Design of Concrete Railing Reinforced with Glass Fiber Reinforced Polymer Bars

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ABSTRACT

The use of Fiber Reinforced Polymer (FRP) reinforcement is a practical alternative to conventional steel rebars in concrete structures subjected to aggressive environments. The solution is attractive for bridge deck and rail applications, as it eliminates corrosion of the steel reinforcement, which is the major instrument of degradation. Due to the peculiar physical and mechanical characteristics of advanced composite materials, the design philosophy of FRP reinforced concrete (RC) structures differs from that of traditional RC. This paper introduces a systematic approach adopted for the structural and functional design of an open-post bridge railing reinforced with Glass FRP bars (GFRP) as compared to steel RC counterparts, according to the AASHTO LRFD Bridge Design Specifications. Design examples, accounting for different gap opening length and rail beam reinforcement, and based on the experimental static response of full-scale post/deck connections, are finally presented and discussed.

INTRODUCTION

Corrosion of the steel reinforcement is a major cause of degradation of concrete decks in a large portion of the bridge inventory worldwide. Effects accrue from the routine use of deicing salts on roads and exposure to harsh environments, leading to reduced strength and functionality, and to safety concerns. The use of GFRP reinforcement ideally eliminates the issue and represents a practical alternative to conventional steel for non-prestressed structures [1]. A number of field implementations, typically as parts of research projects, have demonstrated the validity of the technology. Design principles are fairly well established [2] and guideline documents have been published in North America, Europe, and Japan. In the US, the 2005 “Guide for the Design and Construction of Concrete Reinforced with FRP Bars” by the American Concrete Institute (ACI) [3] will shortly supersede the 2003 document (ACI 440.1R-03).

A new version of the open post Federal Lands Modified Kansas Corral Bridge Rail (MKCR) [4] reinforced with GFRP rebars was designed to develop a truly steel-free deck and rail system, as recently showcased in the accelerated construction of a bridge deck in Greene County, MO, using innovative prefabricated GFRP stay-in-place (SIP)
reinforcing panels [5]. Previous research demonstrated the structural adequacy of highway GFRP RC barriers under pendulum impact load, where the original steel rebars were replaced on a strength equivalence basis [6]. The crashworthiness of a GFRP RC TxDOT T203 open-post rail was also assessed via crash test as per National Cooperative Highway Research Program (NCHRP) Report 350 Test Level 3 (TL-3) criteria [7], i.e. with a 4500 lb pickup truck impacting at a speed of 60 mph and crash angle of 25° [8], as typically required on the National Highway System [9]. The objective of the design presented herein is twofold. First, ensure compliance with the Test Level 2 strength criteria (TL-2, same as TL-3 with speed reduced to 45 mph), i.e. the category of the steel RC MCKR replaced [4], while providing additional redundancy to evaluate upgrade to the TL-3 category. Second, devise a simple prefabricated reinforcement geometrically compatible with the GFRP deck panels. Rapid pre-assembling of lightweight rail post and beam rebar cages significantly improves productivity, while the ease of installation minimizes time-consuming and labor-intensive field operations, thereby conferring actual economical appeal to the solution, besides its safety and durability characteristics.

**RAILING DESIGN**

Bridge railings must contain and redirect errant vehicles while preventing rollover and snagging, and allowing deceleration to a stop at a relatively short distance from the impact section. Therefore, crash testing of bridge safety appurtenances aims at assessing both the structural and geometrical crashworthy, depending on the level of service sought (TL-1 to TL-6, being the latter the most demanding), along with the vehicle occupant risk. Based on the results of a number of full-scale crash tests performed as part of programs under the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), NCHRP and individual states, the AASHTO LRFD Bridge Design Specifications, Section 13 [10], set forth strength and geometry criteria for preliminary design.

