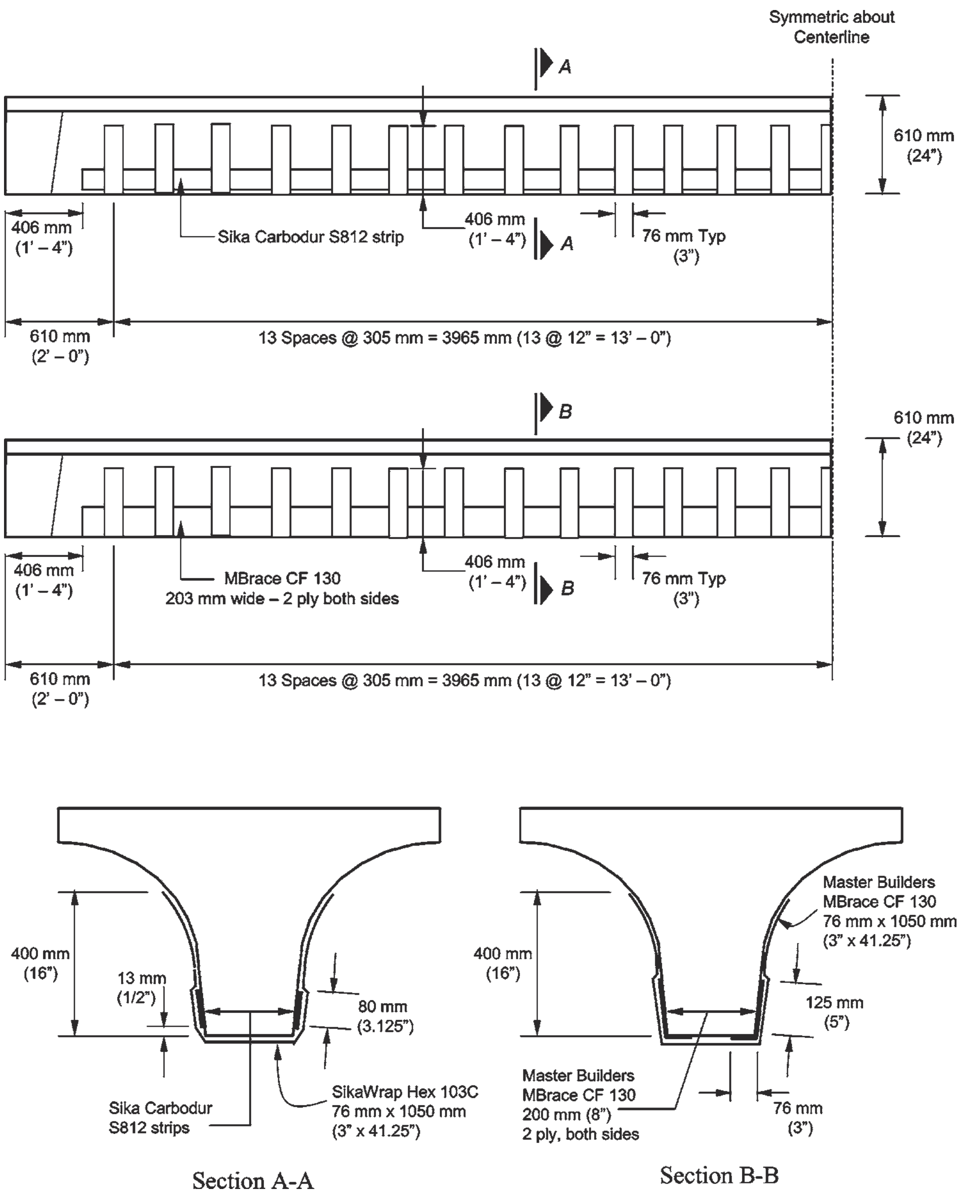

Figure 4—Strengthening Designs

Figure 5—Strengthening of Span with Pultruded Plates

(a) Pultruded Plate System (span A) (b) Wet-Layup System (span B)

Figure 6—Strengthened Pan-girders

(a) Transverse strap debonding (b) Longitudinal laminate debonding

Figure 7—Debonding Sequence of Composites during Testing (specimen J-1)

