ALTERNATIVE DECK SYTEM FOR FOLDED PLATE GIRDER

Quarterly Report

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Sponsored by

Accelerated Bridge Construction University Transportation Center

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1. PROJECT ABSTRACT

Folded plate girder (FPG) is constructed using a steel sheet, which is bent using a bending machine to create box girder with an open bottom flange. FPG can be used in accelerated bridge construction (ABC) by casting full-depth deck panel on top of the FPG which in a precast plant. In order to increase the advantage of FPG in ABC projects, alternative deck systems such as laminated wood or orthotropic decks or prefabricated decks are proposed for FPG. Advantages of the proposed deck systems include accelerated construction if compared to cast-in-place construction. In the suggested experimental work, a large-scale specimen will be tested under fatigue loading for service life design and under ultimate load for AASHTO strength design. The FPG also will suitable for spans up to 100 ft. allowing more bridges to take advantage of such system. Final Report will be prepared to include the design recommendation for the proposed ABC system.

2. RESEARCH PLAN

2.1. PROBLEM STATEMENT

Folded plate girder (FPG) is a superstructure bridge system, which involves a cold bend out of a single sheet with an open bottom flange. The cold bend eliminates the costly and inconsistent shop weld found in conventional steel girders. The FPG concept works for both conventional construction and accelerated bridge construction (ABC). In conventional construction, formwork is needed along with the placement of deck reinforcement then the concrete is placed. In ABC, a full-depth deck panel unit is fabricated in a precast planet of in fabrication yard near the bridge site with transverse steel reinforcing bars extended outside the cured slab portion to form closure joint with adjacent full-depth deck unit then ultra-high performance concrete (UHPC) or normal strength concrete is placed in closure joints.

2.2. RESEARCH APPROACH AND OBJECTIVES

The main objectives of this project are:

1- The main objective of this project was to develop new deck systems for FPG aiming at cost reduction and much faster construction, However, the objective of this project was revised to include the results from fatigue and ultimate test conducted at FIU for the extended length of FPG.

2- Conducting a proof of a concept experimental work on full-scale FPG with either under fatigue and ultimate loads for the FPG with extended length.

Assessment of the performance of the proposed deck system compared to cast in place deck in conventional construction.

3. DETAILED WORK PLAN

An overview of the study tasks is given below.

Task 1 – Literature Review

In this task, a comprehensive literature review on laminated wood decks and FPG will be conducted in order to identify the issues that may concern integrating laminated deck wood with FPG.

Progress: This task is 100% complete. Below is summary of the literature review

FPG has many advantages over other steel girder system including, 1) eliminating the shop welding, 2) eliminating transverse bracings, 3) Easy inspection through the open bottom flange. The FPG system was implemented in ABC using full-depth deck panels fabricated on top of the girder in Pennsylvania near Bradford as shown in Figure 1.

Figure 1 Folded steel plate girder system bridge in Pennsylvania erected near Bradford.

The FPG can be bent using a large steel sheet in 20-30 minutes using a bending machine. Due to the limitation of bending machine, single-span bridges using FPG are not typically exceeding 60 ft. The PI just completed the fatigue testing of FPG specimen using concrete deck system suitable for 100 ft. spans. This progress report presents the results of this system. An ultimate test for this specimen was planned to be conducted in March 2020 but postponed due to COVID 19 outbreak. The proposed system for 100 ft. span length utilizes one of the two connections shown in Figure 2.

Figure 2 Two details to expand the span of FPG girder to 100 ft.

Task 2–Specimen Design and Fatigue Test

In this task, the specimen dimension, test setup and instrumentation plan will be developed. In addition to

Progress: This task is 100% complete by conducting analysis of the fatigue test results on the specimen with concrete deck. Below is summary of the fatigue test results for the extended length of the FPG up to 100 ft.

SPECIMEN CONSTRUCTION

To allow inspection of the girder during the test, FPG had to be placed about 7 ft. above the ground. To achieve that, end supports were constructed with sufficient height to provide the required clearance for inspection and sufficient width to provide stability for the test specimen. Figure 3 shows the formwork and complete casting of the end supports. The final height of the end supports was decided based one some many factors including (1) human height for inspection and instrumentation attachment, (2) the ceiling height of the lab, (3) crane range, and (4) the height of test apparatus.

Figure 4 shows the test specimen placed on top of the end supports. In order to achieve that, (1) the girder specimen were lifted using two forklifts and was inserted through the MTS frame and placed diagonally with an angle, (2) the end support then were placed in the right position apart to generate 39 ft. span length of the girder, and (3) the girder was lifted again from the ground, rotated and placed as shown in Figure 4.

Figure 3 Construction of end supports

Figure 4 Placement of FPG on top of the end supports

In order to cast the top concrete slab, scaffolding was placed on both sides of the girder to support the overhanging portion of the slab during casting. Wooden formwork for the deck slab was designed accordingly and was placed on top of the scaffolds, as shown in Figure 5. Another scaffolding was also used under the girder. This replicates the casting of full depth deck panel on top of the FPG in precast plants with shored construction.

Figure 5 Placement of FPG on top of the end supports

Subsequently, the deck steel reinforcement was placed as designed. #5 bars @ 12” c/c both directions were provided at the bottom and #4 bars @ 12” c/c both directions were provided at the top as shown in Figure 6. Plastic chairs were used to ensure that reinforcement stays in their position during deck casting.

Figure 6 Placement of FPG on top of the End Supports

Figure 7 shows the final specimen. The average thickness of the deck slab was found to be 9.5 in. after casting. The sectional properties were revised for displacement ranges required during the fatigue testing since the original specimen design assumed 8 in. deck thickness. Spacer beam was placed between the top of the deck and the loading beam prior to testing.

Figure 7 Final test specimen.

