holes are drilled for the cross-rods. No metallic parts may be used for connecting the individual parts.

The two planar grids shall be separated and connected by pultruded "shear connectors" normal to the plane of the two planar grids that serve to hold the two planar grids together in a rigid form and also serve to transfer shear forces between the two planar grids. The shear connectors shall be vertical. The two planar grids may be connected to form the 3-D cage using a proprietary connection system. The shear connectors shall be adhesively bonded to the planar grids. No metallic parts may be used for connecting the individual parts.

The FRP plate shall be a minimum of one-eighth (1/8) inch thick plate glass/vinylester pultruded material. The plate shall be continuously bonded to the bottom grid by a two-part epoxy system.

The GridForm panel thus described is a single unit that is lifted and placed directly on the bridge girders. Concrete shall be placed over the GridForm panel and finished at the top surface. The GridForm panel is required to be sufficiently rigid both in its plane and out of its plane to be lifted at specified pick-points and also to be able to withstand construction loads prior to placement of the concrete. The individual members of the GridForm panel must be spaced in such a manner as to permit the concrete to "flow" around all elements during placement so as to completely encase the GridForm and create a void-free bridge deck. Performance requirements for the GridForm panel are described in Section G.5.

The GridForm panel must be produced and delivered to the job-site as a single unit that extends from one edge to the other edge of the bridge deck in its primary direction without requiring any in-field connections or splices. The length and width of the GridForm panels shall be as shown in the plans. In the secondary direction of the GridForm, splices shall be provided between individual GridForm panels as shown in the plans. The length of the overlap shall be at least 12 inches. Bars in the top and bottom planar grids in the splice region must be anchored by no less than three perpendicular bars (i.e., main bars) and must be supported by shear connectors and chairs as shown in the plans.

B. Geometric Requirements

The GridForm described above shall conform to the following:

B.1. Primary Reinforcement

The primary GridForm pultruded "main" bars in the top and bottom planar grids shall be commercially produced 1.5 inch high "I" bars and have a net cross-sectional area of no less than 0.32 in^2 of GV2 material type. Main bars shall be spaced at 4 in on-center. Main bars may be of any color.

B.2. Secondary Reinforcement

The secondary GridForm pultruded bars in the top and bottom planar grids shall be commercially produced multipart bars (cross-rods) and have a total cross-sectional area of no less than 0.21 in^2 of GV1 material. Cross-rods shall be spaced at 4 inches on-center. Cross-rods may be of any color.

B.3. Shear Connectors

The shear connectors between the top and bottom planar grids shall be spaced at a regular uniform spacing in both the primary and the secondary directions. The spacing of the shear connectors in the primary and the secondary directions does not have to be the same. The shear connectors may be vertical bars, inclined bars or a combination of the two. The shear connectors may be of any color.

B.4. Concrete Cover

The location of the GridForm main bars in the top planar grid shall allow for a clear concrete cover of 1.5 inches.

C. Sole Supplier

Greene County, MO, will provide the GridForm panels specified within this provision.

D. Special Coarse Aggregate for FRP Reinforced Bridge Deck

The maximum coarse aggregate size in the bridge deck shall be 1/2 inches or as directed by the Engineer-of-Record (Engineer).

E. Submittals.

E.1. Shop Drawings

Shop drawings of the entire GridForm panel and drawings of the individual components shall be submitted for approval prior to fabrication of the GridForm panels. The drawings shall indicate the type of composite material for each part of the cage. The method of connection between the parts must be clearly described on the shop drawings. The cross-sectional areas and centroidal axes of all parts shall be shown on cross-sectional drawings of the parts.

E.2. Lifting and Placement Plan

The methodology for lifting the GridForm panel from a truck bed shall be submitted for approval. If cranes, specialized devices and/or equipment (strong backs, spreader bars, lifting straps, etc) are required for the lifting and placement process, these devices must be specified. Shop drawings of the devices must be provided for approval. Locations of special pick-points, if necessary, must be shown on a sketch of the GridForm panel and marked on the GridForm panel before shipping. It is the general contractor’s responsibility to ensure that the contractor can readily place the GridForm panel on the bridge girders with existing equipment.

No wood inserts, spacers, dunnage, etc. shall be placed between the top and bottom grids.

E.3. Number of Copies

The general contractor shall anticipate a minimum of 4 copies of blue or black-line prints for shop drawing submittals.

E.4. Timing of Submittals

Deliver each shop drawing, lifting and placement plan, and mock-up section submittals requiring approval in time to allow for a minimum of 7 working days for review and processing, not including re-submittals if necessary. Submit all material test reports (see Section G.6) upon delivery of FRP materials to the jobsite. Failure of the general contractor in these respects will not be considered as grounds for an extension of the contract time. If a submittal must be delayed for coordination with other submittals not yet submitted, the Engineer may at his option either return the submittal with no action or notify the general contractor of the other submittals which must be received before the submittal can be reviewed.

E.5. Approval of Submittals

Contractor shall not commence work which requires review of any submittals until receipt of returned submittals with an acceptable action.

F. Materials and Manufacturing

F.1. Fibers

Any commercial grade E-glass is permitted. The fiber may be in the form of unidirectional rovings or tows of any size or weight, or can be in the form of stitched, woven, braided or non-woven fabrics, or mats of any size or weight. Fiber sizings and coupling agents shall be appropriate for the resin system used. The manufacturer of the fiber and the manufacturer of any fabrics or mats must be reported.

F.2. Resins

Any commercial grade vinylester thermosetting polymer resin is permitted. A vinylester resin is defined as a thermosetting reaction product of an epoxy resin with an unsaturated acid, usually methacrylic acid, which is then diluted with a reactive monomer, usually styrene (ASTM C904). The base polymer in the resin system shall not contain any polyester. Blending of vinylester resins is permitted. The manufacturer of the polymer resin must be reported. Styrene may be added to the polymer resin during processing. The amount of styrene, as a weight percentage of the polymer resin, added during processing shall be reported. Added styrene shall be less than 10% by weight of resin (pph resin).

F.3. Fillers

Commercial grade inorganic fillers such as kaolin clay, calcium carbonate, and alumina trihydrate are permitted and shall not exceed 20% by weight of the polymer resin constituent. The type and manufacturer of the inorganic filler must be reported. Commercial grade additives and process-aids, such as, release agents, low-profile shrink additives, initiators, promoters, hardeners, catalysts, pigments, fire-retardants, and ultra-violet inhibitors are permitted as appropriate for the processing method. Shrink additives shall be less than 10% by weight of the polymer resin. Commercial grade inorganic or organic non-woven surfacing mats or veils are permitted.