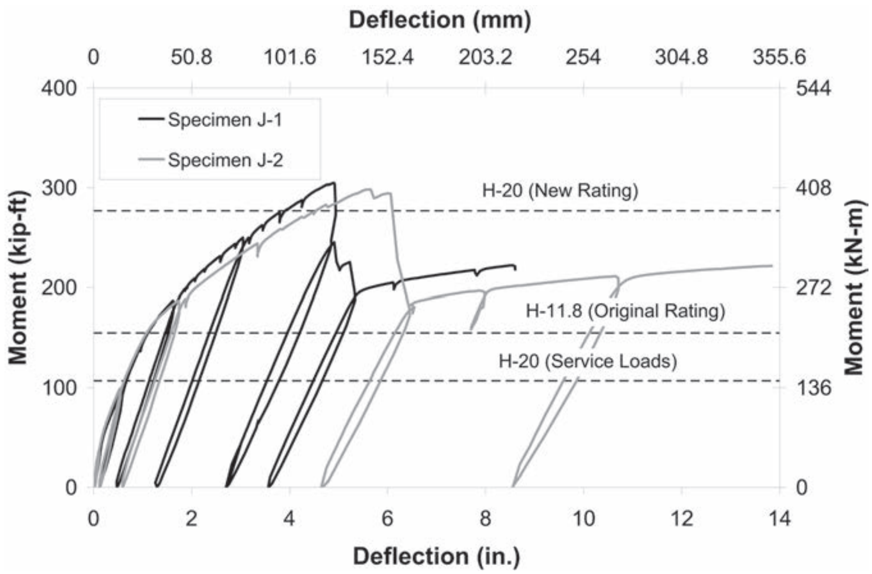

Figure 8—Comparison of Measured Response with Target Strength

Strengthening of a Bridge Using Two FRP Technologies

by P. Casadei, N. Galati, R. Parretti, and A. Nanni

Synopsis: This paper reports on the use of externally bonded fiber reinforced polymers (FRP) laminates and Near Surface Mounted FRP bars for the flexural strengthening of a concrete bridge. The bridge selected for this project is a three-span simply supported reinforced concrete slab with no transverse steel reinforcement, load posted and located on Martin Spring Outer Road in Phelps County, MO. The original construction combined with the presence of very rigid parapets caused the formation of a wide longitudinal crack which resulted in the slab to behave as two separate elements. In order to clarify the behavior of the structure, load tests were performed and a finite element method (FEM) analysis undertaken. The FRP strengthening was designed to avoid further cracking and such that the transverse flexural capacity be higher than the cracking moment. Both FRP techniques were easily implemented and showed satisfactory performance.

Keywords: bridges; carbon fibers; fiber reinforced polymer (FRP); finite element method (FEM); load testing; reinforced concrete; strengthening

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INTRODUCTION

Over 40 percent of the nation's bridges are in need of repair or replacement(1). Budget constraints prohibit many states from repairing or replacing all of these bridges; consequently, states are forced to post load restrictions on their bridges as a temporary solution until more funds become available for repair or replacement. Advanced composite materials made of fiber reinforced polymers (FRP) have a high potential for providing a solution to this problem.

The bridge selected for this project is located on old Route 66, now Martin Springs Outer Road, in Phelps County, Missouri (see Fig. 1). This bridge was commissioned in 1926 and was originally on a gravel road. In 1951, the last miles of US Route 66 through Phelps County were concrete paved. In 1972, Route 66 was replaced by interstate I-44. Commissioning of I-44 led to a significant decrease in traffic along Route 66. Load posting of this bridge (a load restriction posting of S-16 trucks over *13 tons* (*11.79 tons in SI units*) *15 mph* (*24.14 km/hr*), except for single unit trucks H-20 weight limit to *19 tons* (*17.24 tons in SI units*), and all other trucks weight limit *30 tons* (*27.21 tons in SI units*)) was approved around 1985 and had a significant impact on the local economy.

This bridge is a three-span simply supported reinforced concrete slab. The total bridge length is *66 ft* (*20.12 m*) and the total width of the deck is *22.5 ft* (*6.86 m*). Fig. 2 shows a detailed geometry of the bridge. Based on visual and Non Destructive Testing (NDT) evaluation, it was determined that the superstructure is a solid concrete slab *14 in* (*35.56 cm*) thick, running from pier to pier, the longitudinal reinforcement is made of *#8 (□25.4 mm)* bars spaced at *5 in* (*12.7 cm*) on centers, and no transverse reinforcing is present. From cores (cylinders *3 in×6 in, 7.62 cm× 15.24 cm*) , the average compressive strength of the concrete was measured to be *4100 psi* (*28.27 MPa*); the yield of the steel was also tested on one bar sample, and resulted to be *32 ksi* (*220.63 MPa*). The lack of transversal reinforcement and the presence of very rigid parapets caused the slab to crack along all three slabs at mid-span. The cracks are approximately *1 in* (*2.54 cm*) wide, and intersect longitudinal bars (see Fig. 3). There is no significant cracking in any other portion of the slab and only minor corrosion of the bars crossing the crack.