TEST SETUP AND LOADING PROTOCOL

FATIGUE RESISTANCE EQUATION

The applied fatigue stress range is inversely proportional to the cubic root of the number of cycles according to AASHTO-LRFD (Eq. 6.6.1.2.5-2) as shown in Figure 8. S-N curves can then be plotted using this equation. The term (N) is the Number of cycling loading that bridge will be subjected to during its design life, which is assumed to be 75 years. This number can be calculated according to AASHTO-LRFD (Eq. 6.6.1.2.5-3). The term (A) is a fatigue constant based weld category in AASHTO-LRFD (Table 6.6.1.2.5-1). The term (n) is the number of stress range cycles the bridge experiences per truck passage based on AASHTO-LRFD (Table 6.6.1.2.5-2). For short span bridges n is 2. $(ADTT)_{SL}$ is the number of trucks per day in a single lane averaged over the design life specified in AASHTO-LRFD (Article 3.6.1.4). $(\Delta F)_{TH}$ is the constant amplitude fatigue threshold based on AASHTO-LRFD (Table 6.6.1.2.5-3).

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad (6.6.1.2.5\text{-}2)$$

in which:

$$N = (365)(75)n(ADTT)_{SL} \qquad (6.6.1.2.5\text{-}3)$$

where:

A = constant taken from Table 6.6.1.2.5-1 (ksi^3)
n = number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2
$(ADTT)_{SL}$ = single-lane $ADTT$ as specified in Article 3.6.1.4
$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3 (ksi)

Figure 8 Fatigue life equation from AASHTO-LRFD.

SIMULATION OF AASHTO TRUCK IN LAB BY USING S-N CURVE

A typical bridge will be subjected to millions of cycles of truck loading. As an example using equations listed in Figure 10, the number of times that trucks would pass over a bridge during its 75-years design life would be 219,000,000.

However, applying 219,000,000 would take a very long time. To shorten the cyclic test period and at the same time simulate the effect of truck traffic over 75-years design life, the following relationship can be used which is listed below along with the equation derivatives.

$$(\Delta F1)^3 * N1 = A$$

$$(\Delta F2)^3 * N2 = A$$

Now equating both equations:

$$(\Delta F1)^3 * N1 = (\Delta F2)^3 * N2$$

$$\Delta F1 = M1 * y/I \ \& \ \Delta F2 = M2 * y/I$$

Since the cross-sectional properties y & I remain the same, the equation can be simplified as follow:

$$\frac{M1}{M2} = \left(\frac{N2}{N1}\right)^{1/3}$$

Where:

M1= Moment produced by design truck

N1= Number of fatigue cycles induced over the design life of the bridge

M2= Magnified moment required to be applied to test specimen to Simulate AASHTO design truck for (N2).

N2= Number of cycles to be applied to the specimen

CALCULATION OF LOAD APPLIED

Using the relationship developed in Section 3.2.2, five million cycles were chosen to be applied to the specimen simulating 75-years design life for the bridge. This was achieved by applying higher loads at a lower number of cycles, as compared to219,000,000 cycles at much less load. During each cycle, the resulting tensile stress at the bottom flange was about 10 ksi, as compared to about 2.5 ksi that would be induced in fatigue limit state II corresponding to 219,000,000 cycles of truck passage

In short, applying 5,000,000 cycles of load, producing about 10 ksi in the bottom flange will be a point on S-N curve, the same way the point corresponding to 219,0000,000 cycle at about 2.5 ksi tensile stress. The point marked on the S-N curve is shown below in Figure 9.

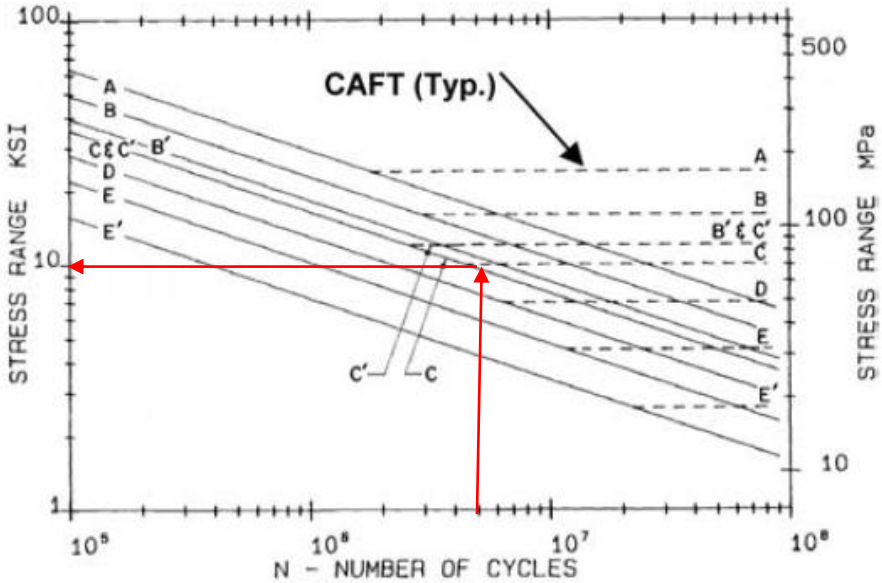


Figure 9 S-N Curve

The equation used for calculating N is shown below:

N= 365*75*n*p*$ADTT_{SL}$ (Table 6.6.1.2.5-3 AASHTO)

Where,

$ADTT_{SL}$=p*Fraction of Trucks *ADT (C3.6.1.4.2-1)

n= 2

P= 1, fraction of Traffic in a Single Lane (Table 3.6.1.4.2-1 AASHTO)

N = 75*365*2*1*0.2*20000

N= 219,000,000

Now calculating moment induced by HS20 truck on the bridge having span L=39 ft., tested in this study:

The Live Load Distribution Factor Calculated= 0.662

Load Factor for Fatigue= 0.75

Bending Moment Produced By HS20 Truck, M1= 192.2 K-ft

No. of Cycles Induced in Lab, N2= 5,000,000 & N1= 219,000,000

So Using the Equation, M2 can be calculated:

$$\frac{M1}{M2} = \left(\frac{N2}{N1}\right)^{1/3}$$

M_2= 669 K-ft

$P= 4*M_2/L$

$P= 69$ *Kips*

INSTRUMENTATION-STRAIN GAUGES

The test specimen was extensively instrumented to monitor possible changes in its behavior in case of the formation of fatigue cracking or another local failure. The data collected was used to evaluate the weld and specimen behaviors. Visual inspection of weld conditions was also performed at time intervals. Figure 10 shows the layout of the strain gauges. The Plan view shows four sections at which strain gauges were installed. Steel strain gauges were applied on the girder at two locations near mid-span (opposite sides of the weld), adjacent to welds, and at a quarter of the span each side. The blowup in Figure 12b shows the strain gauges applied at mid-span on one side of the weld. The denotation ML refers to Middle left. Twelve strain gauges were applied on each side across the weld. In total 44 steel strain gauges were applied. In addition to the steel strain gauges, four embedded vibrating wire strain gauges were placed inside the concrete deck. Three surface strain gauges at the top of the deck slab at mid-span were also attached as shown in section A-A.

