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16. Abstract

Under this project, methodologies were developed to extend the maximum length of the FSPGBS to about 100 ft. Work consisted of envisioning the connection detail and carrying out numerous non-linear finite element analysis and developing the proof of concept test, that includes testing two FSGBS with types of connections envisioned. Under this project plans for testing two test specimens were developed and tests specimens have been fabricated. The next phase of the project, not reported in this project will consist of testing the two test specimen and verifying the assumptions made and have the system ready for field applications.

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Extending Maximum Length of the Folded Steel Plate Girder Bridge System (FSPGBS), exceeding 100 ft. with capability to Incorporate Camber

Final Report

March 6, 2019

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Final Report

Extending Maximum Length of the Folded Steel Plate Girder Bridge System (FSPGBS), exceeding 100 ft. with capability to Incorporate Camber

By Atorod Azizinamini, PhD, P.E.

A. ABSTRACT

Folded Steel Plate Girder Bridge System (FSPGBS), is an economical steel bridge solution for short span bridges. To date several states have used the system that includes Nebraska, Massachusetts, Montana, Pennsylvania and Michigan. Between 2016 and 2018, State of Pennsylvania through its major bridge replacement program used seven FSPGBS. These bridges were used in competitive basis and Walsh the contractor, selected FSPGBS over concrete alternates, based on economy.

FSPGBS, is a proprietary bridge system and provides many advantages. It eliminates need for cross frames, it is easy to inspect, it is best suited for Accelerated Bridge Construction (ABC) application and recently has proven to be very competitive with other materials as demonstrated by a contractor purchasing several of them for major bridge construction initiative in Pennsylvania.

Currently the maximum length of FSPGBS is limited to 60 ft., reflecting the maximum length of press brake available nationally. Further, the system does not allow having camber. Not having camber is not an issue for bridges with lengths, less than 60 ft.

Under this project, methodologies were developed to extend the maximum length of the FSPGBS to about 100 ft. Work consisted of envisioning the connection detail and carrying out numerous non-linear finite element analysis and developing the proof of concept test, that includes testing two FSGBS with types of connections envisioned. Under this project plans for testing two test specimens were developed and tests specimens have been fabricated. The next phase of the project, not reported in this project will consist of testing the two test specimen and verifying the assumptions made and have the system ready for field applications.

B. Background and Objectives

Majority of substandard bridges needing replacement are under 100 ft. long and are simple spans. FSPGBS provide an economical short span steel bridge alternative for bridges less than 60 ft. long. The feedback from contractors and consultant is that developing modified version of FSPGBS, with maximum span lengths approaching 100 ft. will significantly help the bridge industry. In fact inspection of U.S. bridge inventory indicates that majority of substandard bridges are less than 100 ft. long and are simple spans.

The main objective of this research is to develop a new version of FSPGBS with maximum span length, exceeding 100 ft. with allowance to incorporate camber.

C. Overview of FSPGBS

This section provides overview of FSPGBS, as applied to maximum 60 ft. length. Folded Steel Plate Girder Bridge System (FSPGBS), offers an economical solution for many of the nation's bridges in need of replacement or new bridges. The current span lengths is limited to 60 ft. The system consists of a series of standard shapes that are built by bending flat plates into inverted tub sections using a press break. **Figure 1** shows a fabrication process for a typical folded plate girder.

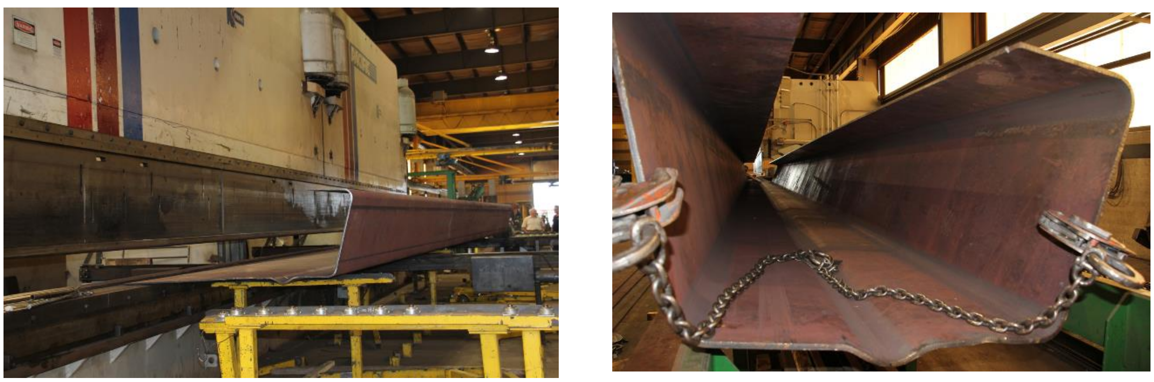


Figure 1. Fabrication of folded plate girder using a press break machine.

FSPGBS have many advantages for both steel fabricators and bridge owners. Folded plate girders suitable for different span lengths differ only by their cross-sectional dimensions. **Figure 2** shows a cross section for a typical folded plate girder.

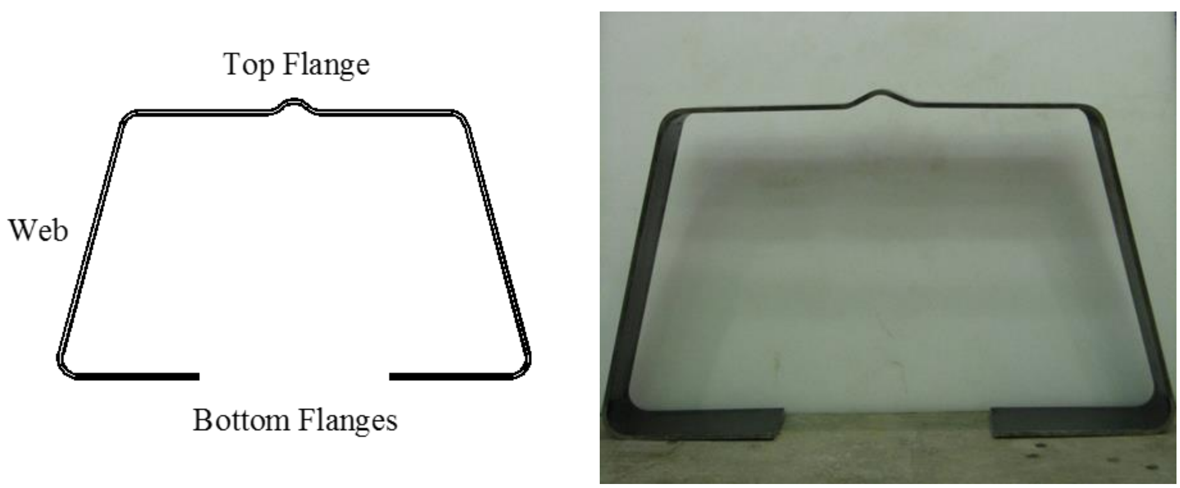


Figure 2. Typical Cross Section for Folded Plate Girder.

More specifically, varying the width of the top and bottom flanges and the depth of the web while keeping the plate thicknesses to either 3/8 or 1/2 inches can accommodate span length requirements of up to 60 ft in length. The different top and bottom flange widths and web depth can easily be accommodated by changing the bend locations so fabricators can build folded girders very quickly while only stocking two plate thicknesses (1/2 and 3/8 inches). That is important because delivery in a timely manner is an important issue for the bridge owners. The maximum span length for this system is currently limited to about 60 ft, reflecting the longest press breaks that are available in the industry.

C.1. Advantages of Folded Plate Girder Bridge System

The shape of the cross section for the FSPGBS has several key advantages in its design and construction. Following are brief descriptions of some of the advantages.

The inverted tub shape produces a very stable bridge girder configuration that does not require internal or external cross frames for either local or global stability. A typical box section needs top lateral bracing, during construction and during replacement of deck. The FSPGBS does not internal or external bracing, which has proven to cause many fatigue cracking challenges. **Error! Reference source not found.** shows a 46 ft. long folded plate girder.

Figure 3. 46 ft. Long Folded Plate Girder.

Figure 4. Conventional formwork for casting concrete deck.

Casting the deck on top of folded plate girder could use conventional construction equipment and practices, if needed. The top flange of the folded plate girder is wide enough (about 25 in. to 35 in.) to serve as a work platform. That alone can reduce many construction hazards associated with workers walking on girders during construction. **Figure 4**, shows conventional formwork that can be used to prepare for casting the concrete deck. Because of the torsional stiffness of the folded plate girder, there is no need for providing internal or external bracing during construction.