Given the very good concrete condition of the bridge, the structure was an ideal candidate for strengthening using CFRP composites[2][3]. Two different strengthening schemes were adopted in this project for evaluation purposes, bonded carbon FRP laminates installed by externally wet lay-up and near surface mounted (NSM) rectangular FRP bars. The paper describes the use of both strengthening techniques.

BRIDGE ANALYSIS

Load Combinations

For the structural analysis of the bridge the ultimate values of bending moments and shear forces are computed by multiplying their nominal values by the dead and live factors and by the impact factor according to AASHTO[4][2] Specifications as shown in Eq.(1):

$$\omega_u = 1.3\left[\beta_d D + 1.67\left(L+I\right)\right] \tag{1}$$

where D is the dead load, L is the live load, $\beta_d=1.0$ as per AASHTO[4] Table 3.22.1A, and I (maximum 30%) is the live load impact calculated as follows:

$$I = \frac{50}{L+125} = \frac{50}{22+125} = 0.34 \le 30\% \tag{2}$$

and $L=22\ ft$ (*6.70 m*) represents the span length from center to center of supports.

Design Truck and Design Lanes

Prior to the design of the strengthening, the analysis of the bridge was conducted by considering a HS20-44 truck load (which represents the design truck load as per AASHTO[4] Section 3.7.4) having geometrical characteristics and weight properties shown in Fig. 4. According to AASHTO[4] Section 3.6.3 fractional parts of design lanes shall not be used for roadway widths less then *20ft* (*6.09 m*).

As a consequence of these specifications, the loading conditions required to be checked are laid out in Fig. 5.

Fig. 5*a* represents the HS20-44 design truck already described in Fig. 4. Given the specific bridge geometry, the worst loading scenario, causing maximum moment at mid span (see Fig. 6) and shear at the support (see Fig. 7), is obtained for the minimum spacing of *14.0 ft* (*4.27 m*) between the two rear axles.

The design lane loading condition (AASHTO[4] Section 3.6) consists of a load of *640 lbs per linear foot* (*9.35 kN/m*), uniformly distributed in the longitudinal direction with a single concentrated load so placed on the span as to produce maximum stress. The concentrated load and uniform load is considered to be uniformly distributed over a *10'-0"* (*3.05 m*) width on a line normal to the center lane of the lane. The intensity of the concentrated load is represented in Fig. 5*b* for both bending moments and shear forces. This load shall be placed in such positions within the design lane as to produce the maximum stress in the member.

Slab Analysis

The deck is considered to be a one-way slab, disregarding the contribution of the parapets. For simplicity, the deck has been studied considering the overall width of the transversal cross section.

The dead load was computed considering the self-weight of the concrete slab plus the permanent weight of the top layer of asphalt. The weight of parapets has been computed according to the geometrical properties of Fig. 5*c* and, for simplicity, distributed throughout the width of the slab. Table 1 presents a summary of these values.

Computations for the *design lane* and the design truck load have been carried out and it has been found that the *design truck load* is the controlling loading condition.

For the flexural analysis, the critical loading condition corresponds to the case when the truck has one of its rear axles at the mid-span of the member (see Fig. 6). The factored ultimate moment demand is computed for the entire slab in Eq.(3):

$$M_u = \frac{1.3 \times \omega_D L^2}{8} + \frac{1.3 \times 1.67 \times 1.3 \times P_2 L}{4} \tag{3}$$

$$M_u = \frac{1.3(5.99)(22)^2}{8} + \frac{1.3 \times 1.67 \times 1.3 \times (32)(22)}{4} = 493.7k - ft\ \ (669kN - m) \tag{4}$$

For the shear analysis, the critical loading condition is when one rear axle is closer to one support and the other is *14 ft* (*4.27 m*) away over the span (see Fig. 7). The factored ultimate shear demand is computed for the entire slab in Eq.(5):

$$V_u = \frac{1.3 \times \omega_D L}{2} + 1.3 \times 1.67 \times 1.3 \left(P_2 + P_2 - \frac{P_2(l + x) + P_2 x}{L} \right) \tag{5}$$

$$V_u = \frac{1.3(5.99)(22)}{2} + 1.3 \times 1.67 \times 1.3 \left(32 + 32 - \frac{32(15)+32(1)}{22} \right) = 200.6 kip\ (892 kN) \quad (6)$$

The bridge geometry and material properties are reported in Table 2 along with the computed nominal flexural and shear capacities based on conventional RC theory[5]. Since both ϕM_n and ϕV_n are larger than M_u and V_u respectively, no flexural and shear strengthening are required in the longitudinal direction.