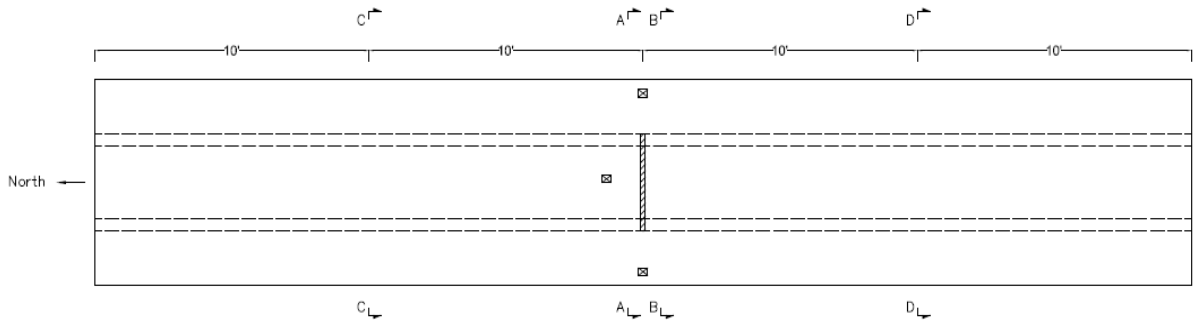


Figure 10a. Strain gauge layout in plan.

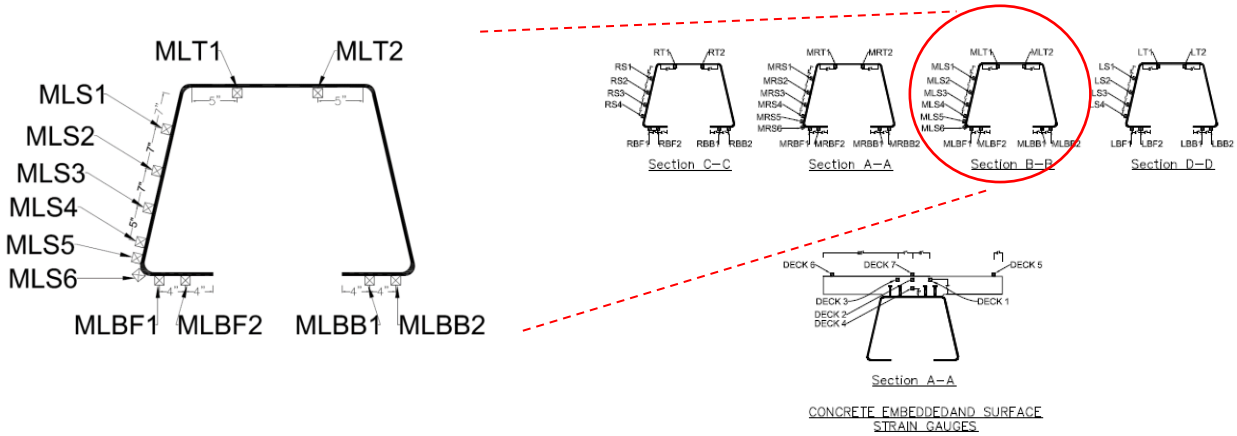


Figure 10b. Strain gauge layout in cross section.

INSTRUMENTATION-POTENTIOMETERS:

Global behavior of the test specimen was also monitored, through monitoring the specimen deflection at several points. Potentiometers were deployed to measure the deflections of the

girder at three locations; mid-span and each quarter point. Girder ends were placed on an elastomeric pad. Therefore, there was a deflection at girder ends. Therefore, the deflections of girder ends were also monitored by placing potentiometers at girder ends. The schematic Elevation and section with applied string potentiometers are shown in Figure 11.

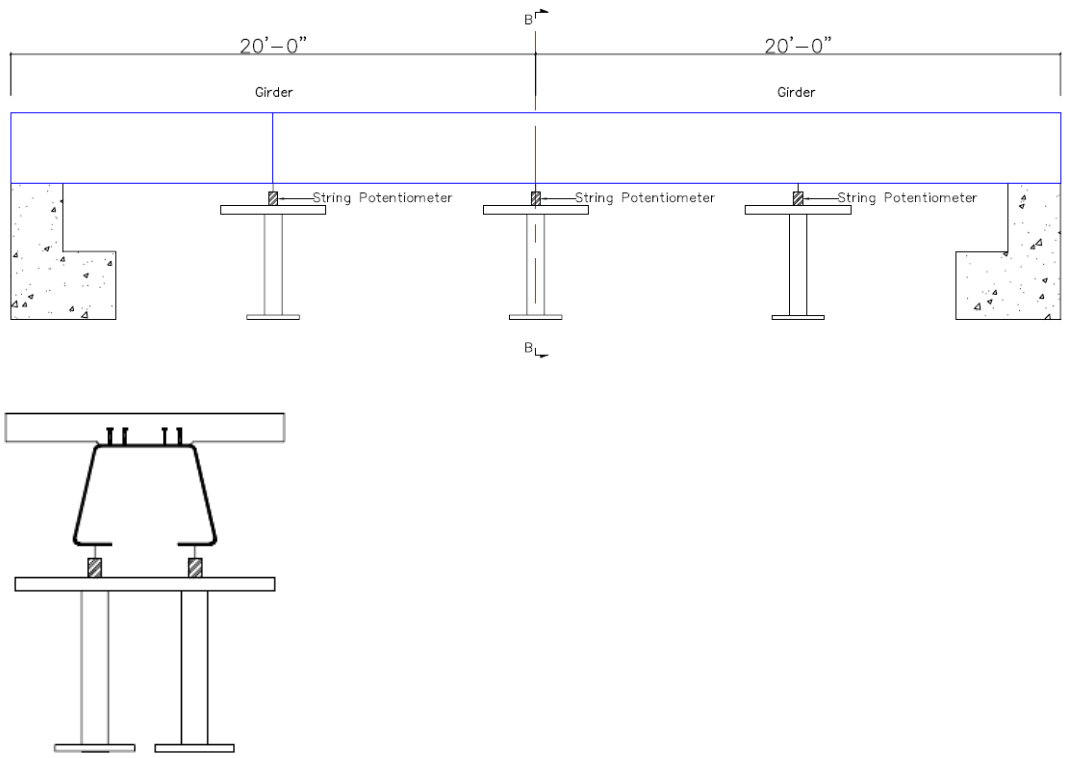


Figure 11. Potentiometer Layout, Elevation and Section.