**Geometry**

The safety performance of an open-post concrete railing greatly depends on its geometry. Critical requirements are:

- sufficient rail height, $H$, and suitable profile shape to reduce the potential for vehicle rollover. A minimum value $H = 27''$ is recommended for both TL-2 and TL-3;
- continuous solid rail beam with smooth and sufficient contact width, $A$, with respect to $H$, to reduce the potential for vehicle wheel, bumper or hood impact with the post. A minimum $A/H$ ratio of 0.25 is recommended, along with specified graphical parametric criteria;
- sufficient post setback distance, $S$, with respect to combination of $A$ and $H$, to reduce the potential for vehicle snagging. Parametric recommendations are provided in graphical fashion to select design alternatives that proved to perform satisfactorily.

Figure 1(a) and 1(b) show the geometry of the GFRP RC MKCR designed. Compared to the profile of the original design (dashed line), $A$ has been increased from 14" to 17", with $H$ increased from 27" to 30". Although vertical barriers typically offer the greatest reduction in rollover potential, despite the tradeoff of increased lateral accelerations [11],
the recommended minimum height may be inadequate, especially in case of higher service levels. This has been recently observed in the (failed) TL-3 crash test of a 27" GFRP RC railing [7, Appendix A], whereas a similar configuration with increased height performed well [7, Appendix B]. The post setback was kept at the original distance $S = 2"$ from the rail beam contact surface, similarly to other steel RC counterparts of same or higher category, such as the Modified Corral Rail (TL-2) and 32" Corral Rail (TL-4) in Kansas, or the Concrete Beam and Post (TL-2) and Open Concrete Bridge Rail (TL-4) in Nebraska [4]. Figure 1(c) and 1(d) show the compliance of the selected design with the LRFD recommendations to minimize the risk of impact on the rail post and vehicle snagging, also correcting the slightly low $A/H$ ratio of 0.52 of the original profile. It is seen from the grey arrows that the addition of any wearing surface would further move the geometric parameters into the preferred safety areas.

![Figure 1](image)

**FIGURE 1**
*Geometry of GFRP RC Federal Lands MKCR with Post and Gap Length of 4' (a, b) and Compliance with AASHTO LRFD Bridge Design Specifications (Blank Circle) [10] (c, d)*

The post and gap opening length, $P$ and $G$, have been changed from the original 3' and 7', respectively, to 4' each (Figure 1(b)), in order to provide additional redundancy needed to evaluate upgrade to TL-3, as well as geometrical compatibility with the 8' long modular GFRP SIP reinforcing panels.

**Load Demand**

The dynamic loads imparted by an impacting vehicle under specified crash test conditions [8] are translated by AASHTO into equivalent factored static loads (transverse, $F_t$, longitudinal, $F_l$, and vertical, $F_v$) to be used for structural design, as summarized in Table 1 [10]. In case of concrete railings designed to resist $F_t$, the effects of $F_l$ and $F_v$ are generally not of concern. For TL-2 and TL-3 crash Test Level, a transverse strength of 27 kip and 54 kip applied on the rail beam at a height $H_c = 20"$ and 24" from the roadway, respectively, is required. $F_t$ should be uniformly distributed along $L_t = 4'$, which is the typical length of significant contact observed experimentally.
TABLE 1
EQUIVALENT STATIC DESIGN FORCES FOR TL-2 AND TL-3 TRAFFIC RAILINGS [10]

<table>
<thead>
<tr>
<th>Railing Test Level</th>
<th>TL-2</th>
<th>TL-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Force, $F_t$</td>
<td>27 kip (uniformly distributed along $L_t = 4'$)</td>
<td>54 kip (uniformly distributed along $L_t = 4'$)</td>
</tr>
<tr>
<td>Longitudinal Force, $F_l$</td>
<td>9 kip (uniformly distributed along $L_l = 4'$)</td>
<td>18 kip (uniformly distributed along $L_l = 4'$)</td>
</tr>
<tr>
<td>Vertical Force, $F_v$</td>
<td>4.5 kip (uniformly distributed along $L_v = 4'$)</td>
<td>4.5 kip (uniformly distributed along $L_v = 4'$)</td>
</tr>
<tr>
<td>Height of $F_t$ and $F_l$ application, $H_e$</td>
<td>20&quot; (min)</td>
<td>24&quot; (min)</td>
</tr>
</tbody>
</table>

Relatively older traffic railings, such as the Federal Lands MKCR, have been proof tested according to the AASHTO Standard Specifications, Section 2.7.1.3 [12], which prescribe a minimum transverse force of 10 kip, uniformly distributed along a length of 5', be applied on single posts at the center of gravity of the rail beam.

