

## **Design, Construction, and Field-Testing of an RC Box Culvert Bridge Reinforced with GFRP Bars**

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### **ABSTRACT**

This paper presents an overview of the design, construction, and laboratory and field testing of a box culvert bridge reinforced with glass FRP (GFRP) bars. The bridge was constructed to replace a bridge that was built in the early 1980s and consisted of three concrete-encased corrugated steel pipes. Due to excessive corrosion of the steel pipes, the original bridge became unsafe to operate. The new box culvert units were designed for maximum forces determined in accordance with AASHTO design guidelines. A concrete precaster fabricated the box culvert units that were reinforced entirely with GFRP bars pre-bent and cut to size by the manufacturer. Two specimens were tested in the lab to verify their design and performance. The boxes were subjected to quasi-static loading cycles up to failure. Deformations and reinforcement strains were measured throughout the test. Test results compared well with theoretical values. The new bridge was opened to traffic in October 1999. The long-term performance of the new bridge is being monitored through periodic in-situ load tests, the first of which was conducted eight months after construction. Deflection measurements taken during the test were small and indicated an elastic bridge behavior.

### **INTRODUCTION**

Culverts are commonly made from a variety of materials including reinforced concrete, corrugated metal, and stone. Precast reinforced concrete (RC) box culverts are very common and usually constructed as single or multicell culverts. Precast RC box culverts offer advantages such as enhanced quality control, use of higher strength concrete, lower cost due mass production, and shorter installation time. In the 1970s there was a significant boom in culvert bridge construction in many countries. Most of these bridges, made of corrugated metal or reinforced concrete, are now approaching 30 years of age and are deteriorating at a high rate. Culvert bridges are subjected to aggressive environments (e.g., exposure to high moisture and treatment with deicing salts). For these structures, combinations of moisture, temperature, and chlorides accelerate the corrosion of the metal pipes or the internal steel reinforcement leading eventually to loss of serviceability. Reinforcement bars made of fiber reinforced polymers (FRP) are corrosion resistant and therefore are suitable for internal reinforcement of RC box culverts.

This paper describes the construction, performance, and long-term monitoring program of a concrete box culvert bridge reinforced entirely with glass FRP (GFRP) bars. The project aimed at investigating the feasibility and effectiveness of FRP internal reinforcement to reduce corrosion-related problems thus increasing the service life of RC culvert bridges. The project consists of design and laboratory verification, field construction, and in-situ load testing for performance monitoring of the new bridge. RC box units were tested in the laboratory to verify the design approach, serviceability conditions, and the ultimate capacity of a box unit.

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## BRIDGE DESCRIPTION

Walker Bridge is located in the City of Rolla, Missouri. The original bridge was constructed in the 1970s with a roadway width of 4.9 m. The bridge consisted of three 1.1 m diameter corrugated steel pipes encased in concrete. Due to corrosion problems, the bridge was scheduled for demolition and a new RC box culvert was to be installed.

## DESIGN OF GFRP REINFORCED BOX CULVERT

Steel reinforcement for precast box sections is commonly determined using the standard design tables presented in ASTM C 789 for box culverts with more than 0.6 m of cover and ASTM C 850 for box culverts with less than 0.6 m of cover. These standard designs are based on ACI's ultimate strength design method (Heger et al., 1976). Special designs for sizes and conditions other than as presented in the ASTM standards are also possible and should conform to the requirements of the respective specifications (ASTM C850).

Based on consideration of hydraulic requirements and site characteristics, the dimensions of the box culvert units were selected to be 1.5 by 1.5 m with a wall thickness of 150 mm. Analysis was achieved using BOXCAR, a software program that was developed for the Federal Highway Administration (FHWA). BOXCAR performs structural analysis and design of buried single cell reinforced concrete box culverts in accordance with AASHTO live load requirements and design requirements (AASHTO, 1996). Based on user-specified box geometry, material properties and loading data, the program computes the maximum design moments, shears and thrust forces at critical locations using the stiffness matrix method and considering the most severe case of 32 load conditions (FHWA, 1989). Using this program the analysis was carried out on the a box unit considering HS15 truck loading and 0.58 m of cover that included the topping soil and asphalt overlay. The maximum positive and negative moments due to factored loads were determined to be 13.1 and 9.7 kN-m/m, respectively. Both maximum moments occurred at mid-span of the bottom slab panel of the box unit. The maximum negative moment for the panel ends occurred at the bottom of the panel and was equal to 7.9 kN-m/m. Maximum shear and thrust forces were 18.4 and 10.7 kN, respectively. Maximum forces in the box herein reported correspond to different loading conditions.