FATIGUE TEST RESULTS

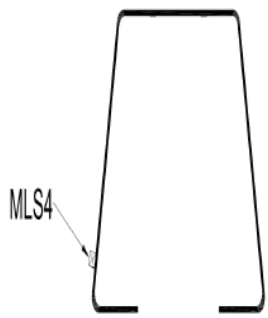
Strain Results:

Data from strain gauges placed at various locations were recorded and analyzed. Table 1 shows the range of strains against the range of force applied along the course of the experiment. This result is shown for the strain gauge at bottom flange located at mid-span on the left side of the weld. Figure 12 is the graphical representation of the range of strains along the course of the experiment.

Analysis of The Results

Based on the calculations carried out before the experiment, It was known what should be the applied load at mid-span to produce tensile stress of about 10 ksi in the bottom flange and corresponding tensile strain. This data allowed to calculate the expected stress range. During the test because of minor changes to boundary conditions, the observed strain ranges changed slightly. Table 1 depicts the deviation of strain range from expected values. It can be seen that the difference was less than about 5%.

Table 1. Range of strains along course of experiment for MLS4.

LOCATION OF SENSOR	NO. OF CYCLES (M)	RECORD	RANGE OF LOAD (KIP)			EXPECTED STRAIN DIFFERENTIAL	RANGE OF STRAIN (με)			% DIFF-STR EXPECTED-STR
			MIN	MAX	DIFF		MIN	MAX	DIFF	%
	0.17	AFTER	6.3	81.0	74.7	344.0	28	372	344	0.00%
	0.55	AFTER	6.6	81.0	74.4	342.6	30	374	344	0.40%
	0.80	AFTER	5.3	76.0	70.8	325.8	22	360	338	3.74%
	1.05	AFTER	5.1	76.8	71.7	330.2	23	358	335	1.46%
	1.40	AFTER	4.2	75.0	70.8	326.0	20	350	330	1.21%
	1.55	AFTER	6.5	79.5	73.0	336.2	40	380	340	1.14%
	1.90	AFTER	7.3	80.0	72.7	334.8	30	377	347	3.65%
	2.25	BEFORE	7.0	79.0	72.0	331.6	27	366	339	2.24%
		AFTER	6.5	81.0	74.5	343.1	28	370	342	-0.31%
	2.50	BEFORE	5.0	78.0	73.0	336.2	19	361	342	1.73%
		AFTER	6.0	78.0	72.0	331.6	21	362	341	2.85%
	2.75	BEFORE	6.5	79.0	72.5	333.9	39	380	341	2.14%
		AFTER	7.0	79.0	72.0	331.6	38	380	342	3.15%
	3.15	BEFORE	6.0	78.0	72.0	331.6	33	378	345	4.05%
		AFTER	6.7	78.0	71.3	328.3	36	379	343	4.46%
	3.45	AFTER	7.0	78.0	71.0	327.0	40	380	340	3.99%
	3.75	BEFORE	5.0	77.0	72.0	331.6	27	370	343	3.45%
		AFTER	5.5	76.5	71.0	327.0	30	370	340	3.99%
	4.10	BEFORE	5.0	76.7	71.7	330.2	24	365	341	3.28%
		AFTER	5.5	76.5	71.0	327.0	27	365	338	3.38%
	4.30	BEFORE	5.0	75.5	70.5	324.7	30	370	340	4.73%
		AFTER	5.0	75.5	70.5	324.7	33	370	337	3.80%
	4.60	BEFORE	5.0	75.5	70.5	324.7	35	372	337	3.80%
		AFTER	5.0	76.0	71.0	327.0	35	372	337	3.07%
	5.00	BEFORE	3.7	74.0	70.3	323.7	16	352	336	3.79%

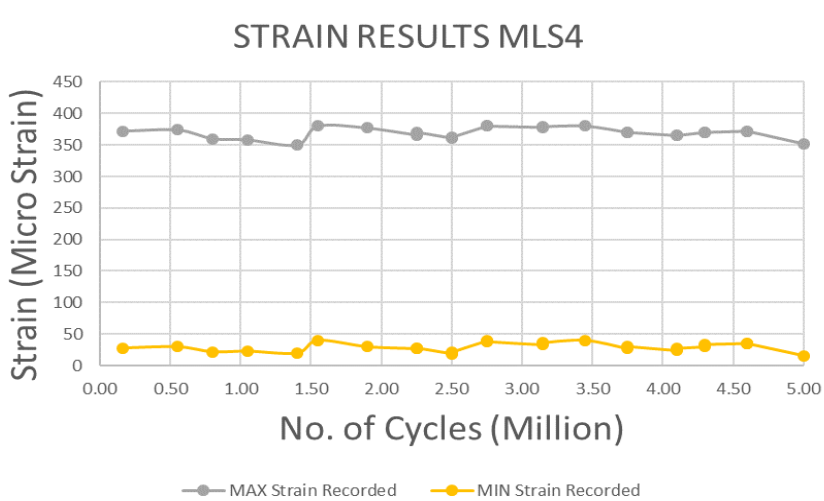


Figure 12. Range of strains along course of experiment FOR MLS4.

Strain Distribution Across Cross Section:

Strain distribution through the cross-section of the bridge at mid-span is shown in Figure 13. The brown line shows the strain predicted from hand calculations. Other lines show the strains produced in the cross-section after every 1 million cycles were completed. It can be noticed that the predicted values for tensile strain, in the steel, were in close agreement to those obtained. The linear strain distribution across the depth of the cross-section is an indication that full composite action was maintained throughout the test.