F.4. Manufacturing Process

FRP materials must be produced using the pultrusion manufacturing process. All FRP material parts provided to the job site must be produced using the same pultrusion die and in the same production lot. The manufacturer shall report maximum internal pultrusion die temperature measured by thermocouple. The manufacturer shall report the date of production and the lot size.

F.5. Fiber Architecture

Three classes of fiber architecture are permitted for the FRP material. The division into the three classes depends on the total fiber volume fraction (expressed as a percent of the total material volume) and the total volume of continuous longitudinal fiber (expressed as a percent of the total fiber volume) along the longitudinal axis of the pultruded part (also called the 0 degree axis.) Non-woven Continuous Filament Mats (CFM) (also known as Continuous Strand Mats (CSM)) are included in the total fiber volume fraction count. Only continuous fibers in the longitudinal direction are included in the total longitudinal fiber volume. The fiber architecture classes are defined as the following:

F.5.1. Type 1 FRP Material
The material shall have a total fiber volume fraction of 55% or greater and shall have a total longitudinal fiber volume of (relative to the total fiber volume) 90% or greater.

F.5.2. Type 2 FRP Material
The material shall have a total fiber volume fraction of 45% or greater and shall have a total longitudinal fiber volume of (relative to the total fiber volume) 75% or greater.

F.5.3. Type 3 FRP Material
The material shall have a total fiber volume fraction of 40% or greater and shall have a total longitudinal fiber volume of (relative to the total fiber volume) 50% or greater.

F.6. Classification Nomenclature

The material shall be classified according to its fiber type, resin type and fiber architecture. Parts having at least three-dimensional transversely isotropic symmetry or two-dimensional (in-plane) orthotropic symmetry are permitted. For in-plane orthotropy, laminates shall be balanced and symmetric. The classification is applied to every distinct laminate thickness and fiber architecture within the FRP part. The classification nomenclature is as follows: fiber type, polymer resin type, architecture type (e.g., GV2 designates a glass/vinylester type 2 FRP material).

Table F-1: Reporting requirements for constituent materials of GridForm

ITEM	TYPE	MANUFACTURER	SPECIAL REQUIREMENTS
Fiber	E-glass roving type	E-glass roving manufacturer	NA
	E-glass fabric type(s)	E-glass fabric manufacturer	NA
	E-glass mat type	E-glass mat manufacturer type	NA
Veil	Surface veil type	Surface veil manufacturer	NA
Resin	Vinylester type(s)	Vinylester manufacturer	NA
	Styrene type	Styrene manufacturer	pph (less than 10 pph resin)
Filler	Filler type	Filler manufacturer	pph (less than 20 pph resin)
Additives	Shrink additive type	Shrink additive manufacturer	pph (less than 10 pph resin)
Process	Pultusion die temperature	NA	NA
	Date of production	NA	NA
	Lot size	NA	NA

Manufacturer shall report items detailed above in a tabular form as shown in Table F-1 for the FRP materials produced. Information shall be provided separately for each distinct pultruded part in the GridForm panel.

F.7. Classification of Parts for GridForm

The FRP materials to be used in the GridForm are classified as glass/vinylester material type GV1, GV2 or GV3. Since the GridForm is assembled from distinct parts, different fiber architecture types are permitted as follows:

F.7.1. Primary Reinforcement
Primary load-bearing bars (I-bars) of the GridForm shall be of GV2 material and shall consist of unidirectional roving, continuous strand mat and surfacing veil.

F.7.2. Secondary Reinforcement
Secondary connecting cross-rods of the GridForm shall be of GV1 material and shall consist of unidirectional glass roving and surfacing veil only.

F.7.3. Shear Connectors
Shear connectors and may be of material type GV1, GV2 or GV3 material.

F.7.4. Bottom Plate
Bottom plate shall be of glass/vinylester pultruded material.

F.8. Physical and Mechanical Properties

Physical and mechanical tests shall be conducted on coupons extracted from the as-produced FRP materials according to the ASTM tests listed below. Tests shall be conducted on material produced specifically for this project. Test data shall be reported for each component (i.e. main bar, cross rod, shear connector, and bottom plate. In cases where the required sized coupon can not be extracted from the cured part, a “test property laminate” shall be used. The test property laminate shall have the same fiber architecture to the laminate within the part and shall be manufactured using the same processing method and under the same conditions as the part. For mechanical property measurements, specimens should be conditioned prior to testing (pre-conditioned) according to Procedure A of ASTM D618. For all mechanical measurements, no fewer than five (5) specimens shall be tested. For physical measurements the number of tests shall be as per the applicable ASTM specification. Individual test data values, mean and coefficient of variation shall be reported for all tests. Deviations from standard test methods shall be reported.

As-produced short-term properties for GV1, GV2 and GV3 materials shall meet or exceed the values shown in Table F-2. The GridForm manufacturer shall conduct tests and provide a certified report with data shown in Table F-2. Test coupons for GV2 material must be cut from the I-bar web for longitudinal property tests and physical tests, and from an equivalent test property laminate for transverse property tests. Test coupons for mechanical tests for GV1 material may be cut from equivalent test property laminates (e.g., unidirectional rods). Test coupons for mechanical tests for GV3 materials may be machined from parts to achieve required ASTM test specimen dimensions or may be cut from test property laminates.

Table F-2: Limiting full-section properties for GridForm materials

Material Property	ASTM Test	GV1 Material	GV2 Material	GV3 Material
Mechanical Property				
Strength Property				
Longitudinal Tensile Strength (min)	D3039, D5083, D638, D3916	95.0 ksi	80.0 ksi	40.0 ksi
Transverse Tensile Strength (min)	D3039, D5083, D638	NA	4.0 ksi	4.0 ksi
Longitudinal Compressive Strength (min)	D3410, D695	75.0 ksi	60.0 ksi	30.0 ksi
Transverse Compressive Strength (min)	D3410, D695	NA	10.0 ksi	10.0 ksi
Long. Short Beam Shear Strength (min)	D2344, D4475	6.0 ksi	5.0 ksi	5.0 ksi
Stiffness Property				
Longitudinal Tensile Modulus (min)	D3039, D5083, D638	5.5 Msi	4.5 Msi	2.5 ksi

MATERIAL PROPERTY	ASTM TEST	GV1 MATERIAL	GV2 MATERIAL	GV3 MATERIAL
Longitudinal Compressive Modulus (min)	D3410, D695	NA	4.0 Msi	2.0 ksi
In-Plane Shear Stiffness (min)	D5379	NA	0.40 Msi	0.30 Msi
Major (Longitudinal) Poisson Ratio (min)	D3039, D5083, D638	NA	0.25	0.25
Physical Property				
Fiber Volume Fraction (min)	D3171, D2584	55%	45%	40%
Barcol Hardness (min)	D2583	50	50	50
Glass Transition Temperature (min)	E1356, D3418, E1640	203°F (95°C)	203°F (95°C)	203°F (95°C)
Water absorption (immerse at 50°C for 48 hours)	D570	0.50%	0.75%	1.00%
Long. Coeff. of Thermal Expansion (max)	D696, E831	6 x 10^{-6}/°F	6 x 10^{-6}/°F	6 x 10^{-6}/°F
Trans. Coeff. of Thermal Expansion (max)	D696, E831	NA	30 x 10^{-6}/°F	30 x 10^{-6}/°F

G. Special Requirements.

G.1. Production and Assembly

All distinct GridForm components must be produced using the same pultrusion operation, using identical shaped dies, and in the same production lot. The manufacturer shall report the date and time of production of the GridForm elements.