The internal reinforcement considered for this application consisted of a commercially available  $\phi 6$  GFRP bar with guaranteed tensile strength  $f_{fu}^*$  of 758 MPa, a guaranteed modulus  $E_f$  of 40.7 GPa, and guaranteed rupture strain  $\epsilon_{fu}^*$  of 1.9%. The design strength of the bar  $f_{fu}$  was taken as  $0.7f_{fu}^*$ . Two cases were considered for design: at panel mid-span with ultimate moment demand of 13.1 kN-m/m and at panel ends with ultimate moment demand of 7.9 kN-m/m. For negative moments at panel ends, the strength of the FRP bars at the bend was determined using the formula proposed by the Japan Society of Civil Engineers as follows (JSCE, 1997):

$$f_{fb} = \left( 0.05 \cdot \frac{r_b}{d_b} + 0.3 \right) f_{fu} \leq f_{fu} \quad (1)$$

For the current design, the specified bend radius  $r_b$  was equal to four bar diameter  $4d_b = 4(6 \text{ mm}) = 24 \text{ mm}$ .; therefore the design strength at the bend was  $f_{fb} = 0.5 f_{fu}$ . Although the slab section of the box was to be reinforced in a symmetric manner, the contribution of compression reinforcement was disregarded. The design was achieved based on

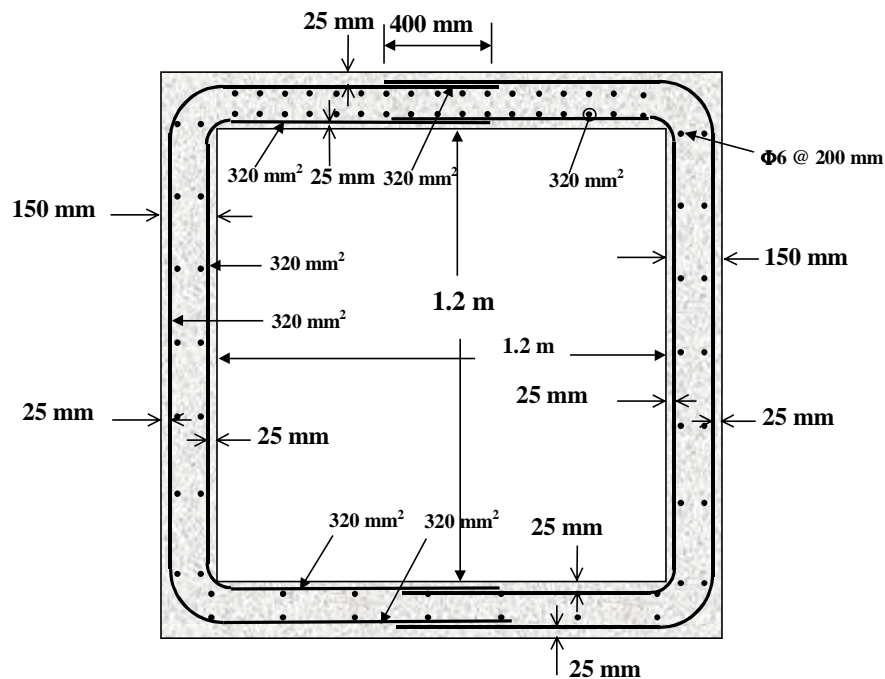
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principles of equilibrium and compatibility, and the constitutive laws of the materials. A strength reduction factor  $\phi$  of 0.75 was used for flexure and shear. Using this approach, a concrete strength of 34.5 MPa, and GFRP area of  $320 \text{ mm}^2/\text{m}$  ( $\phi 6 @ 100 \text{ mm. c/c}$ ) the design strength  $\phi M_n$  of panel was 15.5 kN-m/m and 8.0 kN-m/m at mid-span and supports, respectively. Under service load, the maximum stress computed in GFRP bars was 154 MPa ( $0.29 f_{fu}$ ). The corresponding crack width at service level was 0.5 mm. Given that the box section at service level are expected to be uncracked ( $M_{cr} = 38.2 \text{ in-k/ft}$ ), the service behavior of the box unit was considered satisfactory.