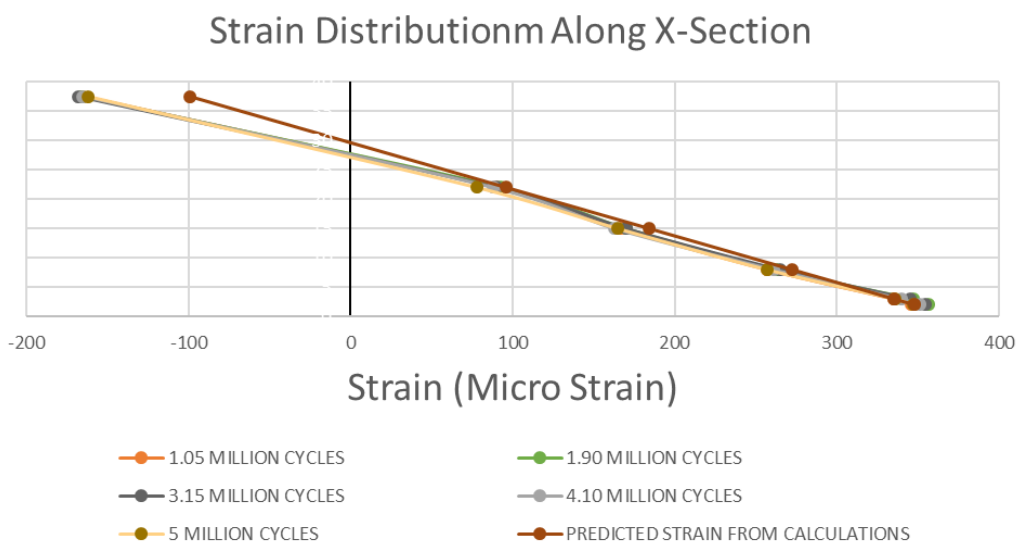


Figure 13 Strain Distribution across cross section of the bridge.

Stiffness

Stiffness (load/displacement) of the specimen along the course of experiment is shown in Figure 14. The reduction in the stiffness was not significant. Visual inspection was carried out after every 0.5 Million cycles along the course of the experiment and no visible cracks on steel girder was seen.

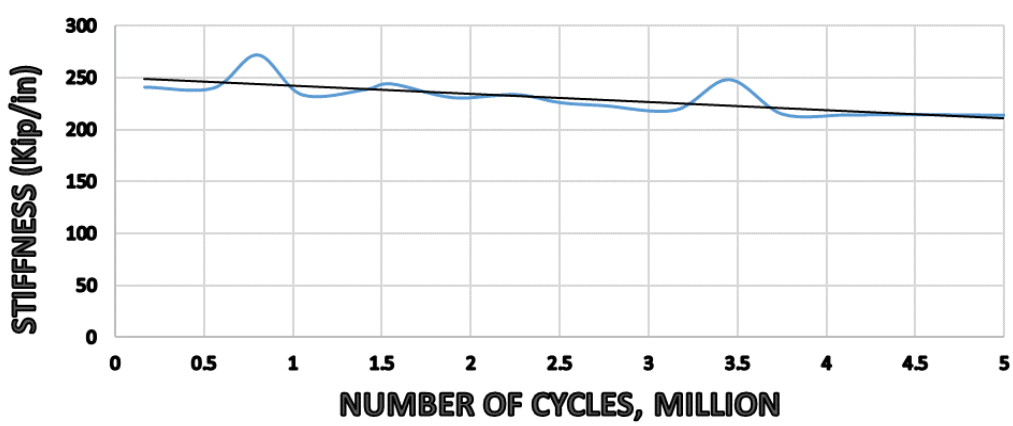


Figure 14 Stiffness Variation along the length of the experiment.

Task 3– Ultimate Testing

The test was conducted in November 2020 in order to measure the ultimate load capacity of folded plate girder. The test was conducted as 4-point load test and is shown below in Figure 15.

Figure 15 Ultimate Load Test Setup for Folded Plate Girder.

INSTRUMENTATION ULTIMATE TEST-STRAIN GAUGES

The instrumentation plan was similar to the one shown above in Figure 10b except for the addition of two more strain gauges on the connection plates which are joining the bottom flanges of the folded plate girder. The strain gauge setup used for the ultimate test is shown below.

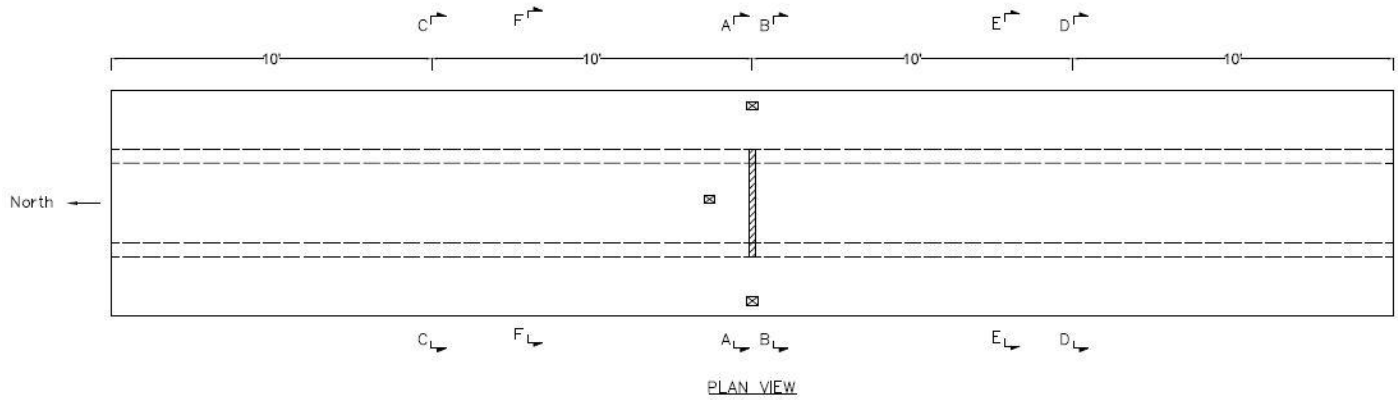


Figure 16a. Strain gauge layout in plan for ultimate test.