Perhaps the major advantage of FSPGBS over tub or box bridges, is the opening from the bottom side that allows easy inspection of the girder and prevents accumulation of moisture. Experience with closed utility poles (even galvanized) and closed box sections indicates that over time debris and moisture find ways to penetrate inside the box and accumulate and can result in significant reduction in service life of the bridge as a system. For longer span bridges the depth of a closed box is large enough to enter inside and inspect and clean if needed. Folded plate girders provide the same characteristics as that of a closed box with the advantage of being able to inspect the inside. The opening on the bottom side of the folded plate also allows passing the utility lines, if needed. **Figure 5** shows the bottom view of a 46 ft. long folded plate. Several alternatives are available to prevent bird nesting inside the box, if desired.

Hot Dip Galvanizing the FSPGBS is a very good option for corrosion protection. Hot dip galvanizing can provides more than 75 years of service life at a very economical cost. Typical cost of hot dip galvanizing ranges between 18 to 22 cents per pound of steel.

C.1.a. Folded Plate Girder Bridge System in Modular Form

The best use of FSPGBS is to utilize it in conjunction with ABC. **Figure 6** shows a pre-topped folded plate girder unit ready for shipping to job side.

Figure 5. Bottom view of 46 ft. Long folded plate girder.

C.2. Brief Summary of Past Research Work Leading to Development of Folded Plate Girder Bridge System

An extensive amount of experimental, numerical and analytical work was performed to comprehend performance of FSPGBS and develop design aids. This section provides brief summary of experimental, numerical and analytical studies carried out.

Figure 6. Pre-top folded plate girder.

C.2.a. Experimental Tests

Experimental testing consisted of conducting 9 tests using 6 test specimens. **Figure 7** shows the generic shape of a folded plate specimen and **Table 1** gives the dimensions of the different specimens that were used in testing. Note that the Trap Width and Trap Height dimensions refer to an idealized trapezoid along the plate midline without corner radii.

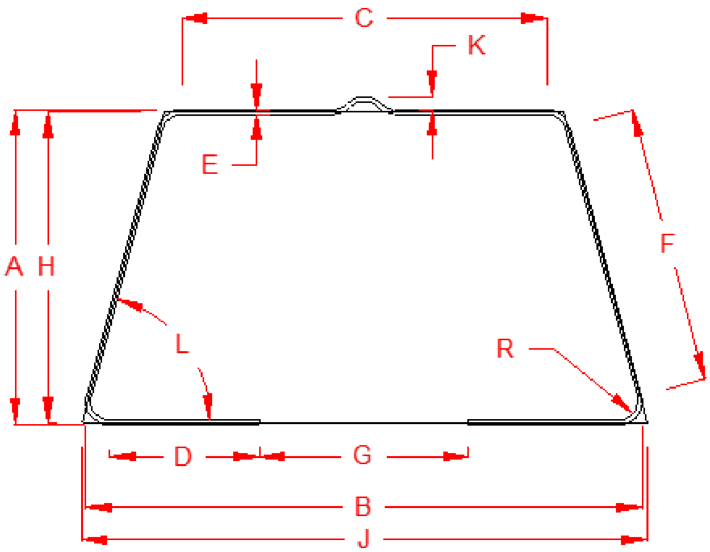


Figure 7. Generic Test Specimen Cross Section.

Table 1. Specimen Geometry.

		Height	Width	Top Flange	Bottom Flange	Thickness	Side Length	Opening	Trap Height	Trap width	Ridge Height	Angle	Bend Radius	Yield Stress
	Units	in	in	in	in	in	in	in	in	in	in	degree	in	ksi
	Label	A	B	C	D	E	F	G	H	J	K	L	R	
Specimen	A	24.75	45.47	30	10	0.375	20.7	20.72	24.38	46.42	0*	75	2	65
	B	24.75	45.47	30	10	0.375	20.7	20.72	24.38	46.42	0*	75	2	65
	C	24.75	45.47	30	10	0.375	20.7	20.72	24.38	46.42	0*	75	2	65
	D	24.88	43.85	28.78	11.8	0.375	21.87	16.50	24.50	44.50	1.0	75	1.5	50
	E	24.88	43.85	28.78	11.8	0.375	21.87	16.50	24.50	44.50	1.0	75	1.5	50
	F	25.0	43.64	27.92	11.1	0.5	20.71	16.5	24.50	44.50	1.0	75	2.0	50

*No ridge in top flange

Table 2. Summary of Testing Program.

Test ID	Specimen	Length*	Type	Stiffener @ load point	Deck	Comments
A1	A	41’	Constructability	No	No	
B1	B	41’	Constructability	Yes	No	
C1	C	41’	Cyclic	No	Yes	
C2	C	41’	Ultimate	No	Yes	
D1	D	46’	Constructability	Yes	No	
E1	E	46’	Ultimate	No	Yes	Galv.
E2	E	22’	Shear	No	Yes	Galv.
E3	E	22’	Shear	No	No	Galv.
E4	E	22’	Shear	Yes	No	Galv.
F	F	46’	Modular Deck	No	Yes	

*Length specifies the span length from centerline of support to centerline of support

As indicated from **Table 2**, tests carried out included testing folded plate girders without any deck (constructability test) and cyclic, shear and ultimate load tests on folded plate girders with deck on the top.

The initial tests were carried out on test specimens without a ridge on the top (k dimension equal to zero). Constructability tests indicated that folded plate girders with flat top flanges demonstrated very pre-mature buckling of top flange and prone to development of corrosion. As a result the ridge was introduced in the top flange to increase compression capacity of the top plate.

The cyclic test (Test C1) was conducted to comprehend the performance of folded plate girder with pre-topped deck, under repeated traffic loads. The test setup is shown in **Figure 8.** The test specimen was subjected to total of about 7.5 million load cycles, as indicated in

. The load level and number of cycles to be applied were determined, following the formulation used for fatigue design of steel bridge detailed, defined in chapter six of AASHTO LRFD Bridge Design Specification.

Figure 8. Composite Test Setup.

Table 3. Cyclic Load Summary.

Load Stage	Cycle Numbers	Load (kips)	Cycle Rate (Hz)
1	0 to 302,797	60	1.4
2	3,027,989 to 5,115,287	60	1.2
3	5,115,287 to 7,179,071	72	1.0

Error! Reference source not found. provides the stiffness of the systems at various cycle numbers. The stiffness of the system was calculated by dividing the applied load by the deflection. As seen, there was very little change in stiffness throughout the cyclic testing, which indicates there was no progressive softening or failure of the specimen.

Table 4. System Stiffness.

Initial	Cycle #1,794,770	Cycle #3,589,540	Cycle #7,179,071
116.1571kips/in	117.0952kips/in	114.1837kips/in	114.0183kips/in

C.2.b. Ultimate Load Test

Specimen E was a galvanized specimen with a stiffening ridge along the top flange as shown in **Figure 9.**

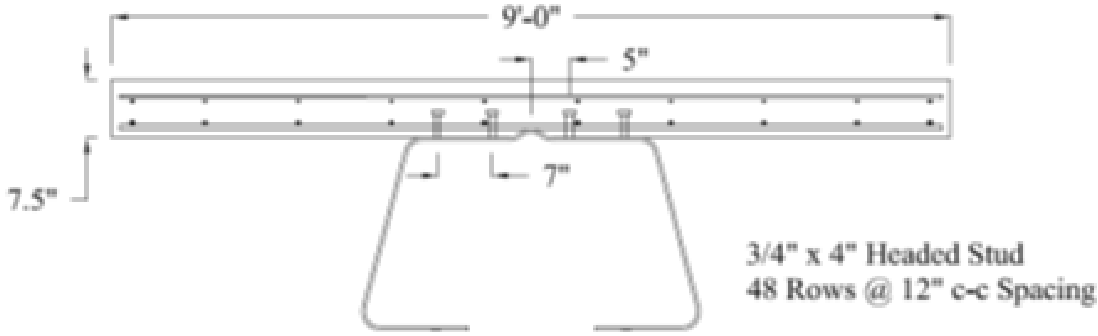


Figure 9. Specimen E – Cross Section.

Figure 10 shows the basic load deflection plot obtained from the test. The load is the total load applied to the girder and the deflection data is obtained from mid-span.