Resistance Function

The nominal resistance to transverse load of steel RC railings is typically determined via yield line analysis [10, 13]. Upon postulation of a failure mode in the form of a kinematically admissible collapse mechanism that satisfies the yield criterion at the yield lines, an upper-bound ultimate load is determined by equating the work done by the external load and the resisting forces. Hence, in case of statically indeterminate systems, redistribution of the bending moments must be assumed with plastic rotations. Possible failure modes considered in the AASHTO LRFD specifications, when a single rail beam span is strong enough to resist $F_t$ and transfer it to the connected posts (“strong beam-strong post” system), involve either a single post and the two adjacent beam spans (1-post) or two posts and adjacent beam spans (2-post), as illustrated in Figure 2. The latter is typically applicable to open post RC railings [13].

![FIGURE 2](image-url)

**FIGURE 2**
SCHEMATIC OF 1-POST FAILURE MODE (a) WITH POST FREE BODY DIAGRAM (b), AND 2-POST FAILURE MODE (c)

Due to the linear elastic behavior of FRP materials, moment redistribution cannot be accounted for in RC design, and for each failure mode assumed both equilibrium and deformation compatibility must be verified. The methodology proposed herein to assess the strength level of a GFRP RC railing is consistent with the bases of the AASHTO
LRFD approach, while the ACI 440 provisions are followed for the flexural and shear design and analysis of the FRP reinforced members and the rail structure.

Failure is assumed to occur either at the post/deck connections or at the rail beam ends, whose rotation is constrained (rotational stiffness $k_\phi \approx \infty$ along with translational constraints along $z$ at the connection with the exterior posts), when the post attains a compatible displacement $\delta = \delta^*$ at $H_e$ (Figure 2(a) and 2(c)). The height $H_e$ is assumed for simplicity, since it lies close to the center of gravity of the beam and small displacements are accounted for. In fact, relatively large deformations are not observed in successful crash tests and are incompatible with optimal functionality characteristics, and thus should be prevented. Hence, torsional effects on the beam can be neglected. The transverse force resisted by the railing, assuming a 1-post or 2-post failure mode ($n = 1$ and $n = 2$, respectively, where $n$ is the number of posts considered), is computed either via equilibrium method or imposing conservation of energy, i.e.

\[ F_p(\delta^*) = F_p^* = \frac{4M_b^*}{G} + nF_p^* = 2V_p^* + nF_p^*, \]  

as illustrated in Figure 2(b) for $n = 1$, wherein the bending moments and shear forces acting at the beam ends in the gap opening are expressed as

\[ M^*_b = \frac{6E_cI_{e,b}(M^*_b)}{G^2} \delta^* \text{ and } V^*_b = \frac{12E_cI_{e,b}(M^*_b)}{G^3} \delta^*, \]  

respectively, and the combined tension force and bending moment at the connection are given as

\[ F_p^* \text{ and } M_p^* = F_p^*H_e, \]

respectively.

In computing the nonlinear beam displacement as a function of the bending moment or shear from (2) or (3), when the applied moment exceeds the cracking limit, $M_{cr,b}$, and the section moment of inertia drops below its gross value, $I_{g,b}$, the flexural stiffness is determined by multiplying the concrete elastic modulus, $E_c$, by the effective moment of inertia

\[ I_{e,b}(M^*_b) = \left( \frac{M_{cr,b}}{M_b^*} \right)^3 \beta_{d,b}I_{g,b} + \left[ 1 - \left( \frac{M_{cr,b}}{M_b^*} \right)^3 \right] I_{cr,b} \leq I_{g,b}. \]  

(6) is the well known Branson’s equation, which has been modified to account for reduced tension stiffening in FRP RC members by means of the factor

\[ \beta_{d,b} = \frac{1}{5} \left( \frac{\rho_{f,b}}{\rho_{fb,b}} \right), \]  

wherein $\rho_{f,b}$ and $\rho_{fb,b}$ are the FRP reinforcement ratio and balanced reinforcement ratio, respectively.