G.2. Product Marking

Each GridForm panel shall be marked with the designation of this provision, manufacturer’s name, date of production, and manufacturer’s part identification number. Markings shall occur at least every 8.0 ft in linear measurement or at least every 100 ft^2 of surface area.

G.3. Dimensional Tolerance

Pultruded GridForm elements shall conform to ASTM 3917.

G.4. Sealing of Cut-Ends

Manufacturer shall seal all cut ends of the pultruded GridForm elements with an epoxy or vinylester resin prior to shipment.

G.5. Performance Requirements

G.5.1. Scope

The 3-D FRP reinforcing cage shall meet the following performance requirements to ensure its ability to maintain integrity during the bridge construction process. The requirements are specifically defined to achieve a cage system that can withstand forces occurring during lifting and handling at the construction site and during casting of the concrete deck. Test specimens used to prove performance criteria shall be constructed of materials and in a fashion that represents an actual bridge GridForm. The assembly shall be representative of normal workmanship. It shall be constructed using full-scale materials.

G.5.2. Lifting Requirements

The GridForm may be lifted by crane in moving from the truck and placing on the bridge girders. The GridForm must have sufficient strength to resist picking loads, the locations of which shall be coordinated with the general contractor.

G.5.3. Construction Loads: Vertical

Under vertical construction loading, before and during placement of the wet concrete, the GridForm must be able to resist vertical construction loads. In this condition the GridForm system will be supported on its integral chair above the formwork. To ensure satisfactory performance, a sub-assembly of the GridForm must meet the following strength and deflection criteria when loaded as specified.

G.5.3.1. Test Sub-Assembly

This performance criterion refers to the behavior of a portion of the full-size GridForm which is referred to as the “test assembly”. The spacing of support chairs in the primary direction of the cage is denoted as S_P. The spacing of the support chairs are denoted as S_S in the secondary direction of the GridForm. The test sub-assembly shall consist of a portion of the full size GridForm that has a length of 1.5 times S_S and a width of 1.5 times S_P. The support chairs shall be placed at 0.25 times S_S or S_P from each edge of the sub-assembly, as illustrated in Figure G-1.

G.5.3.2. Loading Criteria

The sub-assembly must resist a load of 500 lbs. applied over a 1-foot square area on the top surface of the 3-D cage. The load shall be applied at the center of the sub-assembly, mid-way between the support chairs as illustrated in Figure G-1.

G.5.3.3. Acceptance Criteria

The 3-D reinforcing cage must have sufficient strength to resist the 500 lb. vertical load on the top surface without exceeding 50% of the FRP ultimate stress capacity in any portion of the cage. The reinforcing cage must have sufficient stiffness to resist the 500 lb. vertical load without developing a downward deflection of the top surface, relative to the support below the feet, exceeding 0.4 inches directly under the load as illustrated in Figure G-2. Upon removal of the 500 lb. load the top surface must return to its original position +/– 0.05 inches.

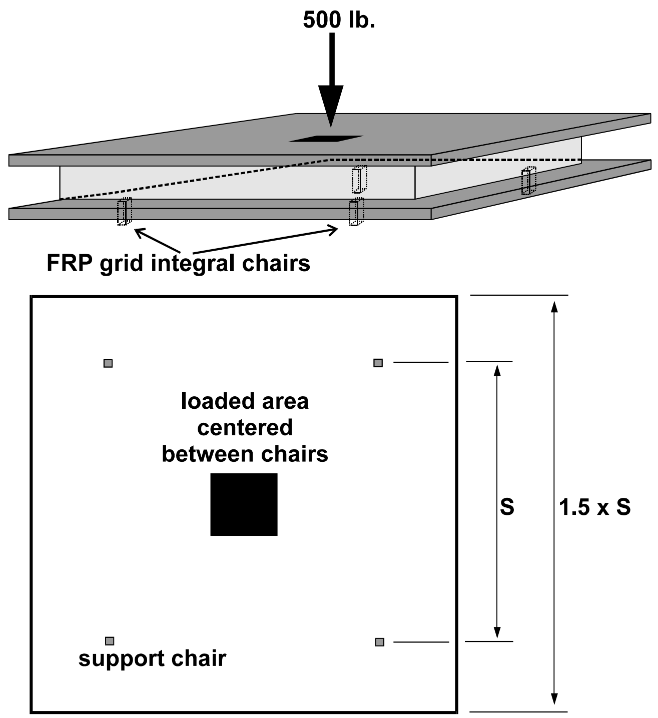

Figure G-1: Schematic of test assembly for measuring performance under vertical construction loads.

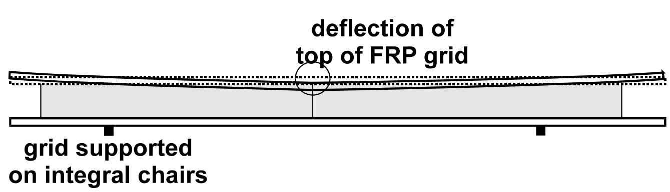

Figure G-2: Deflection limit provided for top surface of GridForm under vertical loading.

G.5.4. Construction loads: Lateral

The GridForm shall resist lateral loads applied to the top surface without excessive deformation or collapse during construction. To ensure satisfactory performance, a sub-assembly of the cage system must meet the following strength and deflection criteria when loaded as specified.

G.5.4.1. Test Sub-Assembly

This performance criterion refers to the behavior of a portion of the full-size GridForm which is referred to as the "test assembly". The spacing of support chairs in the primary direction of the cage is denoted as S_P. The spacing of the support chairs are denoted as S_S in the secondary direction of the GridForm. The test sub-assembly shall consist of a portion of the full size GridForm that has a length of 1.5 times S_S and a width of 1.5 times S_P. The support chairs shall be placed at 0.25 times S_S or S_P from each edge of the sub-assembly, as illustrated in Figure G-1. The test shall be conducted in the primary and the secondary direction of the GridForm.