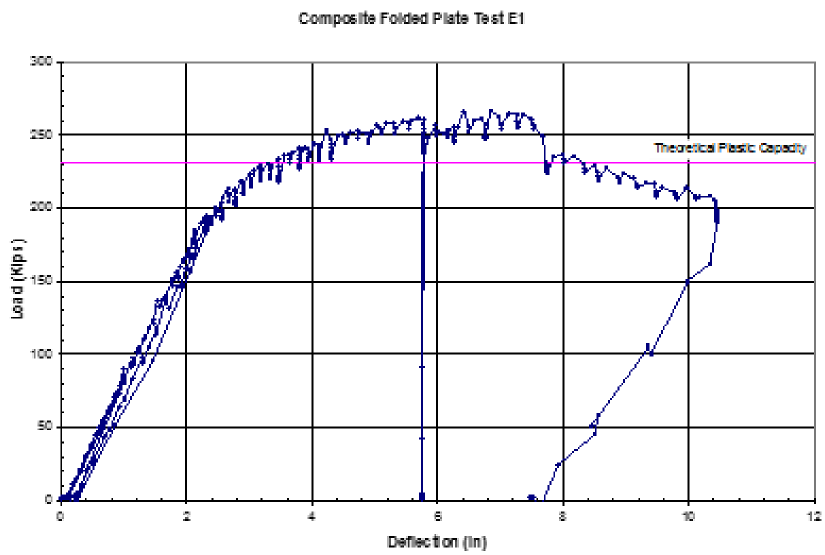


Figure 10. Test E1 – Load versus Displacement.

Also shown in **Figure 11**, is the theoretical plastic moment capacity of the cross section for the test specimen using measured material properties. As indicated from **Figure 10**, specimen exhibited significant amount of displacement ductility before failing.

C.2.c. Shear Testing

Ultimate load testing of test specimen E, consisted of applying a concentrated load at the mid-span of the bridge. The failure of the test specimen E was in the form of crushing of the concrete at mid-span.

Once ultimate load testing of test specimen E was complete, the specimen was cut in half near mid-span where the failure had occurred. One half of the test specimen E was used to carry out shear test.

Figure 11 shows the test set up for shear test E2. **Figure 12** shows the resulting applied load versus the deflection at the point of load application. Specimen was able to carry very high load (600 kips) before failure.

Figure 11. Test E2 – Load Configuration.

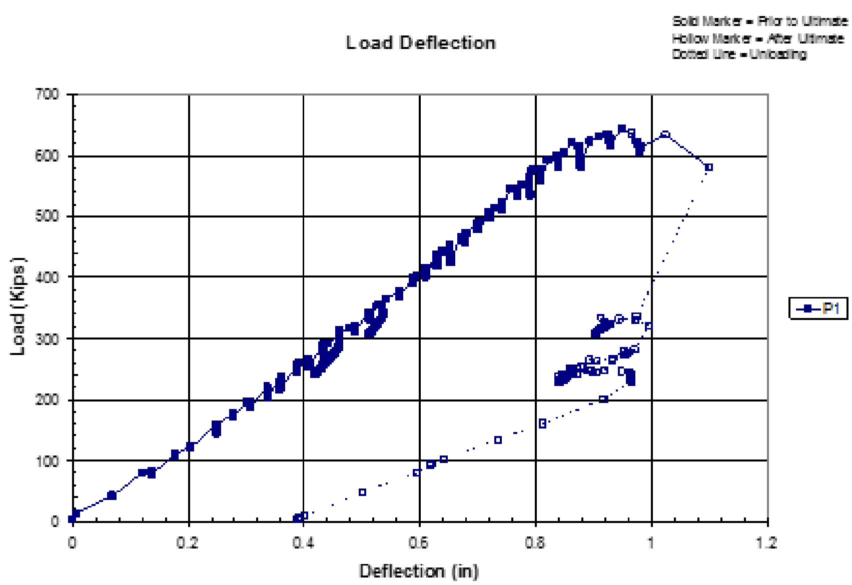


Figure 12. Test E2 – Load Deflection.

C.2.d. Development of Design Aids

Significant amount of work was carried out to develop design aid for use of FSPGBS in practice. This included development of standard sections, customized distribution factors and design sheet for each standard cross section.

Error! Reference source not found. shows the standard shapes that were developed. The name specifies the width (W) and height (H) of the defining trapezoid as well as the clear opening (O) between the bottom flanges. The weight (Wt), moment of inertia about the strong bending axis (Ixx), and location of neutral axis relative to the bottom of the bottom flange (NA) is given for plate thicknesses (t) of 3/8-inch and 1/2-inch.

Table 5. Standard Shapes.

Name	t (in)	Wt (plf)	Ixx (in^4)	NA (in)
W36H16O16	3/8	99	1260	9.2
	1/2	133	1485	9.3
W36H18O16	3/8	103	1615	10.2
	1/2	138	1895	10.3
W36H20O16	3/8	107	2015	11.1
	1/2	143	2360	11.2
W40H20O16	3/8	117	2315	11.1
	1/2	157	2710	11.1
W40H24O16	3/8	125	3390	12.9
	1/2	167	3945	12.9
W44H24O16	3/8	135	3825	12.9
	1/2	181	4450	12.9
W40H28O16	3/8	133	4680	14.7
	1/2	178	5420	14.7
W40H32O16	3/8	141	6180	16.4
	1/2	188	7135	16.4
W44H32O18	3/8	149	6750	16.7
	1/2	194	8080	16.7
W40H34O16	3/8	145	7015	17.2
	1/2	199	7785	17.3
W44H34O20	3/8	150	7430	17.8
	1/2	200	8545	17.9

In order to use conventional design procedures, the amount of load distributed to each girder in a multi-girder system must be evaluated. Simplified distribution relationships exist for many common structure types, some of which could be argued are applicable to the folded plate girder system. A study was carried out to establish a more exact distribution factor for the folded plate girder system. A power fit method, similar to that used in the AASHTO LRFD Specifications approximate analysis table, was applied to the results. The resulting equations are given in **Table 6**.

An additional suite of analyses was performed investigating skew and found that the equations given by the current AASHTO LRFD Specifications to account for skew – both for flexure and shear – yields conservative results under all conditions.

Table 6. Distribution Factor Equations.

		Interior	Exterior
Flexure	Single	$0.25+\left(\frac{S}{10}\right)^{0.6}\left(\frac{S}{L}\right)^{0.7}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.35}$	Lever Rule
	Multiple	$g_{SL}+\frac{S}{36}-\frac{40}{L^2}$	$g_{ML}=1.15g_{SL}-\frac{1}{6}\left(\frac{K_g}{12.0Lt_s^3}\right)$
	Skew Adjustment Factor	$1-0.25\left(\frac{K_g}{12.0\,L\,t_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}(\tan\theta)^{1.5}$	
Shear	Single	$0.25+\left(\frac{S}{12}\right)^{0.8}\left(\frac{S}{L}\right)^{0.4}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.35}$	Lever Rule
	Multiple	$g_{SL}+\frac{S}{45}-\frac{25}{L^2}$	$g_{ML}=g_{SL}+0.07$
	Skew Adjustment Factor	$1+0.2\left(\frac{12.0\,L\,t_s^3}{K_g}\right)^{0.3}\tan\theta$	

Where: $K_g=n\left(I+Ae_g^2\right)$ with $n=\frac{E_B}{E_D}$

Variables and Units are consistent with the approximate distribution factor tables contained in the AASHTO LRFD Bridge Design Specifications.