The ultimate transverse load in (1) is attained when the post/deck connection reaches its strength, or when the beam moment reaches its nominal value, $M_{n,b}$, assumed equal for both positive and negative bending (symmetric reinforcement). It should be noted that connections of vertical posts often exhibit only a fraction of the theoretical resistance of either the adjoined post or deck sections. Failure patterns may thus develop within the bridge deck at load levels considerably smaller than that otherwise expected. Typical factors that affect the behavior of connection details (not necessarily concurrently) are:
• effectiveness of the post/deck construction joint;
• effectiveness of the anchorage of the post tension bent bars within the deck;
• developable tensile stress in straight or bent bars in the deck top mat;
• contribution of adjacent deck portions.

The resistance function (1) can be rearranged to yield the strength demand function

\[ F_{p,\text{min}}(\delta) = \frac{F_i - 2V_i \delta}{n}, \]

wherein \( F_i = 27 \text{ kip} \) and \( 54 \text{ kip} \) for TL-2 and TL-3, respectively. For each compatible displacement \( \delta \) and given rail beam design, (8) defines the minimum load applied at a height \( H_e \) to be resisted at the connection, without contribution of the beam elements, in order to meet the equivalent static strength of the selected crash Test Level. Therefore, the resistance function \( F_{p,\delta} \) of a candidate connection can be determined experimentally, and evaluated using (8) for a given beam resistance function \( M_{b,\delta} \) and rail geometry. Similar connections may be considered for railings with different service levels by modifying the beam design (geometry, reinforcement) and/or the gap opening length. A similar approach was used to modify the post and beam design at the open joints.

**DESIGN EXAMPLES**

Two full-scale post/deck overhang subassemblies were tested under static load to determine their force-displacement response (Figure 3), in order to assess compliance of the new design with strength requirements. A preliminary linear elastic finite element analysis was performed to select a setup representative of the actual boundary conditions. Experiments were conducted as part of a research program aimed at developing and implementing a steel-free deck and railing system for accelerated bridge construction, thereby complementing an innovative GFRP deck reinforcement made of large-size lightweight SIP grating panels [5]. The configurations shown in Figure 3(a) and 3(b), herein referred to as M1 and M2, respectively, were tested to evaluate the influence of reinforcement layout and construction details devised to improve constructibility.

**FIGURE 3**

*THRU-POST GFRP REINFORCEMENT IN SPECIMEN M1 (a) AND M2 (b), AND STATIC TEST OF POST/DECK OVERHANG SUBASSEMBLY (c)*
Table 2 summarizes the nominal moment capacity of the GFRP RC structural sections (beam, post and deck), computed according to the ACI 440 provisions, and the actual ultimate capacity of the connections tested. All the sections were over-reinforced, since a concrete crushing failure mode is preferred to more brittle FRP rupture [3].

Theoretical beam response was used to define the strength demand function $F_{p,min\cdot\delta}$ (8) for the post/deck cantilever subassembly under different scenarios, as illustrated in Figure 4, where (8) has been scaled to account for horizontal deflection measured at $H = 29-1/2"$ instead of the $H_e$ value for the crash Test Level considered.