G.5.4.2. Loading Criteria

The sub-assembly must resist a combination of two loads of 200 lbs., one applied in the plane of the top surface and the other applied in the plane of the bottom surface as shown in Figure G-3. The loads are to be applied at the middle of the top and bottom edges using a 2 inch wide nylon strap wrapped around a member of the top grid or an equivalent load application method.

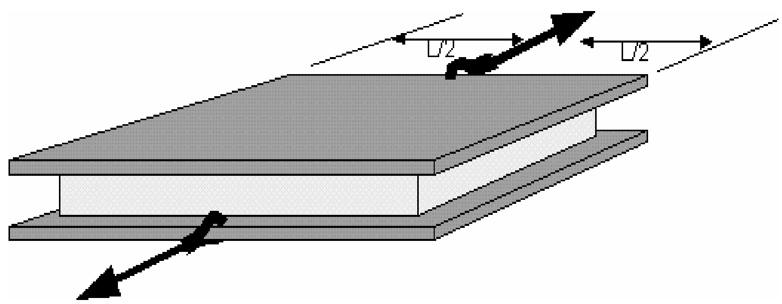

Figure G-3: Load application for lateral load test assembly.

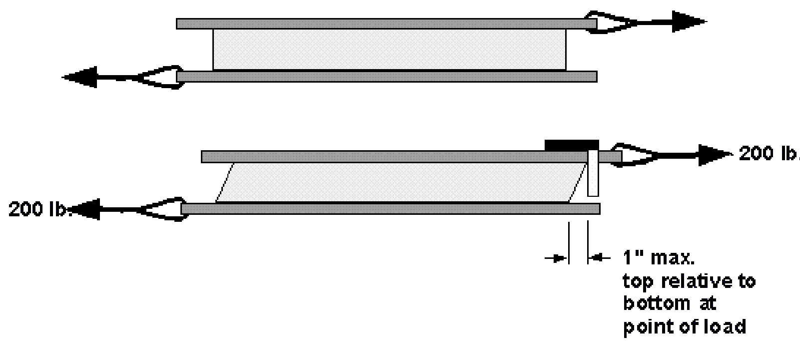

Figure G-4: Acceptance criteria for lateral load test.

G.5.4.3. Acceptance Criteria

The slope of the lateral displacement of the top surface, relative to the bottom surface, must not exceed 0.5 inch under the 200 lb. load as illustrated in

Figure G-4. The top and bottom grids must return to their original undeflected position, +/– 0.1 inches, upon removal of the 200 lb. load.

G.5.5. Construction Loads: In Plane Racking

The GridForm shall resist lateral loads applied to the top surface without excessive deformation or collapse during construction. To ensure satisfactory performance, a sub-assembly of the cage system must meet the following strength and deflection criteria when loaded as specified.

G.5.5.1. Test Sub-Assembly

This performance criteria refers to the behavior of a portion of the full-size GridForm which is here referred to as the "test assembly". The sub-assembly consists of a portion of the full-size GridForm which has a length of 6 feet and width of 6 feet, and has appropriately spaced shear connectors.

G.5.5.2. Loading Criteria

The sub-assembly must resist two loads of 100 lbs., one applied at one corner of the surface and the other applied at a diagonally opposite corner as illustrated in the plan view of Figure G-5. The loads are to be applied using a 2-inch wide nylon strap wrapped around the corners of the top grid, or an equivalent application method.

G.5.5.3. Acceptance Criteria

The change in length of either diagonal between corners of the surface after loading with 100 lb. loads must not exceed 0.5 inch or 1% of the original length before loading. Upon removal of the load the surface must return to its original undeformed shape, +/– 0.05 inches.

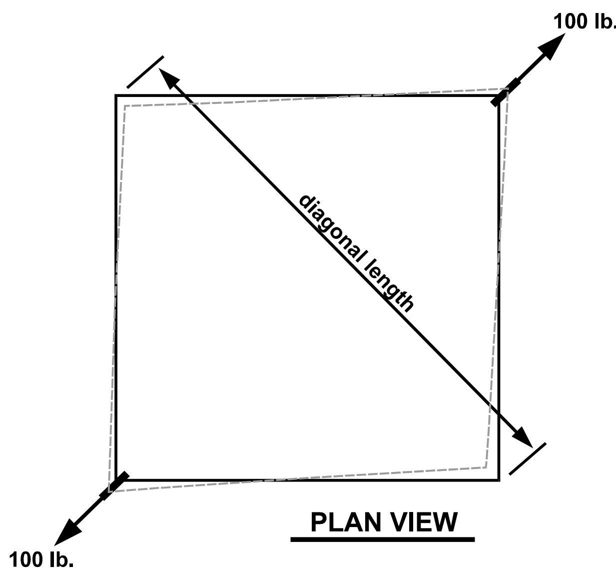

Figure G-5: Loading on sub-assembly for in-plane racking test.

G.5.6. Construction Loads: Vertical Load on Splice Overlap

The top grid layer at the lap along the edges of the GridForm shall withstand vertical loads, remain stable, and not show excessive deformation or damage. To ensure satisfactory performance, a sub-assembly of the cage shall meet the following strength, stability and deflection criteria when loaded in the specified fashion.

G.5.6.1. Test Sub-Assembly

This performance criterion refers to the behavior of a portion of the full-size GridForm which is here referred to as the "test assembly". The sub-assembly consists of a portion of the GridForm that has a width perpendicular to the lapped edge equal to that of the full scale GridForm, and a length parallel to the primary reinforcing direction of 1.5 S_P. S_P is the spacing of the support chairs in the primary direction of the cage (Figure G-1). An adjacent piece of GridForm that is relied upon to support the overhang of the lap on the top grid, must be placed adjoining the test assembly with that support feature in place.

G.5.6.2. Loading Criteria

The test assembly must resist a vertical load of 200 lbs applied over a 1-foot square on the top surface of the GridForm. The square foot loaded area is placed adjacent to the edge of the top grid where the top grid extends beyond the lower grid.

G.5.6.3. Acceptance Criteria

The GridForm shall have sufficient strength to resist the 200 lb. vertical load without exceeding 50% of the FRP ultimate stress capacity in any portion of the cage. The GridForm must have sufficient stiffness to resist the 200 lb. load without developing a downward deflection of the top surface, relative to the support surface below the support chairs, exceeding 0.4 inches in the direction of the load. Upon removal of the load the top surface shall return to its original position +/– 0.1 inches.

G.5.7. Construction Loads: Vertical Concrete Load

The GridForm shall withstand the vertical load imposed upon it due to the weight of the wet concrete, remain stable, and not show excessive deformation or damage. To ensure satisfactory performance, a sub-assembly of the cage shall meet the following strength, stability and deflection criteria when loaded in the specified fashion.