For ease of construction, it was decided to use U-shaped FRP bars, overlapped at the sides of the box. The development length of FRP bars was determined using a conservative formula that was based on test results available in the literature as follows (Ehsani et al. 1996 and Gao et al. 1998):

$$\ell_{df} = \frac{d_b f_{fu}}{2700} \quad (2)$$

where  $f_{fu}$  is in psi. The length of a lap splice was determined using  $1.6\ell_{df}$  as proposed by Benmokrane et al. (1997). Using Eq. (2), the required development length was  $\ell_{df} = 175 \text{ mm}$ . Accordingly, the required lap-splice length was 280 mm and a lap splice length of 400 mm was adopted. This length was determined by substituting the guaranteed tensile strength of the bar in Eq. (2). As per AASHTO recommendations (AASHTO, 1996), transverse reinforcement was provided in top panel ( $\phi 6 @ 100 \text{ mm. GFRP bars}$ ) to ensure load distribution. Transverse reinforcement ( $\phi 6 @ 200 \text{ mm. GFRP bars}$ ) was also provided in the walls and bottom panel to facilitate the reinforcement cage construction. The results of the final design of the box culvert unit are shown in Figure 1.



**Figure 1.** Details and reinforcement of GFRP reinforced box culvert.

### FABRICATION OF BOX CULVERT UNITS

A concrete pipe precaster fabricated the box culvert units. The boxes were entirely reinforced with glass FRP reinforcement. The bar manufacturer pre-bent and cut the reinforcing bars to the required sizes and shapes. The bars were tied together using plastic ties, as shown in Figure 2. Figure 3 shows a completed cage prior to casting. The GFRP reinforcement in two boxes was instrumented with strain gages and fiber optic sensors for long-term monitoring purposes. In addition, two more boxes were instrumented with strain gages for laboratory testing purpose (see Figure 4). The specified concrete strength was 34.5 MPa. Conventional concrete made of portland cement, fly ash, water, and Missouri River aggregate with a maximum aggregate size of 3/8 inch was used to cast the box units. The box units were cast using steel forms consisting of an exterior shell and an interior shell. Plastic wheel spacers were used to maintain a cover thickness of 1 inch, as shown in Figure 5. The concrete boxes were cast using a dry cast process that uses low frequency-high amplitude vibration to distribute and densely compact the mix in the form. Using this procedure, it was possible to remove the form immediately after casting. The boxes were manufactured with tongue and groove joints.