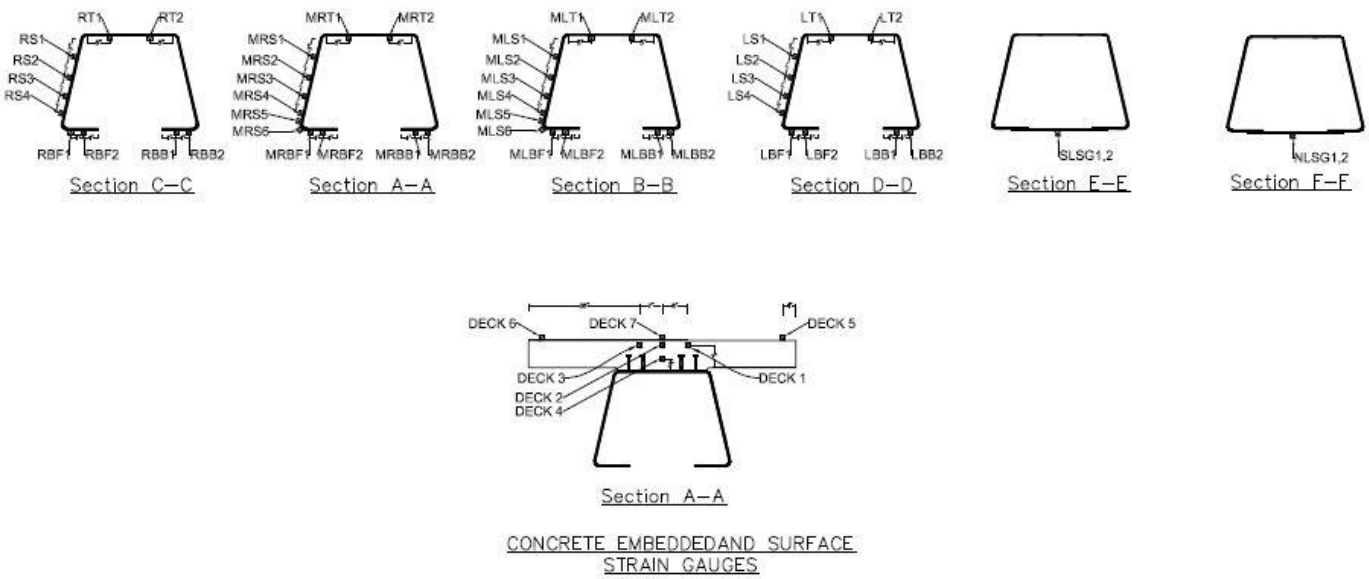


Figure 16b. Strain gauge layout in section for ultimate test.

INSTRUMENTATION ULTIMATE TEST-POTENTIOMETERS

The same potentiometers setup was used for the ultimate test, however there was an additional potentiometer attached with the deck slab to monitor and compare the deflection of deck slab with the folded plate girder. The section view of the deployed potentiometers is shown below. The potentiometers are denoted as DM which means displacement at mid span, next part represents the location i.e. deck means displacement of the deck.

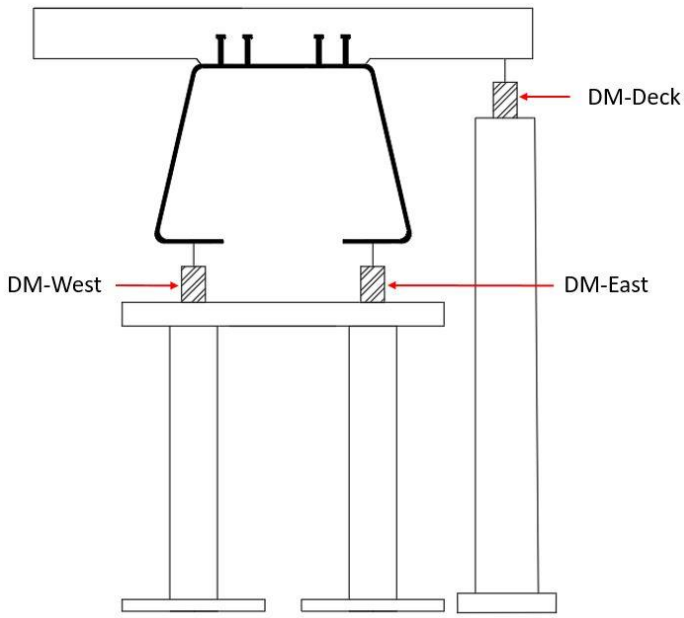


Figure 17. Potentiometer Setup, Ultimate Test

ULTIMATE TEST RESULTS

Strain Results:

Data from strain gauges placed at various locations were recorded during the ultimate test and analyzed. The variation in strain value against force for strain gauge MRS4 shown in Figure 16b has been plotted below. It can be seen from the result that the load has been increased in intervals of loading and unloading. The result shows the maximum strain of about 10600 micro strains at about 893 Kips of load.

Figure 18. Load Vs Strain

Strain Distribution Across Cross Section:

Strain distribution through the cross-section of the bridge at mid-span during ultimate load test is shown below. Each dot on respective line shows the location of the sensor along the cross section. Each line shows strain values in sensors at certain load i.e. 100 to 700 kips.