G.5.7.1. Test Sub-Assembly

This performance criteria refers to the behavior of a portion of the full-size GridForm which is here referred to as the "test assembly". The sub-assembly consists of a portion of the full-size GridForm which has a length of 8 feet and width of 8 feet, and has appropriately spaced shear connectors.

G.5.7.2. Loading Criteria

The test assembly must resist a uniform vertical load equivalent to the 7 inch concrete bridge deck applied uniformly to the bottom surface of the GridForm. The GridForm shall span 6 ft center to center of supports. It is recommended that sand or a similar material be used to provide the required load to the GridForm.

G.5.7.3. Acceptance Criteria

The GridForm shall have sufficient strength to resist the uniform vertical load equivalent to the 7 inch concrete bridge deck without exceeding 50% of the FRP ultimate stress capacity in any portion of the cage. The GridForm must have sufficient stiffness to resist the uniform vertical load without developing a downward deflection of the bottom surface, relative to the support surface, exceeding 0.5 inches in the direction of the load. Upon removal of the load the top surface shall return to its original position +/– 0.1 inches.

G.6. Reporting and Certification Requirements

The manufacturer of FRP materials shall furnish upon request a report including items detailed in Section F and provide tabulated values as in Table F-1 and Table F-2. Test data shall be supplied separately as per Section F.7. The manufacturer of the GridForm panels shall furnish upon request a certificate of conformance certifying that the GridForm panels meet the performance requirements of Section G.5. An authorized company representative shall sign, date and certify all test reports. Two copies of the certified test reports shall be provided at the time of material delivery. Reports and certifications shall be provided by the manufacturer to the Engineer for approval.

G.7. Referenced ASTM Methods

Standards of the American Society of Testing and Materials referred to in this provision are listed below. All standards appear in the current annual edition of ASTM standards published by the American Society of Testing and Materials, West Conshohocken, PA.

C904 - Standard Terminology Relating to Chemical-Resistant Nonmetallic Materials
D570 - Standard Test Method for Water Absorption of Plastics
D618 - Standard Practice for Conditioning Plastics for Testing
D638 - Standard Test Method for Tensile Properties of Plastics
D695 - Standard Test Method for Compressive Properties of Rigid Plastics
D696 - Standard Test method for Coefficient of Linear Thermal Expansion of Plastics between -30°C and 30°C with a Vitreous Silica Dilatometer
D2344 - Standard Test Method for Short-Beam Strength of Polymer Matrix Composite Materials and Their Laminates
D2583 - Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a

Barcol Impressor

D2584 - Standard Test Method for Ignition Loss of Cured Reinforced Resins

D3039 - Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials

D3171 - Standard Test Method for Constituent Content of Composite Materials

D3410 - Standard Test Method for Compressive Properties of Polymer Matrix Composite Materials with Unsupported Gage Section by Shear Loading

D3418 - Standard Test Method for Transition Temperatures of Polymers By Differential Scanning Calorimetry

D3916 - Standard Test Method for Tensile Properties of Pultruded Glass-Fiber-Reinforced Plastic Rod

D3917 - Standard Specification for Dimensional Tolerance of Thermosetting Glass-Reinforced Plastic Pultruded Shapes

D4475 - Standard Test Method for Apparent Horizontal Shear Strength of Pultruded Reinforced Plastic Rods By The Short-Beam Method

D5083 - Standard Test Method for Tensile Properties of Reinforced Using Straight-Sided Specimens

E831 - Standard Test Method for Linear Thermal Expansion of Solid Materials by Thermomechanical Analysis

E1356 - Standard Test Method for Assignment of the Glass Transition Temperatures by Differential Scanning Calorimetry or Differential Thermal Analysis

E1640 - Standard Test Method for Assignment of the Glass Transition Temperature by Dynamic Mechanical Analysis

H. Construction Methods

H.1. Handling

Lifting of GridForm panels shall be as specified in Section E.2 and Section G.5.2.

H.2. Storage

The GridForm panels shall be stored off the ground until placement on the bridge girders and in a manner to prevent foreign materials such as soil, grease, or dust, from coating the grid members. The grid surface shall be clean and dry prior to placement of concrete.

H.3. Installation

The GridForm panels shall be placed in position over the girders with a lateral dimensional tolerance of +/– 1/2 inches of the plan specified location. Miscellaneous reinforcing may be tied directly to the GridForm members but should be kept within the interior of the GridForm. All ties used to connect portions of the GridForm or other FRP reinforcing materials must be of plastic material. Any ties used to tie materials other than FRP members shall be of the same material and coating as the item to be tied. GridForm panels shall be mechanically secured to the steel bridge girders when they are placed to prevent uplift and movement prior to the pouring of the concrete deck. The transverse butt joint between in the bottom pultruded plate of adjacent GridForm panels shall be covered with a 3 inch wide by 1/8 inch thick pultruded plate across the entire width of the bridge to prevent concrete from seeping out during pouring.

H.4. Repair

Damage to any part of the GridForm must be reported to the Engineer. The decision to repair and the type of repair must be approved by the Engineer before commencing.

I. Method of Measurement

The quantity of GridForm reinforcement to be paid for will be measured by the square foot of bridge deck, as measured from out-to-out of parapets and between the paving notches.

APPENDIX B – DESIGN CALCULATIONS

Design of GridForm RC deck based on ACI 440.1R-03: Strength and serviceability under positive bending moment.

Section geometry

$t := 7 \cdot in$ (deck thickness excluding non-structural formwork plate)

$b := 1 \cdot ft$ (width of strip considered)

Flexural reinforcement layout

$N_b := 3$ (number of FRP reinforcement I-bars on bottom layer)

$d_b := 2\sqrt{\dfrac{0.32 \cdot in^2}{\pi}}$ $d_b = 0.638\,in$ (diameter of one equivalent FRP I-bar on tension side)

$d := 6.25 \cdot in$ (distance between extreme compression fiber and centroid of tension reinforcement)

FRP I-bars properties (Material Type GV2)

$E_f := 4.5 \cdot msi$ (min. modulus of elasticity of FRP GV2 I-bars)

$f^*_{fu} := 80 \cdot ksi$ (min. tensile strength of FRP GV2 I-bars)

$A_f := 0.32 \cdot in^2$ (nominal cross sectional area of one FRP GV2 I-bar)

$C_E := 0.7$ (environmental reduction factor)

$f_{fu} := f^*_{fu} \cdot C_E$ $f_{fu} = 56.0\,ksi$ (design tensile strength)

$\varepsilon_{fu} := C_E \cdot \dfrac{f^*_{fu}}{E_f}$ $\varepsilon_{fu} = 0.012$ (design rupture strain)

Concrete properties

$f'_c := 4000 \cdot psi$ (compressive strength)

$\varepsilon_{cu} := 0.003$ (rupture strain)