S = Girder Spacing (ft)
L = Span Length (ft)
ts = Slab Thickness (in)
de = distance from the exterior web of exterior beam to the interior edge of curb or traffic barrier (ft)

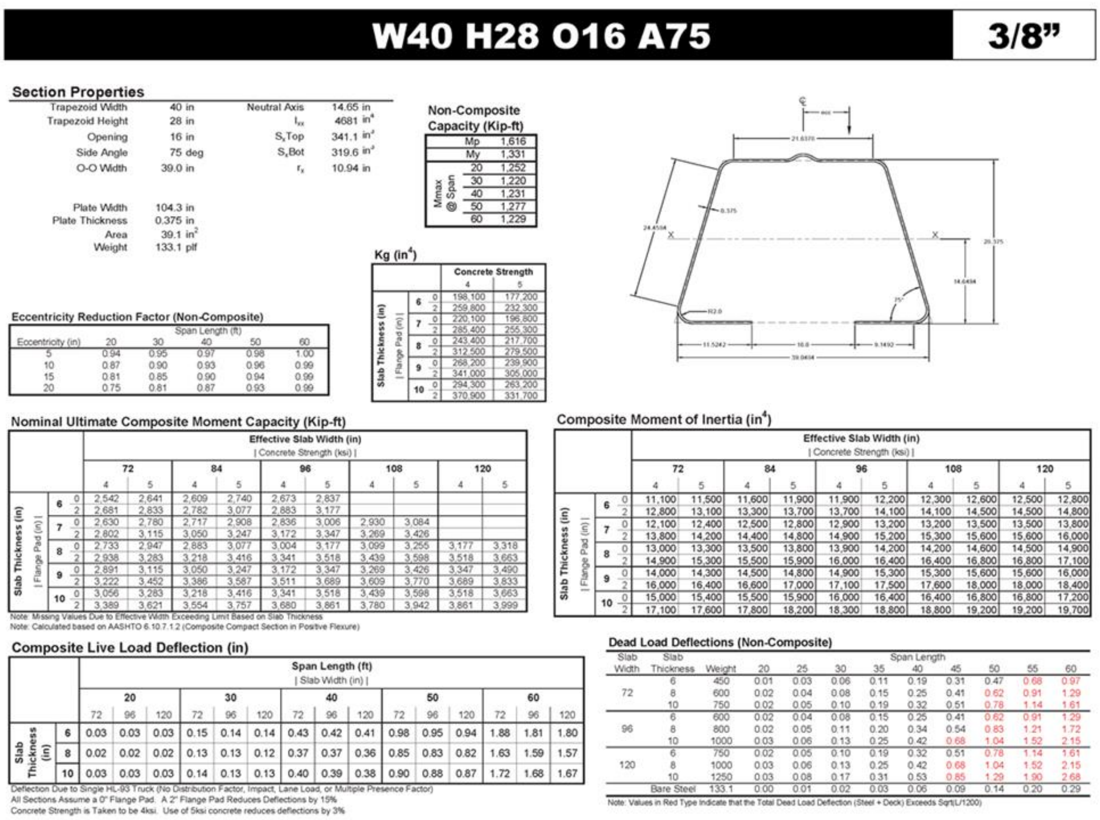
Finite element analysis was also used to evaluate the transverse bending moment in the deck slab. The primary recommendation is that the slab be designed utilizing the empirical design method as specified in AASHTO LRFD 9.7.2. In lieu of empirical design, the traditional method specified in AASHTO LRFD 9.7.3 may be used. The loading values may be obtained from AASHTO LRFD Table A4-1. For the case of negative bending, the girder spacing value contained in the table shall be taken as the distance between the webs of adjacent girders, which may be calculated based on the dimensions of the defining trapezoid. For the case of positive bending, the girder spacing value contained in the table shall be taken as the girder centerline spacing. Use of the centerline spacing accounts for the torsional flexibility of the girder since an individual web near mid-span will not provide vertical restraint in the same way that it does near a support.

C.2.e. Design Data Sheets

For each of the 11 standard sections, a design data sheet was prepared that could assist the designer to select the desired section and contains the following information:

- A dimensioned drawing of the section
- Sectional Properties such as Area, Moment of Inertia, etc.
- Non-Composite (Dead Load) Capacity
- Nominal Ultimate Composite Moment Capacity
- Composite Moment of Inertia
- Composite Live Load Deflection
- Non-Composite Dead Load Deflections
- Kg
- Eccentricity Reduction Factor

A sample data sheet is shown in **Figure 13** and further details are provided in the sections to follow.



W40 H28 O16 A75 **3/8"**

Section Properties

Trapezoid Width	40 in	Neutral Axis	14.65 in
Trapezoid Height	28 in	I_{xx}	4681 in^4
Opening	16 in	S_xTop	341.1 in^3
Side Angle	75 deg	S_xBot	319.6 in^3
O-O Width	39.0 in	r_x	10.94 in
Plate Width	104.3 in		
Plate Thickness	0.375 in		
Area	39.1 in^2		
Weight	133.1 plf		

Non-Composite Capacity (Kip-ft)

Mp		1,616
My		1,331
Mmax @ Span	20	1,252
	30	1,220
	40	1,231
	50	1,277
	60	1,229

Kg (in^4)

Slab Thickness (in)	Flange Pad (in)	Concrete Strength 4	Concrete Strength 5
6	0	198,100	177,200
6	2	259,800	232,300
7	0	220,100	196,800
7	2	285,400	255,300
8	0	243,400	217,700
8	2	312,500	279,500
9	0	268,200	239,900
9	2	341,000	305,000
10	0	294,300	263,200
10	2	370,900	331,700

Eccentricity Reduction Factor (Non-Composite)

Eccentricity (in)	Span Length (ft) 20	30	40	50	60
5	0.94	0.95	0.97	0.98	1.00
10	0.87	0.90	0.93	0.96	0.99
15	0.81	0.85	0.90	0.94	0.99
20	0.75	0.81	0.87	0.93	0.99

Nominal Ultimate Composite Moment Capacity (Kip-ft)

Slab Thickness (in)	Flange Pad (in)	72 / 4	72 / 5	84 / 4	84 / 5	96 / 4	96 / 5	108 / 4	108 / 5	120 / 4	120 / 5
6	0	2,542	2,641	2,609	2,740	2,673	2,837				
6	2	2,681	2,833	2,782	3,077	2,883	3,177				
7	0	2,630	2,780	2,717	2,908	2,836	3,006	2,930	3,084		
7	2	2,802	3,115	3,050	3,247	3,172	3,347	3,269	3,426		
8	0	2,733	2,947	2,883	3,077	3,004	3,177	3,099	3,255	3,177	3,318
8	2	2,938	3,283	3,218	3,416	3,341	3,518	3,439	3,598	3,518	3,663
9	0	2,891	3,115	3,050	3,247	3,172	3,347	3,269	3,426	3,347	3,490
9	2	3,222	3,452	3,386	3,587	3,511	3,689	3,609	3,770	3,689	3,833
10	0	3,056	3,283	3,218	3,416	3,341	3,518	3,439	3,598	3,518	3,663
10	2	3,389	3,621	3,554	3,757	3,680	3,861	3,780	3,942	3,861	3,999

Effective Slab Width (in) | Concrete Strength (ksi) |

Note: Missing Values Due to Effective Width Exceeding Limit Based on Slab Thickness
Note: Calculated based on AASHTO 6.10.7.1.2 (Composite Compact Section in Positive Flexure)

Composite Moment of Inertia (in^4)

Slab Thickness (in)	Flange Pad (in)	72 / 4	72 / 5	84 / 4	84 / 5	96 / 4	96 / 5	108 / 4	108 / 5	120 / 4	120 / 5
6	0	11,100	11,500	11,600	11,900	11,900	12,200	12,300	12,600	12,500	12,800
6	2	12,800	13,100	13,300	13,700	13,700	14,100	14,100	14,500	14,500	14,800
7	0	12,100	12,400	12,500	12,800	12,900	13,200	13,200	13,500	13,500	13,800
7	2	13,800	14,200	14,400	14,800	14,900	15,200	15,300	15,600	15,600	16,000
8	0	13,000	13,300	13,500	13,800	13,900	14,200	14,200	14,600	14,500	14,900
8	2	14,900	15,300	15,500	15,900	16,000	16,400	16,400	16,800	16,800	17,100
9	0	14,000	14,300	14,500	14,800	14,900	15,300	15,300	15,600	15,600	16,000
9	2	16,000	16,400	16,600	17,000	17,100	17,500	17,600	18,000	18,000	18,400
10	0	15,000	15,400	15,500	15,900	16,000	16,400	16,400	16,800	16,800	17,200
10	2	17,100	17,600	17,800	18,200	18,300	18,800	18,800	19,200	19,200	19,700

Effective Slab Width (in) | Concrete Strength (ksi) |

Composite Live Load Deflection (in)

Slab Thickness (in)	20 / 72	20 / 96	20 / 120	30 / 72	30 / 96	30 / 120	40 / 72	40 / 96	40 / 120	50 / 72	50 / 96	50 / 120	60 / 72	60 / 96	60 / 120
6	0.03	0.03	0.03	0.15	0.14	0.14	0.43	0.42	0.41	0.98	0.95	0.94	1.88	1.81	1.80
8	0.02	0.02	0.02	0.13	0.13	0.12	0.37	0.37	0.36	0.85	0.83	0.82	1.63	1.59	1.57
10	0.03	0.03	0.03	0.14	0.13	0.13	0.40	0.39	0.38	0.90	0.88	0.87	1.72	1.68	1.67