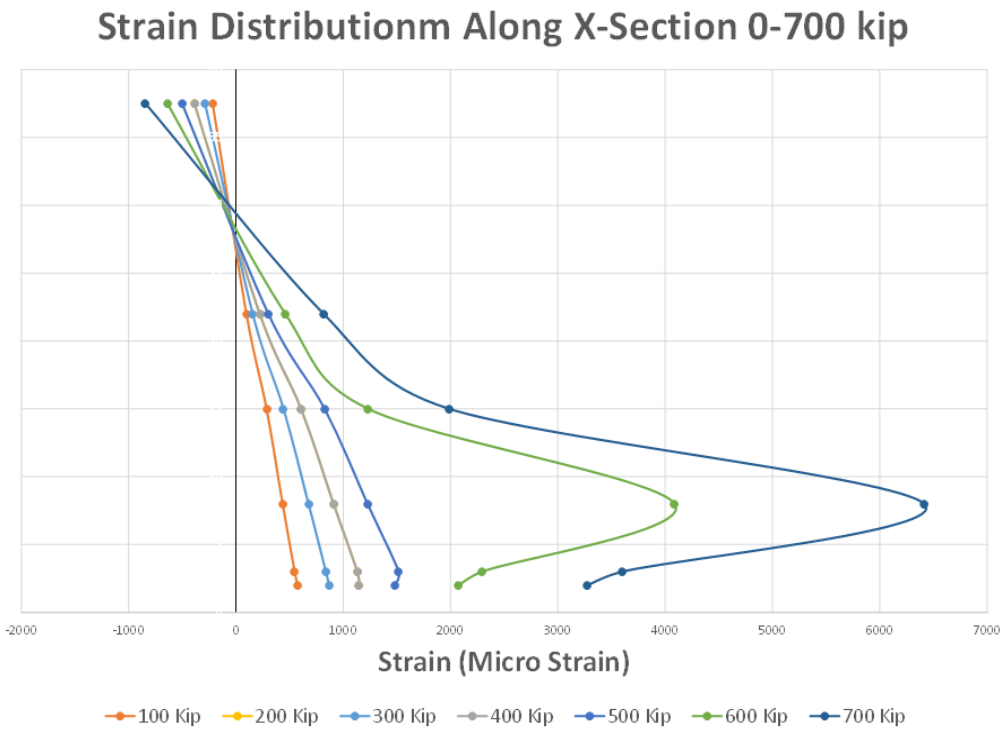


Figure 19. Strain Distribution Curve for loads 0-700 kips

Load Deflection Curve:

The deflection measurements were taken during the ultimate test from potentiometers as shown above in Figure 17. The load deflection curve for deck slab and potentiometer DM west are shown below in Figure 20a and 20b.