$E_c := 57 \cdot \sqrt{f'_c \cdot psi^{-1}} \cdot ksi$ $E_c = 3.60\,msi$ (modulus of elasticity)

$f_r := 7.5 \cdot \sqrt{f'_c} \cdot \sqrt{psi}$ $f_r = 474.3\,psi$ (modulus of rupture)

Compute strength reduction factor

$A_f := N_b \cdot A_f$ $\qquad$ $A_f = 0.96\,in^2$ $\qquad$ (total area of FRP flexural reinforcement)

$\rho_f := \dfrac{A_f}{b \cdot d}$ $\qquad$ $\rho_f = 0.013$ $\qquad$ (FRP reinforcement ratio)

$\beta_1 := 0.85 - \dfrac{f'_c - 4000 \cdot psi}{1000 \cdot psi} \cdot 0.05$ $\qquad$ $\beta_1 = 0.85$ $\qquad$ (concrete strength factor)

$\rho_{fb} := 0.85 \cdot \beta_1 \cdot \dfrac{f'_c}{f_{fu}} \cdot \dfrac{0.003 \cdot E_f}{0.003 \cdot E_f + f_{fu}}$ $\qquad$ $\rho_{fb} = 0.01$ $\qquad$ (FRP balanced reinforcement ratio)

$$\phi_f := \begin{cases} 0.50 & \text{if } \rho_f \le \rho_{fb} \\ \dfrac{\rho_f}{2 \cdot \rho_{fb}} & \text{if } \rho_{fb} < \rho_f < 1.4 \cdot \rho_{fb} \\ 0.70 & \text{otherwise} \end{cases}$$

$\phi_f = 0.638$ $\qquad$ (strength reduction factor)

Compute design flexural capacity

$$f_f := \sqrt{\frac{(E_f\,\varepsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_f} \cdot E_f\,\varepsilon_{cu}} - 0.5 \cdot E_f\,\varepsilon_{cu}$$

$f_f = 48.87\,ksi$

(stress level in FRP)

$$M_n := \begin{cases} 0.8 A_f\, f_{fu} \cdot \left[d - \dfrac{\beta_1 \cdot \left(\dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) \cdot d}{2} \right] & \text{if } \rho_f < \rho_{fb} \\ \rho_f \min(f_f, f_{fu}) \cdot \left(1 - 0.59 \cdot \dfrac{\rho_f \min(f_f, f_{fu})}{f'_c} \right) \cdot b \cdot d^2 & \text{if } \rho_f \ge \rho_{fb} \end{cases}$$

$Failure\ Mode := if\left(\rho_f \le \rho_{fb}, \text{"FRP RUPTURE"}, \text{"CONCRETE CRUSHING"}\right)$

$Failure\ Mode = \text{"CONCRETE CRUSHING"}$

$\dfrac{\phi_f M_n}{b} = 14.16\,kip \cdot \dfrac{ft}{ft}$ $\qquad$ **(design positive moment capacity)**

Check crack width

$s := 4 \cdot in$		(I-bar spacing)
$M_D := (0.16 + 0.14) \cdot kip \cdot ft$	$M_D = 0.30\,kip \cdot ft$	(moment due to dead load of deck and railing)
$I := 0.3$		(impact factor for live load)
$M_{LL+I} := (1 + I) \cdot 3.07 \cdot kip \cdot ft$	$M_{LL+I} = 3.99\,kip \cdot ft$	(moment due to live load including impact factor)
$n := \dfrac{E_f}{E_c}$	$n = 1.25$	(modular ratio between GFRP reinforcement and concrete)
$k := \sqrt{2 \cdot \rho_f n + (\rho_f n)^2} - \rho_f n$	$k = 0.163$	(ratio of depth of neutral axis to reinforcement depth)
$f_f := \dfrac{M_D + M_{LL+I}}{A_f d \cdot \left(1 - \dfrac{k}{3}\right)}$	$f_f = 9.077\,ksi$	(stress level in GFRP I-bars under service load)
$\beta := \dfrac{t - k \cdot d}{d - k \cdot d}$	$\beta = 1.143$	(ratio of distances to neutral axis from extreme tension fiber and from centroid of reinforcement)
$w := 2 \cdot \dfrac{f_f}{E_f} \cdot \beta \cdot s$	$w = 0.018\,in$	**(maximum crack width < 0.028 in. limit)**

Check short term deflection

$S := 6 \cdot ft - 4 \cdot in$	$S = 68.0\,in$	(effective span length between girders. 4 *in* = width of half top flange)
$M_{cr} := \dfrac{2 \cdot f_r \cdot \left(\dfrac{b \cdot t^3}{12}\right)}{t}$	$M_{cr} = 3.87\,kip \cdot ft$	(cracking moment < $M_D + M_{LL,I}$)
$I_{cr} := \dfrac{b \cdot d^3}{3} \cdot k^3 + n \cdot A_f d^2 \cdot (1 - k)^2$	$I_{cr} = 37.0\,in^4$	(moment of inertia of transformed cracked section)
$I_g := \dfrac{b \cdot t^3}{12}$	$I_g = 343.0\,in^4$	(gross moment of inertia)

$\beta_d := \frac{1}{5} \cdot \frac{\rho_f}{\rho_{fb}}$ $\beta_d = 0.255$ (reduction coefficient for deflection)

$$I_{e,D+LL,I} := \left(\frac{M_{cr}}{M_D + M_{LL+I}}\right)^3 \cdot \beta_d \cdot I_g + \left[1 - \left(\frac{M_{cr}}{M_D + M_{LL+I}}\right)^3\right] \cdot I_{cr}$$

$I_{e,D+LL,I} = 74.2\,in^4$ (effective moment of inertia)

$\Delta i_D := \frac{M_D \cdot S^2}{16 \cdot E_c \cdot I_{e,D+LL,I}}$ $\Delta i_D = 0.004\,in$ (short term deflection due to dead load, assuming effective span between fixed supports)

$\Delta i_{LL+I} := \frac{M_{LL+I} \cdot S^2}{24 \cdot E_c \cdot I_{e,D+LL,I}}$ $\Delta i_{LL+I} = 0.034\,in$ (short term deflection due to live load and impact, assuming point load at mid effective span between fixed supports)

$\Delta i_D + \Delta i_{LL+I} = 0.038\,in$ **(short term service deflection due to dead load and live load including impact)**

$\frac{S}{\Delta i_{LL+I}} = 1972.1$ **(*S* / short term live load and impact deflection ratio > 300 minimum per AASHTO LRFD, 1998)**

Compute maximum long term deflection

$\xi := 2.0$ (time dependent factor for sustained load)

$\lambda := 0.6 \cdot \xi$ (multiplier for additional long term deflection)

$$\Delta i_{LT} := \Delta i_{LL+I} + \lambda \cdot \left(\Delta i_D + 0.2 \cdot \Delta i_{LL+I}\right)$$

$\Delta i_{LT} = 0.047\,in$ **(long term service deflection. 20% of live load including impact factor considered permanent)**

$\frac{S}{\Delta i_{LT}} = 1433.9$ **(*S* / long term deflection ratio)**

Design of GridForm RC deck based on ACI 440.1R-03 and AASHTO Standard Specifications 2002: Strength and serviceability under negative bending moment.