**Figure 2.** Rig for GFRP cage assembly



**Figure 3.** GFRP cage



**Figure 4.** Strain gage on GFRP bar

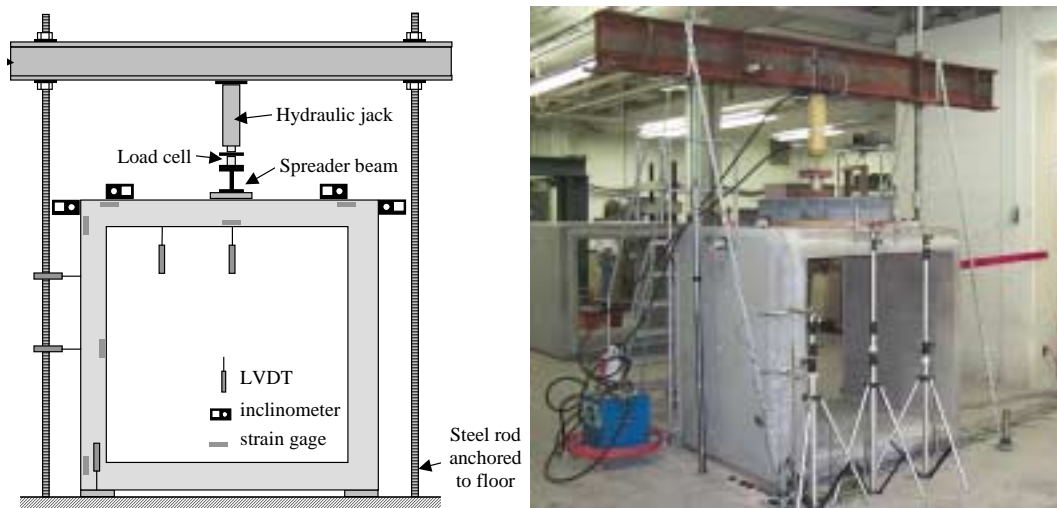


**Figure 5.** Plastic wheel spacers

## LABORATORY VERIFICATION

In addition to quality control tests by the precaster, eight standard concrete cylinders were acquired from different concrete batches and tested in the lab to determine the compressive strength of the concrete. The average strength of concrete cylinders was 42.7 MPa.

Two box culvert units (Box 1 and Box 2) were instrumented and tested in the laboratory. The test setup and instrumentation are shown in Figure 6. The two specimens were identical to the units installed in the field. Each specimen was instrumented with five LVDTs and four inclinometers to measure deformations and rotations, respectively. Load was applied using a hydraulic jack that reacted against a steel beam anchored to the strong floor using two high-strength threaded steel rods. The load was distributed across the top slab at mid-span using a steel beam. Load measurement was obtained using a load cell. Specimens were tested to failure by applying quasi-static load cycles in which the magnitude of the maximum load used in each successive load cycle was incremented until mid-span deflection of the top slab become excessive and failure signs were observed.



**Figure 6.** Test setup and instrumentation

## TEST RESULTS AND DISCUSSION

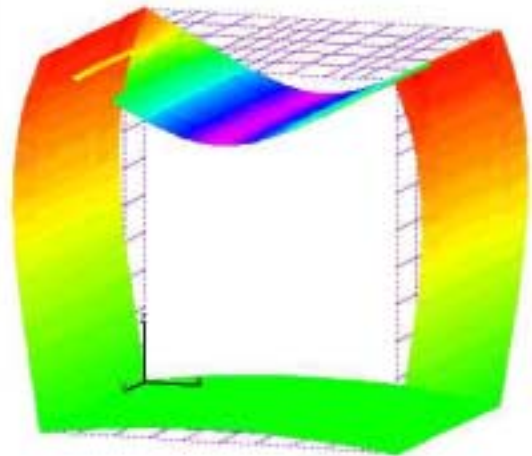
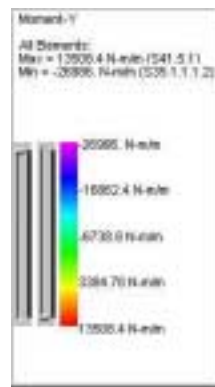
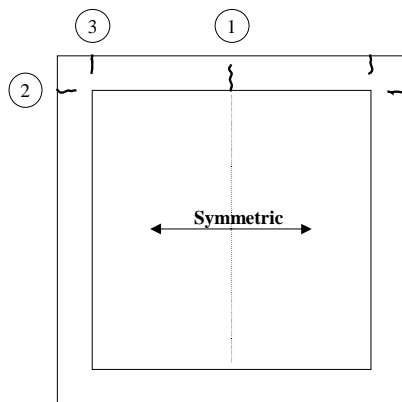
For the first box, the first crack occurred at mid-span of the top panel at approximately 124.5 kN. This cracking load was comparable with the theoretical cracking load of 112.5 kN, computed based on elastic behavior. The following cracking occurred at the negative moment region at the top of the wall panels. Two cracks occurred simultaneously on both walls at approximately 142.3 kN. Final cracks occurred at the negative moment region of the top slab panel at approximately 155.7 kN, the maximum load attained during the test. Figure 7 shows location and sequence of cracking. Occurrence of each crack was accompanied by an increase in mid-span deflection caused by stiffness degradation. The behavior of the second box was similar to the first one.