Span Length (ft) | Slab Width (in) |

Deflection Due to Single HL-93 Truck (No Distribution Factor, Impact, Lane Load, or Multiple Presence Factor)
All Sections Assume a 0" Flange Pad. A 2" Flange Pad Reduces Deflections by 15%
Concrete Strength is Taken to be 4ksi. Use of 5ksi concrete reduces deflections by 3%

Dead Load Deflections (Non-Composite)

Slab Width	Slab Thickness	Weight	Span Length 20	25	30	35	40	45	50	55	60
72	6	450	0.01	0.03	0.06	0.11	0.19	0.31	0.47	0.68	0.97
	8	600	0.02	0.04	0.08	0.15	0.25	0.41	0.62	0.91	1.29
	10	750	0.02	0.05	0.10	0.19	0.32	0.51	0.78	1.14	1.61
96	6	600	0.02	0.04	0.08	0.15	0.25	0.41	0.62	0.91	1.29
	8	800	0.02	0.05	0.11	0.20	0.34	0.54	0.83	1.21	1.72
	10	1000	0.03	0.06	0.13	0.25	0.42	0.68	1.04	1.52	2.15
120	6	750	0.02	0.05	0.10	0.19	0.32	0.51	0.78	1.14	1.61
	8	1000	0.03	0.06	0.13	0.25	0.42	0.68	1.04	1.52	2.15
	10	1250	0.03	0.08	0.17	0.31	0.53	0.85	1.29	1.90	2.68
	Bare Steel	133.1	0.00	0.01	0.02	0.03	0.06	0.09	0.14	0.20	0.29

Note: Values in Red Type Indicate that the Total Dead Load Deflection (Steel + Deck) Exceeds Sqrt(L/1200)

Figure 13. Sample Design sheet to assist in selecting the optimized Folded Girder.

The specified section parameters such as trapezoid width and height are repeated for reference. The total plate width is the un-folded width of the plate used to form the section. This value includes 0.5 inches to account for the ridge along the top flange. The listed area and weight of the section is obtained from this total plate width. The calculated section properties such as Ixx, Sx, rx, and Kg ignore any stiffening ridge along the top flange.

The predicted plastic and yield moment capacities of the non-composite section are calculated assuming a yield strength of 50 ksi. These values do not include any stiffening ridge on the top flange. The maximum moment values listed for different span lengths are the moments correspond to the maximum values obtained from the finite element analyses. Note that all of these values are unfactored

The ultimate moment capacity values are calculated based on AASHTO LRFD 6.10.7.1.2, which governs composite compact sections in positive flexure. The base strength is the plastic moment capacity of the section, which is then reduced based on the relative location of the neutral axis with respect to the total depth of the section. Sections where the neutral axis is located high in the section behave more ductile and are allowed to utilize a greater portion of their plastic moment capacity. The plastic moment capacity of the section was obtained using an idealized trapezoid shape (ignoring the radius in the corners). The plastic neutral axis was found considering a balance of forces. The concrete stress in the compressive region was taken to be 0.85f'c. The flange pad (haunch) thickness is the distance from the top of the top flange (ignoring any ridge) to the bottom of the deck. The effective width of the slab is limited to six times the slab thickness beyond the edge of each web (i.e. beff<=TopWidth + 12ts). Combinations that violate this requirement are left blank.

C.3. Example FSPGBS in Service

To date several FSPGBS has been constructed and are in service. The first FSPGBS is a 46 foot stream crossing in Uxbridge, Massachusetts. Error! Reference source not found.**14** shows the FSPGBS in Massachusetts, during construction and after opening to traffic on November 2011. The Massachusetts FSPGBS used integral abutment detail.

The second FSPGBS was constructed in Nebraska and was opened to traffic in 2014. **Figure 15** and **Figure 16** shows Nebraska FSPGBS during construction.

Nebraska FSPGBS used a simple end and full depth deck. Further the Nebraska FSPGBS used Ultra High Performance Concrete (UHPC) for closure pour joints, whereas Massachusetts FSPGBS used a headed bar and normal strength concrete for closure joints.

Figure 14. FSPGBS in Uxbridge, Massachusetts

Figure 15. Construction of Nebraska FSPGBS

Figure 16. Construction of Nebraska FSPGBS

The First of seven FSPGBS bridges constructed in Pennsylvania used a full depth deck, integral abutment, 180 hooked detail in conjunction with normal strength concrete in closure joints and fully precast rails as shown in **Figure 17**.

Figure 17. First FSPGBS constructed in Pennsylvania

Additional FSPGBS are constructed in other States and its use is expected to increase when the maximum length is extended to beyond 100 ft.

Development of FSPGBS in mid 2000s by modern steel magazine was judged to be one of the most important developments within last 50 years in steel industry. Introduction of any new idea in the bridge engineering is time consuming. Recent selection of FSPGBS by contractor solely based on economy is a proof that FSGBS is fast becoming a viable alternative for many of the nation's substandard bridges in need of replacement. The cost of FSPGBS is also getting lower, which further adds to its merit for use in replacement of many substandard bridges we have in our inventory. The feedback from contractors are excellent and they have had an extremely easy time during construction, mainly because of its simplicity, evenly spaces girders, not having to install cross frames in the field and light weight of FSPGBS, that requires very light construction equipment.

Currently, many consultants and contractors have been asking a need to increase maximum length of the FSPGBS to about 100 ft. Having an alternative bridge system that can span more than 100 ft, without needing cross frame, evenly spaced girders, light weight, easy to inspect, best suited for ABC application and no fatigue prone detail in bound to provide a superior ABC bridge system option.

D. Connection Details for Extending Maximum Length

The main approach to extend the maximum length of FSPGBS is in the form of slicing Folded Plate girders with lengths of less than 60 ft. to form longer lengths. To keep the number of connections to minimum, preference is to splice the two piece in the middle.

Another reason to splice the girders in the middle is to incorporate camber by cutting ends of the piece at slight angle and then connect the two piece.

Three main connection types were envisioned and series of non-linear finite element analysis were carried out to comprehend their behavior. Two classes of connection types were investigated- Bolted and welded. Within each category, several variations were considered. Another connection alternative is to direct welding of two pieces using full penetration weld, which did not require any analysis.

For each connection detail categories, numerous non-linear FEA were carried out and only select results are reported here for brevity.

E. Span Length Study

Investigation included conducting a parametric study to develop series of design that could accommodate various span lengths exceeding 60 ft and up to 105 ft. Results of the study is shown in **Table 7**.

Table 7. Results of Span Length Study.

W44H34O16							
Unfolded L = 119 Inches							
	Span	Thickness	Yield	Pos Str	Service	Fatigue	Defl
	90	0.5	50	0.86	1.17	0.81	1.15
	85	0.5	50	0.79	1.08	0.76	1
80	80	0.5	50	0.73	0.99	0.71	0.92
	75	0.5	50				
	65	0.5	50				
95 Alt	95	0.5	70	0.73	0.9	0.86	1.29
90	90	0.5	70	0.67	0.84	0.81	1.15
	85	0.5	70				
	75	0.5	70				
	65	0.5	70				
	105	0.75	50	0.85	1.11	0.7	1.13
95	95	0.75	50	0.74	0.96	0.63	0.93
	85	0.75	50				
	75	0.75	50				
105	105	0.75	70	0.65	0.79	0.7	1.13
	95	0.75	70				
	85	0.75	70				
	75	0.75	70				

Figure below shows the cross sectional geometry used in all analysis listed in Table 7 as well as the details of the prototype bridge considered.