Section geometry

$t := 7 \cdot in$ (deck thickness excluding non-structural formwork plate)

$b := 1 \cdot ft$ (width of strip considered)

Flexural reinforcement layout

$N_b := 3$ (number of FRP reinforcement I-bars on bottom layer)

$d_b := 2\sqrt{\dfrac{0.32 \cdot in^2}{\pi}}$ $d_b = 0.638\,in$ (diameter of one equivalent FRP I-bar on tension side)

$d := 4.75 \cdot in$ (distance between extreme compression fiber and centroid of tension reinforcement)

FRP I-bars properties (Material Type GV2)

$E_f := 4.5 \cdot msi$ (min. modulus of elasticity of FRP GV2 I-bars)

$f^*_{fu} := 80 \cdot ksi$ (min. tensile strength of FRP GV2 I-bars)

$A_f := 0.32 \cdot in^2$ (nominal cross sectional area of one FRP GV2 I-bar)

$C_E := 0.7$ (environmental reduction factor)

$f_{fu} := f^*_{fu} \cdot C_E$ $f_{fu} = 56.0\,ksi$ (design tensile strength)

$\varepsilon_{fu} := C_E \cdot \dfrac{f^*_{fu}}{E_f}$ $\varepsilon_{fu} = 0.012$ (design rupture strain)

Concrete properties

$f'_c := 4000 \cdot psi$ (compressive strength)

$\varepsilon_{cu} := 0.003$ (rupture strain)

$E_c := 57 \cdot \sqrt{f'_c \cdot psi^{-1}} \cdot ksi$ $E_c = 3.60\,msi$ (modulus of elasticity)

$f_r := 7.5 \cdot \sqrt{f'_c} \cdot \sqrt{psi}$ $f_r = 474.3\,psi$ (modulus of rupture)

Compute strength reduction factor

$A_f := N_b \cdot A_f$ $\quad A_f = 0.96\,in^2$ (total area of FRP flexural reinforcement)

$\rho_f := \dfrac{A_f}{b \cdot d}$ $\quad \rho_f = 0.017$ (FRP reinforcement ratio)

$\beta_1 := 0.85 - \dfrac{f'_c - 4000 \cdot psi}{1000 \cdot psi} \cdot 0.05$ $\quad \beta_1 = 0.85$ (concrete strength factor)

$\rho_{fb} := 0.85 \cdot \beta_1 \cdot \dfrac{f'_c}{f_{fu}} \cdot \dfrac{0.003 \cdot E_f}{0.003 \cdot E_f + f_{fu}}$ $\quad \rho_{fb} = 0.01$ (FRP balanced reinforcement ratio)

$$\phi_f := \begin{cases} 0.50 & \text{if } \rho_f \le \rho_{fb} \\ \dfrac{\rho_f}{2 \cdot \rho_{fb}} & \text{if } \rho_{fb} < \rho_f < 1.4 \cdot \rho_{fb} \\ 0.70 & \text{otherwise} \end{cases}$$

$\phi_f = 0.7$ (strength reduction factor)

Compute design flexural capacity

$$f_f := \sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_f} \cdot E_f \varepsilon_{cu}} - 0.5 \cdot E_f \varepsilon_{cu}$$

$f_f = 41.851\,ksi$

(stress level in FRP)

$$M_n := \begin{cases} 0.8 A_f f_{fu} \cdot \left[d - \dfrac{\beta_1 \cdot \left(\dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) \cdot d}{2} \right] & \text{if } \rho_f < \rho_{fb} \\ \rho_f \min(f_f, f_{fu}) \cdot \left(1 - 0.59 \cdot \dfrac{\rho_f \min(f_f, f_{fu})}{f'_c} \right) \cdot b \cdot d^2 & \text{if } \rho_f \ge \rho_{fb} \end{cases}$$

Failure Mode := if $(\rho_f \le \rho_{fb}$, "FRP RUPTURE" , "CONCRETE CRUSHING")

Failure Mode = "CONCRETE CRUSHING"

$\dfrac{\phi_f M_n}{b} = 9.98\,kip \cdot \dfrac{ft}{ft}$ **(design positive moment capacity)**

Check crack width

$s := 4 \cdot in$		(I-bar spacing)
$M_D := 0.24 \cdot kip \cdot ft$	$M_D = 0.24\, kip \cdot ft$	(moment due to dead load of deck on interior girders)
$I := 0.3$		(impact factor for live load)
$M_{LL+I} := (1 + I) \cdot 3.07 \cdot kip \cdot ft$	$M_{LL+I} = 3.99\, kip \cdot ft$	(moment due to live load including impact factor)
$n := \frac{E_f}{E_c}$	$n = 1.25$	(modular ratio between GFRP reinforcement and concrete)
$k := \sqrt{2 \cdot \rho_f n + (\rho_f n)^2} - \rho_f n$	$k = 0.185$	(ratio of depth of neutral axis to reinforcement depth)
$f_f := \frac{M_D + M_{LL+I}}{A_f d \cdot \left(1 - \frac{k}{3}\right)}$	$f_f = 11.866\, ksi$	(stress level in GFRP I-bars under service load)
$\beta := \frac{t - k \cdot d}{d - k \cdot d}$	$\beta = 1.581$	(ratio of distances to neutral axis from extreme tension fiber and from centroid of reinforcement)
$w := 2 \cdot \frac{f_f}{E_f} \cdot \beta \cdot s$	$w = 0.033\, in$	**(maximum crack width > 0.028 in. limit)**

Design of NCHRP 350 Crash Test Level 2 open-post RC railing internally reinforced with GFRP bars: transverse strength of post-to-deck connection based on ACI 440.1R-03 and AASHTO Standard Specifications 2002.