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The box was modeled using Visual Analysis, a commercially available structural analysis software. The model consists of a three-dimensional mesh of horizontal and vertical shell elements with thickness equal to the thickness of the box walls. Pin supports were assumed at the bottom end of the wall panel on one side, and roller supports on the wall on the other side. The load applied to the model consisted of a uniform load distributed at mid-span of the top panel over a width of 150 mm. The uniform load was calculated such that its resultant was equal to the point load desired for analysis. Since the software is limited to linear elastic analysis, the behavior of the box unit after cracking was investigated by assuming elements with smaller stiffness at crack locations. The width of these elements was arbitrarily taken as 3 inches. The thickness of these elements was calculated such that the moment of inertia of its cross section was equal to that of the cracked section. The analysis of the box section indicated that when GFRP bars rupture, the maximum concrete strain is 0.0014. Considering this strain value and that GFRP bars are elastic up to failure, using linear elastic model was reasonably accurate for the current case.

The load was applied to the model in increments at which mid-span deflection of the top slab was recorded. When the applied load was equal to a cracking load, elements at crack locations were modified as indicated above. This procedure was continued until the full load-deflection diagram was obtained. Theoretical results indicated that, after all the cracks were modeled, the moment at top panel mid-span due to a total applied load of 133.4 kN was approximately 27.6 kN-m/m. This capacity was comparable to the analytical strength of the section without reduction factors, which was 29.5 kN-m/m. The moment distribution in the model at maximum load is shown in Figure 8. Figure 9 illustrates a comparison of theoretical and experimental results of the two boxes where a good correlation could be observed. Also, as seen in this figure, the initial cracking load is well above the ultimate design load. The box culvert units are therefore not expected to crack at service level.



**Figure 7.** Cracking locations

**Figure 8.** Theoretical moment distribution at maximum load.



**Figure 9.** Comparison of experimental and theoretical load-deflection behavior

#### COMPARISON WITH PROPOSED DESIGN APPROACH BY ACI COMMITTEE 440H

According to the latest draft of ACI Committee 440H (ACI 440H, 2000), the design tensile strength of GFRP bars  $f_{fu}$  for reinforcement of concrete exposed to earth and weather should be taken as  $0.7f_{fu}^*$  and the strength at the bend portion  $f_{fb}$  is calculated using Eq. (1). The strength of an FRP reinforced flexural member  $M_n$  in which failure is governed by FRP rupture can be conservatively calculated using the following expressions:

$$M_n = 0.8A_f f_d \left( d - \frac{\beta_f c_b}{2} \right) \quad (3)$$

$$c_b = \left( \frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fu}} \right) d \quad (4)$$

in which,  $A_f$  is the area of FRP reinforcement,  $c_b$  is the depth of neutral axis at balanced strain condition,  $d$  is the depth of tensile reinforcement,  $f_d$  design strength equal to  $f_{fu}$  or  $f_b$  depending on design condition, and  $\epsilon_{cu}$  is the ultimate strain of concrete for design taken as 0.003. For strength controlled by FRP rupture, the committee proposed a conservative strength reduction factor  $\phi$  of 0.5 to ensure adequate margin of safety against brittle-type failure. Using this approach, the design strength  $\phi M_n$  of the box panel was 21.8 in-k/ft and 10.9 in-k/ft at mid-span and supports, respectively. Comparing with the observed behavior of tested box units, these results are very

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conservative (see Figure 9). Design based on the approach proposed by committee 440 would yield a stiffer cracked member with relatively high margin of safety against failure.

### **BRIDGE CONSTRUCTION**

The new bridge was 36 ft wide and consisted of 18 precast concrete boxes arranged in two rows, nine boxes per row. A crew from Rolla City Public Work Department constructed the new bridge, as shown in Figure 10. The new Walker Avenue Bridge was opened to traffic on October 13, 1999 (see Figure 11).



**Figure 10.** Bridge construction



**Figure 11.** New Walker Avenue Bridge

### **FIELD LOAD TESTING**

The in-situ load test was the first of a series of tests aimed at monitoring the performance of the bridge over the course of three years. In each test, the elastic response of the bridge is measured under similar loading and weather conditions.