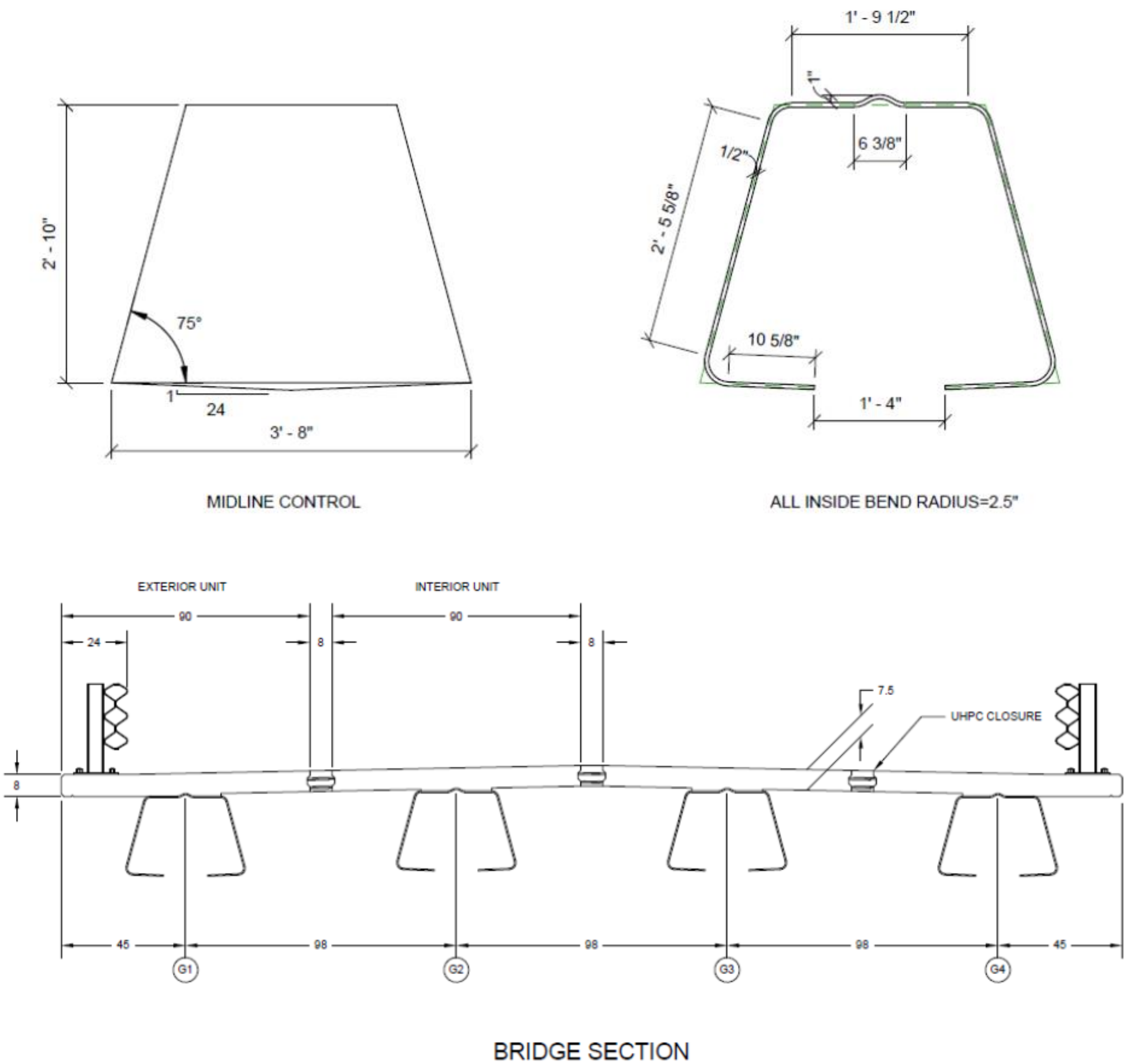


Figure 18. First FSPGBS constructed in Pennsylvania.

Following are additional information related to span parametric study conducted

The distribution factors for flexure are 0.605 and 0.316 for strength and fatigue, respectively. The load values and resulting The values assume shored (pre-topped) construction and a 2" future wearing surface plus a 10% miscellaneous steel factor combinations are given in **Table 8**.

Table 8. Load Combination Flexural Results.

		STR I	STR 2X LL	SRV II	FAT I	FAT II
	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)
DC	513	1.25	1.25	1		
DW	121	1.5	1.5	1		
DF(1.33TRK+LN)	888	1.75	3.5	1.3		
DF(1.15TRK)	237				1.5	0.75
		2377	3931	1788	356	178

As noted in **Table 7**, the main parameter varied in the design parametric study was the plate thickness and plate yield strength. In this table for each span lengths several possibilities are provided. For each option following information are provided.

Ratio of maximum positive moment produced by load at strength limit state I/Maximum moment capacity

Ratio of maximum stress in bottom flange, at service limit state II/Allowable service limit state II

Ratio of maximum fatigue stress in bottom flange/Allowable stress for fatigue

Ratio of maximum deflection/Allowable deflection

Columns with performance indicators below one indicate meeting all AASHTO requirements. As an example for a bridge with 80 ft simple span length, one could use the cross section shown above, ½ inch thick plate with 50 ksi yield strength.

E.1. Summary of Bolted Connection Results

The first attempt was to consider the following detail shown in Figure 19.

In this concept concrete slab was cast onto the bottom flange of the box to aid in the transfer of forces. The splice would occur at the middle of a simply supported span, thus placing the bottom flange in tension and the top in compression.

Shear studs attached to the bottom flange would provide for the connection to each girder, and post-tensioning along with possible mild reinforcement would tie the two together. This connection would provide the tension component of the force transfer. The deck would provide the final compressive component. The girders would be attached to the deck with typical shear studs.

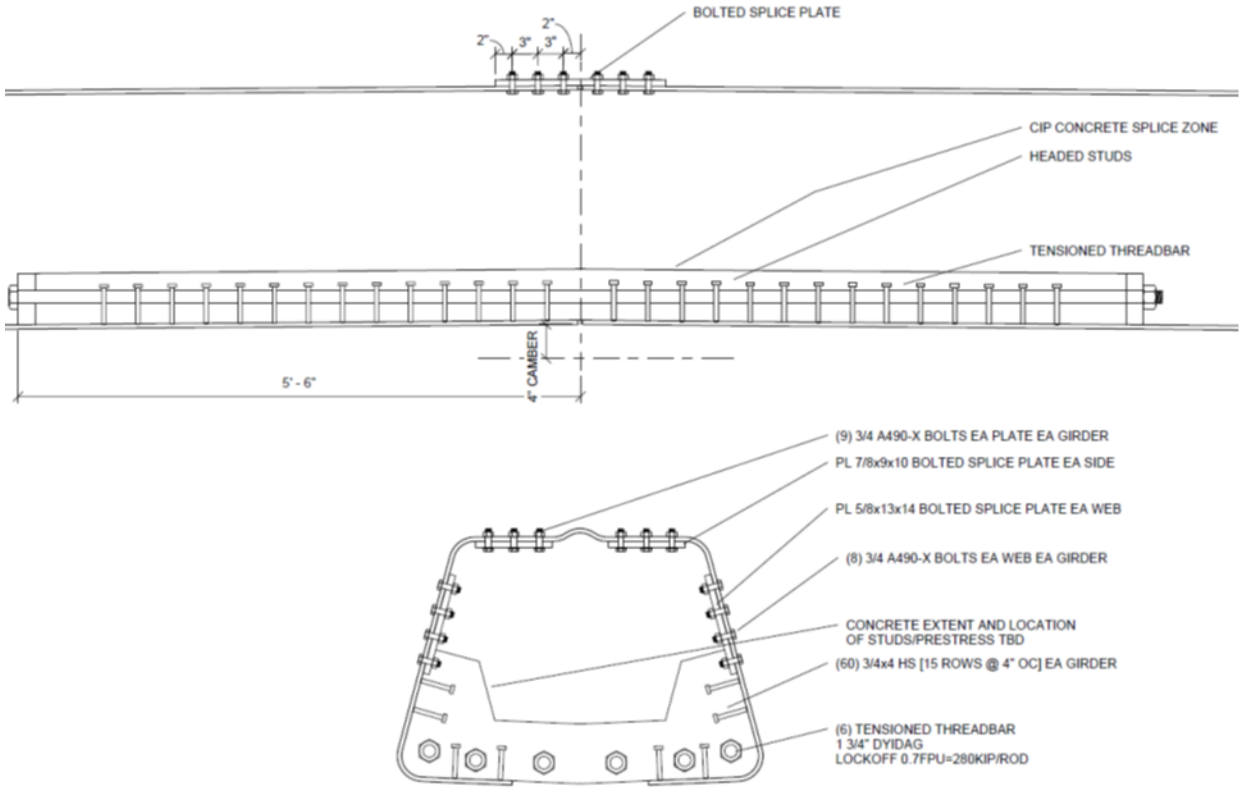


Figure 19. First Attempt for Bolted Connections.

Bolted splice plates at the top flange and web would provide for alignment of the elements and aid in the compressive force transfer. Depending on whether a shored or free spanning construction method was used, this connection may also support compressive loads during casting of the deck.

In an effort to develop a more efficient connection, investigations into a direct bolted alternative began next. A fill plate between the flanges was included to help distribute the load to the bolts near the center of the cross-section. The fill plate fills the gap between the flanges and provides additional length over which to spread the connection.

Initially, it was desired to keep all of the bolts on the exterior of the box to facilitate construction; **Figure 20** (left) shows this arrangement. However, it was soon discovered that the large amount of force to be transferred required that the bolts be clustered at the bottom flange, both above and below the flange. If bolting was going to be required on the inside of the box for strength, then bolting at the webs would be placed to the inside as well for aesthetics, as shown in **Figure 20** (right).