<table>
<thead>
<tr>
<th>Connection specimen ID</th>
<th>M1</th>
<th>M2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal moment beam, $M_{n,b}$</td>
<td>58.8 kip-ft ($f'_c = 4$ ksi, theoretical)</td>
<td>58.8 kip-ft ($f'_c = 4$ ksi, theoretical)</td>
</tr>
<tr>
<td>Nominal moment post, $M_{n,p}$</td>
<td>158.2 kip-ft ($f'_c = 5.8$ ksi, experimental)</td>
<td>173.8 kip-ft ($f'_c = 8.4$ ksi, experimental)</td>
</tr>
<tr>
<td>Nominal moment deck, $M_{n,d}$</td>
<td>66.5 kip-ft ($f'_c = 7.8$ ksi, experimental)</td>
<td>94.4 kip-ft ($f'_c = 5.1$ ksi, experimental)</td>
</tr>
<tr>
<td>Nominal moment deck, $M_{n,d}$ (contribution of sections adjacent to post is neglected)</td>
<td>26.7 kip-ft (13.3 kip) (experimental)</td>
<td>24.5 kip-ft (12.2 kip) (experimental)</td>
</tr>
<tr>
<td>$M_{n,d}$ / $M_{n,p}$</td>
<td>0.40</td>
<td>0.26</td>
</tr>
</tbody>
</table>

TABLE 2
Nominal Moment Capacity of GFRP RC Post, Deck Section at Connection ($C_E = 1$), and Ultimate Strength of Connection M1 and M2

FIGURE 4
Resistance Function of GFRP RC Rail Beam (Theoretical) and Connections M1 and M2 (Experimental) for $G = 4'$ (a) and $G = 8'$ (b), and Connection Strength Demand Functions (8)

In Figure 4(a), the static response of connections M1 and M2 are compared with the strength demand function for railing configurations comprising the beam shown in Figure
3(a) and 3(b), i.e. reinforced with three #5 bars in tension, $P = G = 4'$, and assuming a conservative 1-post failure mode for TL-2 (TL-2 1-post) and TL-3 (TL-3 1-post), and a typical 2-post failure mode for TL-3 (TL-3 2-post). Solid and dashed lines indicate beam response and post strength demand assuming the environmental reduction factor for the guaranteed GFRP bar strength (typically 90-110 ksi) as $C_E = 0.7$, as recommended for design purposes, and $C_E = 1$, respectively. Solid circles mark the nominal capacity of the beam section and the strength level of the connections. Blank circles mark the design moment of the beam section. It is seen that the design appears highly redundant for TL-2 even assuming a rather conservative 1-post failure mode. TL-3 demand is not satisfied when $C_E = 0.7$ and the beam design capacity is considered instead of the nominal value, although this may not be representative of the actual crash test conditions. Additional reinforcement and/or increase in the rail beam width may be considered for upgrade. It is also noted that a TL-3 open post GFRP RC railing with similar geometry and reinforcement was recently successfully crash tested [7].

In Figure 4(b), a railing configuration with an additional #5 tension bar in the beam, $P = 4'$ and $G = 8'$, is evaluated for TL-2 demand under both 1-post and 2-post failure mode assumptions. For comparison purposes, the horizontal dashed line indicates the post strength demand according to the AASHTO Standard Specifications [12], where the required 10 kip load applied at the center of gravity of the rail beam has been scaled to the correspondent 8.96 kip applied at $H_e = 2'$. With respect to Figure 4(a), it is seen that design is controlled by the connection instead of the rail beam. The static response of the railing appears adequate for TL-2, especially if the 2-post failure mode is considered, which is consistent with the results of crash tests on other similar TL-2 or higher level steel RC open-post railings commonly used in the US.

CONCLUSIONS

A systematic approach for the design and analysis of open post GFRP RC railings has been presented. The proposed methodology is consistent with the geometrical and structural design bases set forth in the AASHTO LRFD specifications [10], with appropriate modifications to comply with the ACI 440 guidelines for concrete internally reinforced with FRP bars [3].

Resistance functions were defined by imposing force equilibrium and displacement compatibility while evaluating realistic failure scenarios. Design examples for constant rail profile and different beam reinforcement and gap opening length configurations have been presented. Preliminary results, to be corroborated by full-scale crash testing [8], indicate that performance is consistent with that of similar steel and one GFRP RC counterparts crash tested to date.

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