The first elastic load test was performed 8 months after the bridge was constructed using a test vehicle with a front axle load of 64.5 MPa. and a rear axle load of 65.4 MPa. Deflection measurements were taken using LVDTs located at the center of three consecutive box units, as shown in Figure 12. The data was collected for five axle locations at quarter points of the clear wall-to-wall distance on the transverse axis of the north and south boxes of the bridge. Axle locations are also shown in Figure 12. For each box row (North and South), the test was performed by stopping the vehicle at the predetermined location and measuring the resulting deflections. The test truck made two passes over the bridge, one for each box (row). The truck was driven forward until the front wheel line coincided with the first mark (location 1). Deflection measurements were then taken and the test vehicle was then slowly moved to the following mark (see Figure 13) and deflection measurements were taken again. This process was continued until all measurements were obtained. The vehicle was then driven until the rear wheel line coincided with the first mark on the box. Deflection measurements were taken in a similar fashion to that of the front wheel line. The test vehicle was then backed up for few minutes until deflection measurements stabilized and the whole process was repeated for the South box.



### IN-SITU LOAD TEST RESULTS

Table 1 presents tabulated results of the bridge deck deflections measured on the North Box. Maximum deflection due to front axle loading was 0.206 mm measured on LVDT #1 when the wheel line was at location 3 (mid-span). Maximum deflection due to rear axle loading was also measured on LVDT #1 when the wheel line was at location 3. Maximum deflection due to rear axle loading was 0.409 mm. As seen from the test data, the measured deflections on the three consecutive box units were not uniform. Box units subjected to a similar loading configuration (e.g., at LVDT #1 and LVDT #3) measured larger deflections on LVDT #1 for both front and rear axle loading. A number of hair-like cracks were observed on these box units. Their larger deflections could therefore be related to a lower flexural stiffness caused by cracking. Deflections measured on the middle box unit (LVDT #2) were always less than those measured on the other two units (LVDT #1 and LVDT #3). This behavior indicates that only small portion of the applied load is carried from the loaded unit to an adjacent unit. However, such behavior depends on the relative stiffness of adjacent box units.

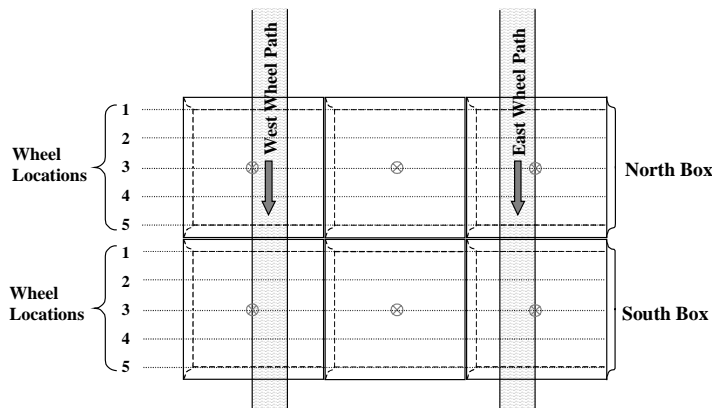


Figure 12. LVDT Layout and Axles Locations



Figure 13. In-situ load test

Table 1. Maximum Deflections Measured on the North Box

Loading	Axle Location	LVDT Deflections (inch x 10 <sup>-3</sup> )		
		#1	#2	#3
Front Axle	1	0.066	0.005	0.056
	2	0.150	0.008	0.109
	3	0.206	0.013	0.114
	4	0.183	0.010	0.104
	5	0.099	0.008	0.058
Rear Axle	1	0.109	0.018	0.109
	2	0.249	0.025	0.211
	3	0.409	0.030	0.218
	4	0.328	0.018	0.157
	5	0.196	0.008	0.086

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## CONCLUSION

This paper presents an overview of the design, construction, and laboratory and field testing of a box culvert bridge reinforced with internal glass FRP bars. Box units were designed for failure controlled by GFRP rupture using equilibrium and compatibility equations. Laboratory testing indicated that the design approach is accurate and conservative and the experimental displacement behavior compared well with theoretical values. The design approach proposed by ACI Committee 440H is very conservative and would yield a stiffer member. The in-situ load test of the bridge indicated that bridge deflections were very small. Elastic deflection of the box units located under the west wheel path were higher than those obtained under the east wheel path under similar conditions. Variation in the elastic response of box units under similar loading condition was related to the presence of minor cracks. After load was removed there was no residual deflections. Results obtained from this test will be used as a benchmark.

## ACKNOWLEDGEMENTS

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