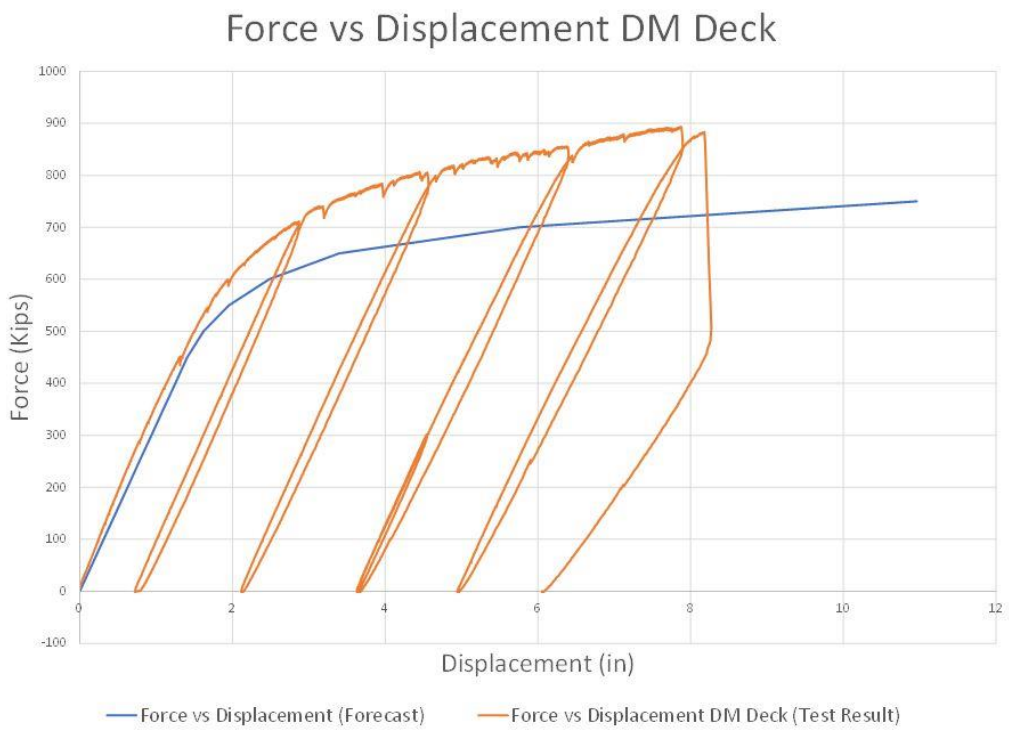


Figure 20a. Comparison of Force Vs Displacement, DM Deck with Forecast values

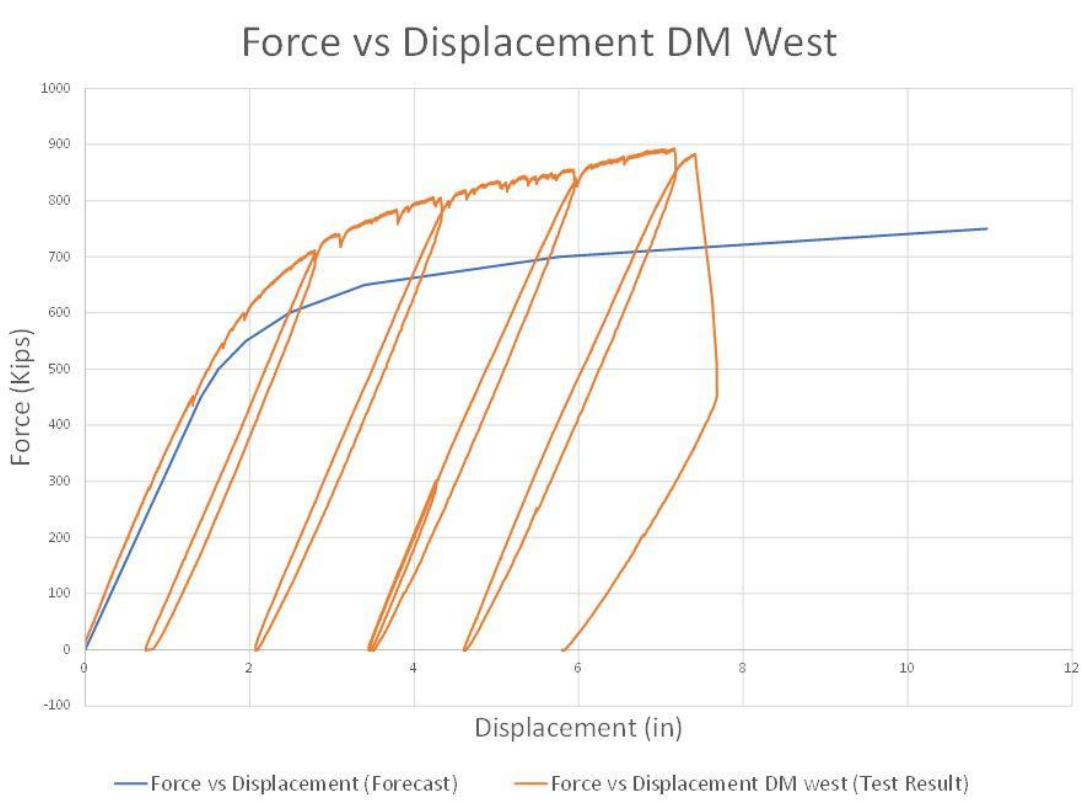


Figure 20b. Comparison of Force Vs Displacement, DM West with Forecast values

CAPACITY CALCULATION BY MOMENT CURVATURE ANALYSIS:

In order to forecast the moment capacity of the composite section of folded plate girder with concrete deck, moment curvature analysis was performed. The composite section is shown in Figure 21. The moment curvature curve is shown below in Figure 22. The maximum moment capacity calculated was 68932 Kip inches and the ultimate load predicted was 820.62 Kips.

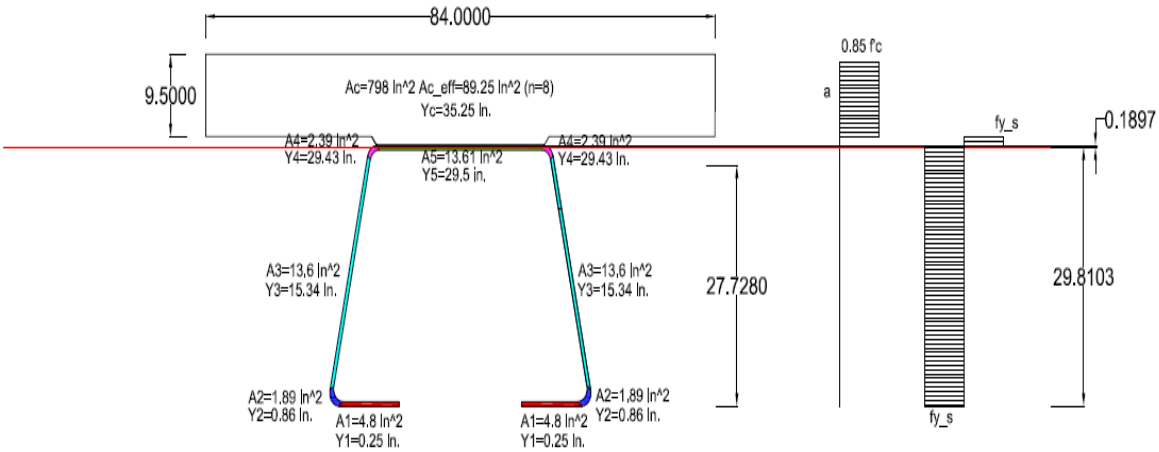


Figure 21. Composite Section of the Tested Bridge

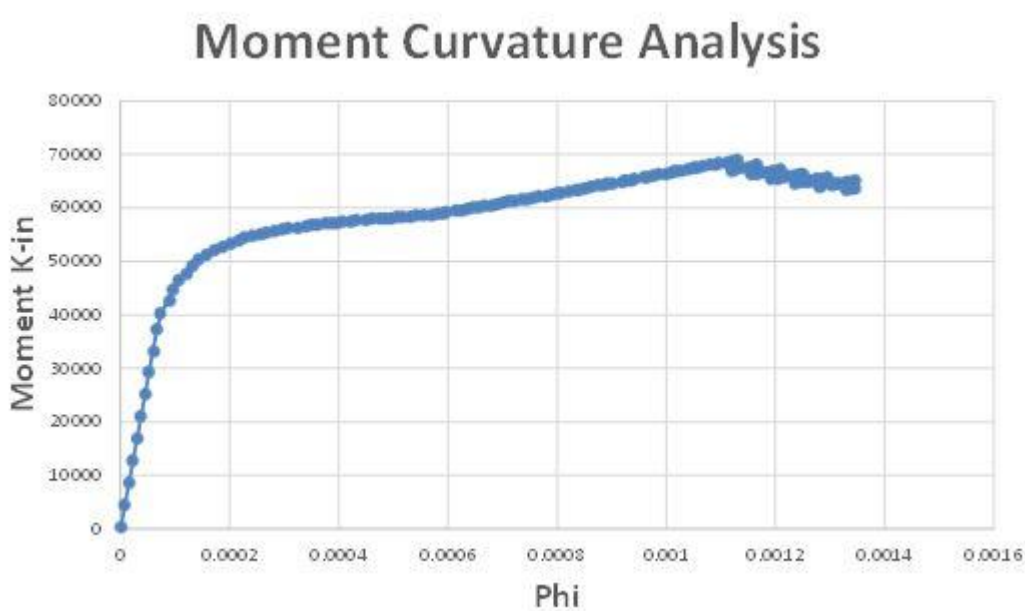


Figure 22. Moment Curvature Analysis

CONCLUSIONS

The fatigue load test and ultimate test was conducted on a 40 feet span folded plate girder with cast in situ concrete deck. The girder was spliced with a full penetration weld at mid span in order to increase the span length of folded plate girders to over 100 feet.

The girder performed very well under the fatigue load in which five million cycles were applied. The fatigue load and number of cycles were applied simulating AASHTO truck passage over the bridge in 75 years of its design life. It was observed that there was no significant loss of stiffness during the fatigue load testing. The bridge was inspected after completion of every half million cycles and no damage or cracking was observed during inspection.

The bridge specimen was loaded until failure to determine the ultimate load of the specimen and to observe the failure pattern of the bridge. It was observed that the welded connection remained intact during ultimate test and the specimen failed due to crushing of concrete at points of load application in the deck. The cracked specimen is shown below in Figure 23. The ultimate load which was applied on the specimen was 893 Kips, the predicted ultimate load was about 821 Kips so the specimen behaved slightly better than forecasted.

Figure 23. Crushed Specimen after Ultimate Test