The cracking moment of a unit strip has been computed (see Eq.(7)) to design a strengthening scheme able to ensure that $\phi M_{n,transv.}$ is larger or equal than the cracking moment.

$$M_{cr} = \frac{7.5\sqrt{f_c^{'}} I_g}{h/2} = \frac{7.5\sqrt{4100}(2744)}{7} = 15.7\, k - ft/ft\ \ (21 kN - m/m) \quad (7)$$

Where I_g represents the gross moment of inertia of the concrete cross section with *b = 12 in* (*30.48 cm*) and *h = 14 in* (*35.56 cm*).

BRIDGE STRENTHENING

The strengthening design follows the previous considerations and has the purpose of giving the bridge a moment capacity in the transversal direction equal or greater to the cracking moment computed in Eq.(7), in order to avoid further crack openings and deterioration of the concrete due to water percolation through the cracks.

Two different FRP strengthening techniques have been adopted: (1) externally bonded CFRP laminates installed by manual wet lay-up, and (2) Near-surface mounted CFRP rods embedded in pre-made grooves and bonded in place with an epoxy-based paste. The main difference between these two techniques belongs to the surface preparation necessary before the application of the strengthening that in turn depends upon the conditions of the concrete substrate on which the laminates and bars are bonded.

Before surface preparation for FRP application, the central crack was repaired in order to re-establish material continuity and assure no water percolation through the crack. For this purpose, the crack was sealed using an epoxy-paste and then injected with a very low viscosity resin as shown in Fig. 8a-b. Once the crack had been repaired, FRP have been applied following the design provisions.

The design of both FRP technologies is carried out according to the principles of ACI 440.2R-02[6] (ACI 440[6] in the following). The properties of the FRP composite materials used in the design are summarized in Table 3 and Table 4. The reported FRP properties are guaranteed values.

The ϕ factors used to convert nominal values to design capacities are obtained as specified in AASHTO[4] for the as-built and from ACI 440[6] for the strengthened members.

Material properties of the FRP reinforcement reported by manufacturers, such as the ultimate tensile strength, typically do not consider long-term exposure to environmental conditions, and should be considered as initial properties. FRP properties to be used in all design equations are given as follows (ACI 440[6]):

$$f_{fu} = C_E f_{fu}^* \\ \varepsilon_{fu} = C_E \varepsilon_{fu}^* \tag{8}$$

where f_{fu} and ε_{fu} are the FRP design tensile strength and ultimate strain considering the environmental reduction factor *(C_E)* as given in Table 7.1 (ACI 440[6]), and f^*_{fu} and ε^*_{fu} represent the FRP guaranteed tensile strength and ultimate strain as reported by the manufacturer. The FRP design modulus of elasticity is the average value as reported by the manufacturer.

Externally Bonded CFRP Laminates

The material properties of the laminates that have been used are listed on Table 3. The design for externally bonded laminates called for a total of six, *12 in* (*30.48 cm*) wide, single ply CFRP strips overlapping at center span for *10 ft* (*3.05 m*). The strips were evenly spaced over the width of *20 ft* (*6.09 m*) and ran the entire width of the slab, as shown in Fig. 9. The moment capacity provided with this strengthening scheme is equal to ϕM_n*=16.5 k-ft* (*23 kN-m*). The CFRP laminates were applied by a certified contractor in accordance to manufacturer's specification[9] (see Fig. 10).

Near Surface Mounted Rectangular Bars

The material properties of the NSM and epoxy paste that have been used are listed on Table 4. The required number of near-surface mounted reinforcement was determined to be two CRFP tapes per slot on a *9 in* (*22.86 cm*) groove spacing. The bars were embedded in *17 ft* (*5.18 m*) long, *¾ in* (*19.05 mm*) deep, and *¼ in* (*6.35 mm*) wide grooves cut onto the soffit of the bridge deck as shown in Fig 11. The moment capacity provided with this strengthening scheme is equal to ϕM_n*=15.5 k-ft* (*21.01 kN-m*). NSM bars were applied by a certified contractor following the specifications[8] prescribed by the University of Missouri - Rolla (see Fig. 12).

IN-SITU LOAD TESTING

In order to validate the behavior of the bridge prior and after strengthening, static load tests were performed with a H20 truck (see Fig. 13). Although H20 and HS20 trucks differ in their geometry, the loading configuration that maximize the stresses and deflections at mid span could still be accomplished (see Fig. 14).

Displacements in the longitudinal and transversal direction were measured using eight Linear Variable Differential Transducers (LVDTs) and a data acquisition system under a total of three passes, one central and two laterals. For each pass three stops were