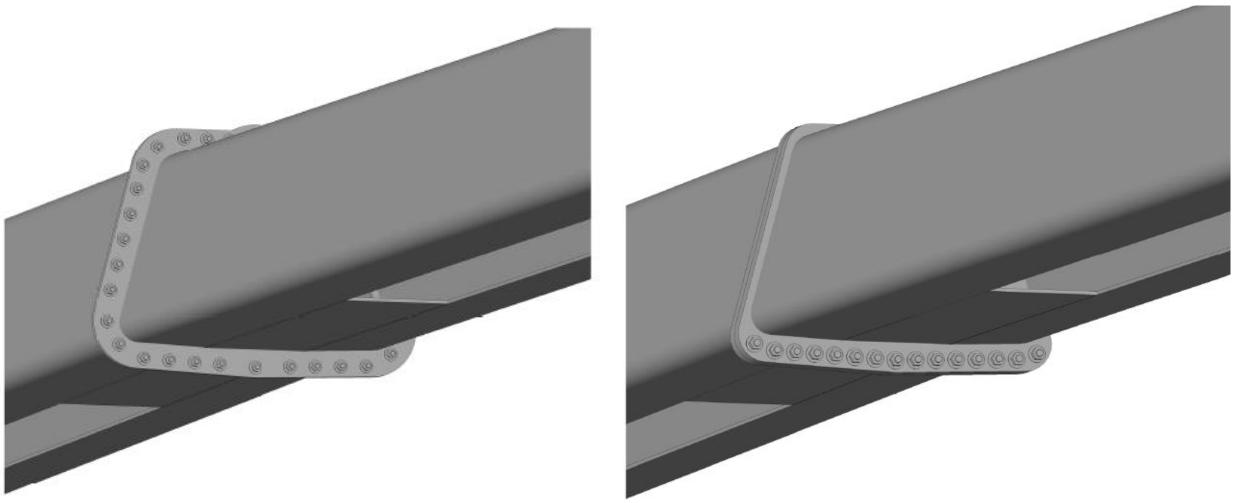


Figure 20. Bolted Splice. Exterior Bolts (left), Interior Bolts- Bottom Clustered (right).

The general-purpose finite element program ANSYS (benchtop interface) was used to analyze various connection details. Element selection and modeling techniques evolved over time.

The first models considered only one-half of a span, with a rigid interface representing the other span. In fact, the initial intent of the analysis was to investigate the demands on the base plate for the test setup. **Figure 21** shows the stress results from one of the early models.

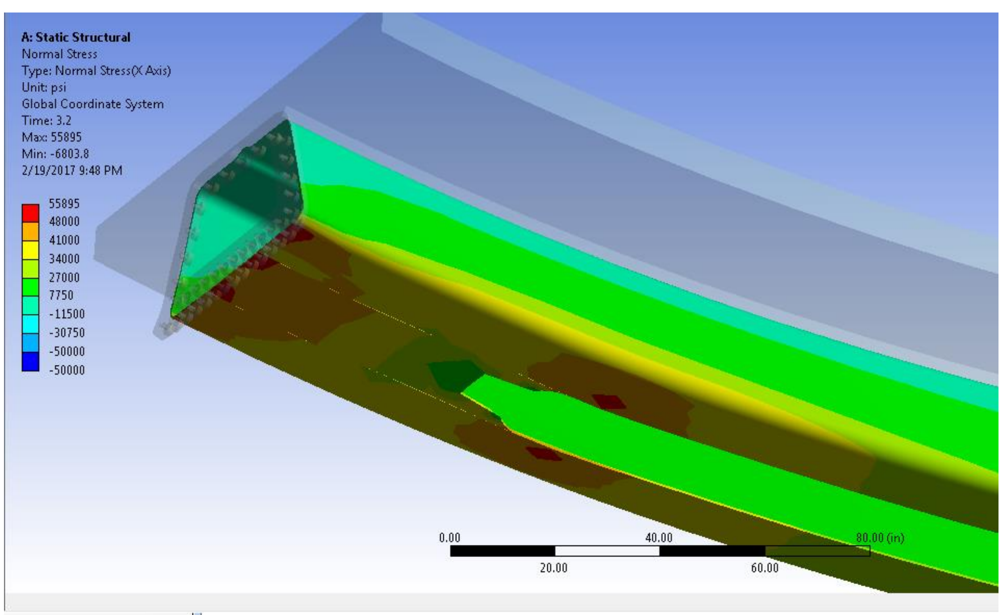


Figure 21. Preliminary Half-Span FEM Results.

The global behavior of the model matched the hand calculated predicted values well. Early on, localized effects were observed and efforts were undertaken to reduce stress concentrations. The model shown above represents an early attempt at smoothing the transition to the fill plate. Note that the hotspot in the bottom flange is several inches away from the actual end of the fill plate. It has been found that the hotspot is due to both a stress concentration effect and a geometric restraint effect. If the box is continuous with nothing between the flanges, the flanges tend to move apart under vertical loading. The filler plate and connection plate restrains this movement, which creates tension on the inside face of the flange at the same location where the stress concentration occurs. This was observed by providing various levels of lateral restraint along the length of the flanges and comparing the results.

Figure 22 shows the final configuration that was chosen for the filler plate. The deep V shape reduces the lateral stiffness, thereby easing the stress riser due to the lateral restraint. From the elastic analysis, the fatigue stress was found to be 4 to 5 ksi. Under an ultimate loading pattern, nonlinearity was first observed at a load level corresponding to 1.5x live load.

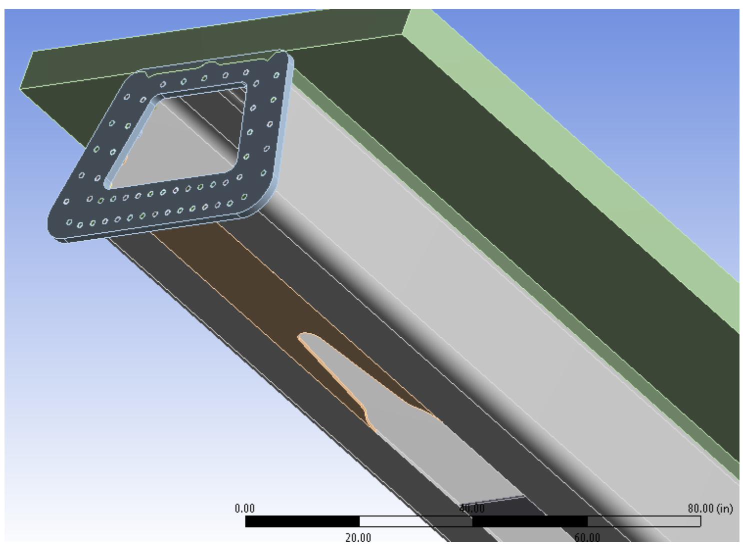


Figure 22. Keyhole Fill Plate.

The moment-deflection curve for the bolted models is shown in **Figure 23**. Load levels corresponding to various conditions are also indicated in the figure.

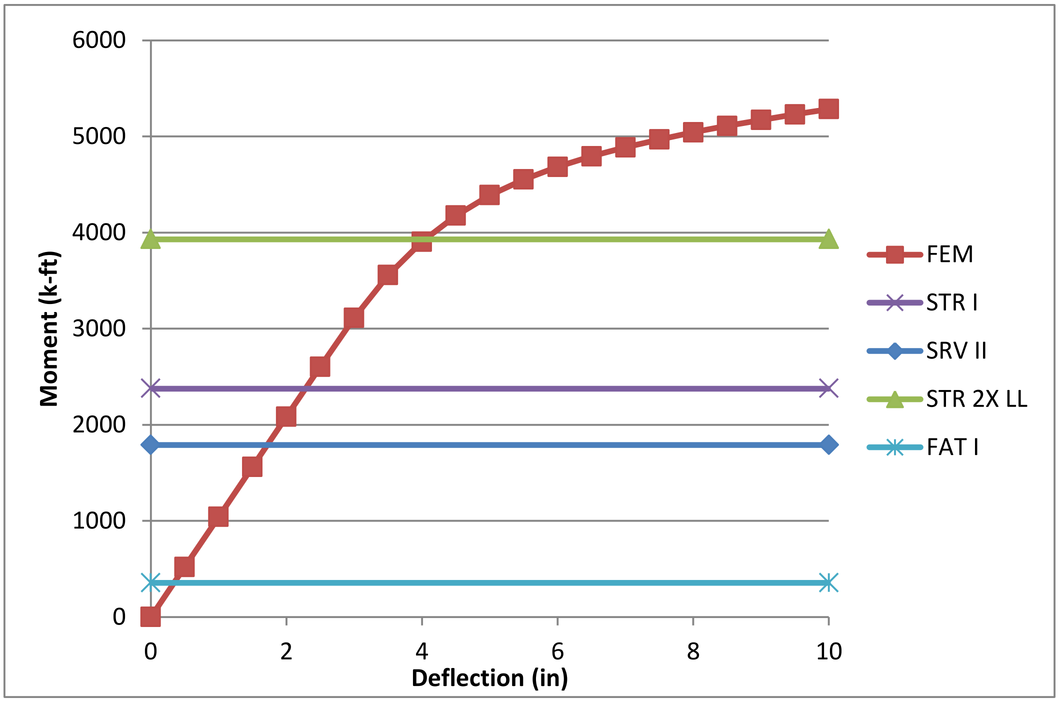


Figure 23. Moment Versus Applied Deflection.