Open-post rail geometry

$P_{int} := 4 \cdot ft$ (length of intermediate post)

$G := 4 \cdot ft$ (length of opening)

$H := 30 \cdot in$ (total height from deck surface/roadway pavement)

$H_e := 20 \cdot in$ (minimum vertical distance between applied TL-3 impact force and roadway per LRFD)

Section geometry

(h/b = height/width of rectangular cross section; d = deck, p = post, b = beam)

$h_d := 7 \cdot in$ $\quad h_p := 9.5 \cdot in$ $\quad h_b := 12 \cdot in$

$b_d := 48 \cdot in$ $\quad b_p := 48 \cdot in$ $\quad b_b := 17 \cdot in$

FRP flexural reinforcement layout

N_b = number of FRP reinforcement bars on tension side

d_b = diameter of one FRP bar

d = distance between extreme compression fiber and centroid of tension reinforcement

$N_{b,d} := 16$ $\quad N_{b,p} := 10$ $\quad N_{b,b} := 3$

$d_{b,d} := 0.625 \cdot in$ $\quad d_{b,p} := 0.625 \cdot in$ $\quad d_{b,b} := 0.625 \cdot in$

$d_d := 5.22 \cdot in$ $\quad d_p := 7.5625 \cdot in$ $\quad d_b := 10.1875 \cdot in$

FRP rebars properties

(http://www.hughesbros.com/Aslan100/Aslan100_GFRP_rebar.html)

E_f = guaranteed modulus of elasticity

C_E = environmental reduction factor

f^*_{fu} = guaranteed tensile strength

f_{fu} = design tensile strength

ε_{fu} = ultimate tensile strain

$E_{f,d} := 5.92 \cdot msi$ $\quad E_{f,p} := 5.92 \cdot msi$ $\quad E_{f,b} := 5.92 \cdot msi$

$C_{E,d} := 0.7$ $\quad C_{E,p} := C_{E,d}$ $\quad C_{E,b} := C_{E,d}$

$f^*_{fu,d} = 95.00\ ksi$ $\quad f^*_{fu,d} = 95.00\ ksi$ $\quad f^*_{fu,d} = 95.00\ ksi$

$f_{fu,d} = 66.50\ ksi$ $\quad f_{fu,d} = 66.50\ ksi$ $\quad f_{fu,d} = 66.50\ ksi$

$\varepsilon_{fu,d} = 0.01$ $\quad \varepsilon_{fu,d} = 0.01$ $\quad \varepsilon_{fu,d} = 0.01$

Concrete properties

$f'_{c,d} := 4000 \cdot psi$ $f'_{c,p} := 4000 \cdot psi$ $f'_{c,b} := 4000 \cdot psi$

$E_{c,d} = 3.60\, msi$ $E_{c,p} = 3.60\, msi$ $E_{c,b} = 3.60\, msi$

$\varepsilon_{cu,d} := 0.003$ $\varepsilon_{cu,p} := 0.003$ $\varepsilon_{cu,b} := 0.003$

Compute strength reduction factor

$A_{f,b}$ = nominal cross sectional area of one FRP bar
ρ_f = FRP reinforcement ratio
ρ_{fb} = FRP balanced reinforcement ratio
ϕ_f = strength reduction factor

$\frac{\rho_{f,d}}{\rho_{fb,d}} = 2.35$ $\frac{\rho_{f,p}}{\rho_{fb,p}} = 1.01$ $\frac{\rho_{f,b}}{\rho_{fb,b}} = 0.64$ **(reinforcement ratios)**

$\phi_{f,d} = 0.70$ $\phi_{f,p} = 0.51$ $\phi_{f,b} = 0.50$ **(strength reduction factors)**

Compute design flexural capacity

f_f = nominal cross sectional area of one FRP bar
M_n = nominal flexural capacity
FM = failure mode

$M_{n,d} = 83.25\, kip \cdot ft$ $M_{n,p} = 127.54\, kip \cdot ft$ $M_{n,b} = 41.60\, kip \cdot ft$

$\phi_{f,d} \cdot M_{n,d} = 58.27\, kip \cdot ft$ $\phi_{f,p} \cdot M_{n,p} = 64.67\, kip \cdot ft$ $\phi_{f,b} \cdot M_{n,b} = 20.8\, kip \cdot ft$

FM_d = "CONCRETE CRUSHING" FM_p = "CONCRETE CRUSHING" FM_b = "FRP RUPTURE"

Assessment of structural strength
(consistent with AASHTO LRFD 1998 and 2004 provisions - Section 13)

$\left(M_{cr,d} \;\; M_{cr,p} \;\; M_{cr,b}\right) = (15.50 \;\; 28.54 \;\; 16.13)\, kip\cdot ft$ **(cracking moments)**

Post/deck connection strength: design assuming on diagonal tension failure at corner

$G = 4.00\, ft$ (length of railing opening for TL-3)

$l_{dc} := 7\cdot in$ (length of theoretical diagonal crack at corner)

$\alpha := 34\cdot deg$ (angle between theoretical diagonal crack plane and horizontal - slab plane)

$f'_c := 4000\; psi$ (concrete compression strength in slab)

Define force components for equilibrium of corner free-body diagram (Nilsson & Losberg 1976)

$V_p := 11.88\cdot kip$ **(trial value for load on post - Note: change until convergence)**

$T(f'_c) := \frac{2}{3}\cdot\left(7.5\cdot\sqrt{f'_c}\cdot\sqrt{psi}\right)\cdot P_{int}\cdot l_{dc}$ (tension perpendicular to crack plane assuming parabolic stress distribution)

$F_{f,d}(f'_c) := T(f'_c)\cdot sin(\alpha) + 0.5\cdot V_p$ (tension in FRP bars in top mat of slab)

$F_{f,p}(f'_c) := T(f'_c)\cdot cos(\alpha)$ (tension in FRP bars in post)

Define axial stress in FRP bars in top mat of slab (i.e., weaker section of in post/deck connection) at $M = M_n$

$$f_{f,d}(f'_c) := min\left[f_{fu,d},\ \sqrt{\frac{\left(E_{f,d}\cdot\varepsilon_{cu,d}\right)^2}{4} + \frac{0.85\cdot\beta_{1,d}\cdot f'_c}{\rho_{f,d}}\cdot E_{f,d}\cdot\varepsilon_{cu,d}} - 0.5\cdot E_{f,d}\cdot\varepsilon_{cu,d}\right]$$

Compute efficiency ratio and nominal/design strength of connection

$$\eta := \frac{F_{f,d}(f'_c)}{A_{f,d}}\cdot\frac{1}{f_{f,d}(f'_c)}$$

$\eta = 0.297$

$$F_{n,p\;AASHTO\;2002} := \eta\cdot\frac{M_{n,d}}{21.5\cdot in + \frac{h_d}{2}}$$

$F_{n,p\;AASHTO\;2002} = 11.88\, kip$

(Check convergence of nominal strength of post/deck connection assuming cracked deck, with respect to V_p. Force is assumed applied at mid-height of railing beam)

$\phi_{DT} := 0.85$ **(strength reduction factor for diagonal tension failure)**

$\phi_{DT}\cdot F_{n,p\;AASHTO\;2002} = 10.10\, kip$ **(design strength of post > 10 kip mandated by AASHTO Standard Specifications 2002)**