The factored and distributed moment at the Strength I load level is 2377 k-ft. The moment occurred at deflection level of 2.3 inches in the finite element model. **Figure 2**4 shows the stresses at the Strength I limit state.

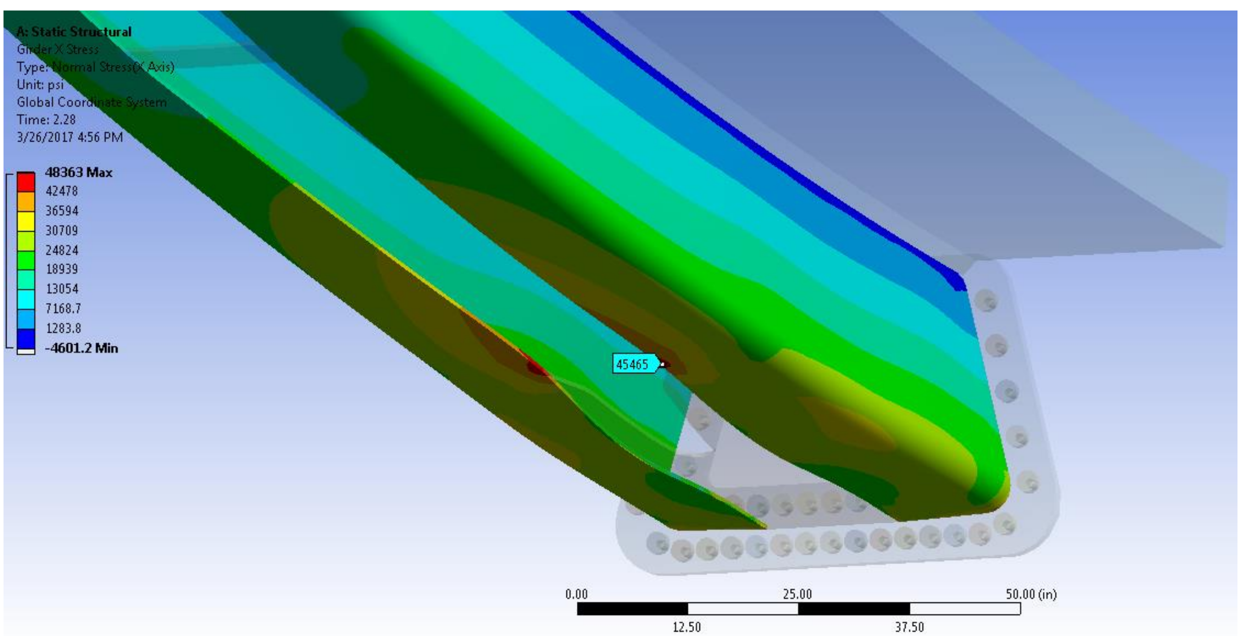


Figure 24. Strength I Stresses.

E.2. Direct Weld

Due to the folded plate fabrication process, the manufacture of the folded plate girder has inherent uncertainties concerning geometry. The greatest advantage of the bolted connection was that misalignment between the flanges could be compensated for with the splice plates. After speaking with several fabricators, it was determined that similar alignment could be achieved with a direct weld process as well.

Figure 25 shows the initial direct weld concept. Each segment of girder is directly welded to a common splice plate. The analysis results indicated a significant stress concentration at the corner of the flange. Considerable effort was directed at smoothing the transition. A filler plate was added to the inside of the flange to provide a radiused transition and the groove weld at the plate was built out and radiused. However, it was eventually determined that none of these measures lowered the stress concentrations to an acceptable range.

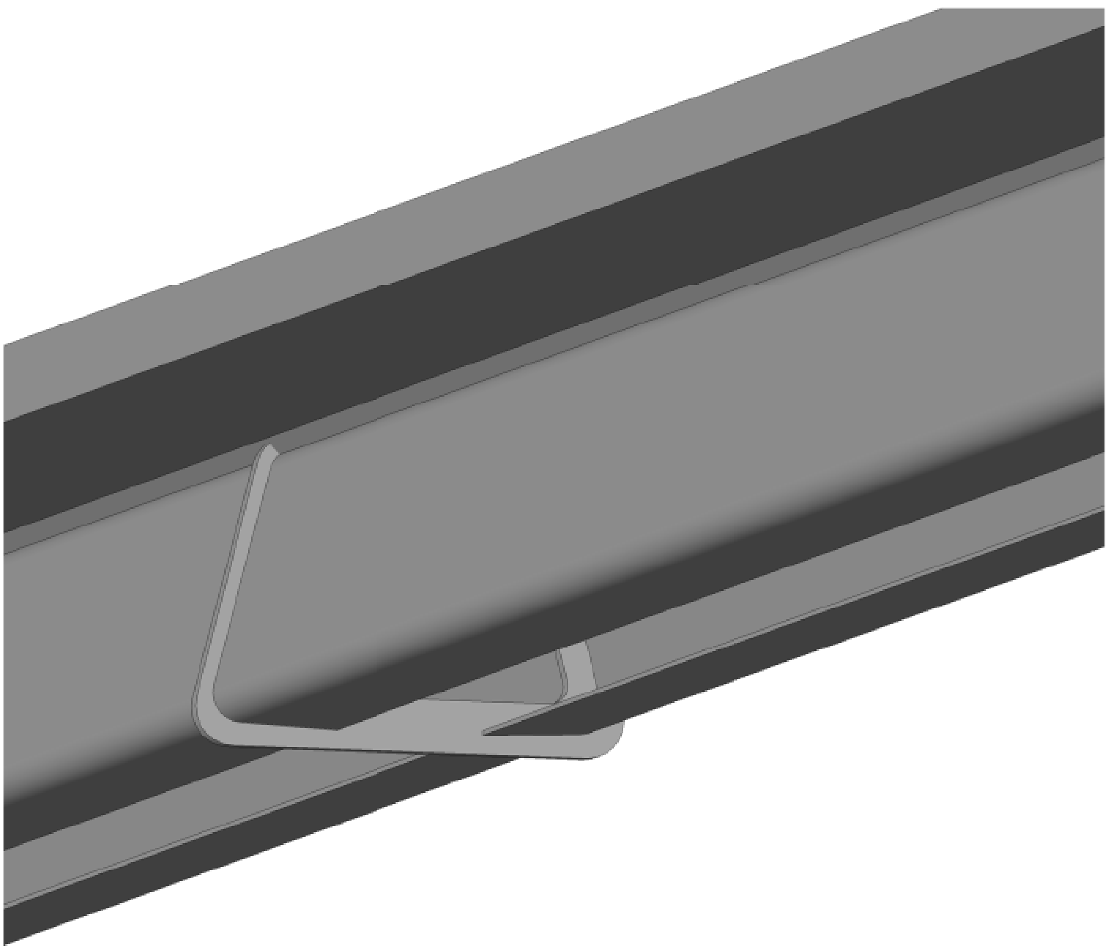


Figure 25. Initial Direct Weld Model.

E.3. Direct Weld with Fill Plate

Based on the results of analyses attempting to smooth the transition at the splice plate, it was ultimately decided to return to the fill plate. The fill plate is a segment of plate that fills the gap between the bottom flanges near the splice plate; see **Error! Reference source not found.** 26. The fill plate welds direct to the splice plate itself and drags the load back into the flanges through shear.

A major parameter that influences the behavior is the length of the fill plate. This is investigated in the following section.

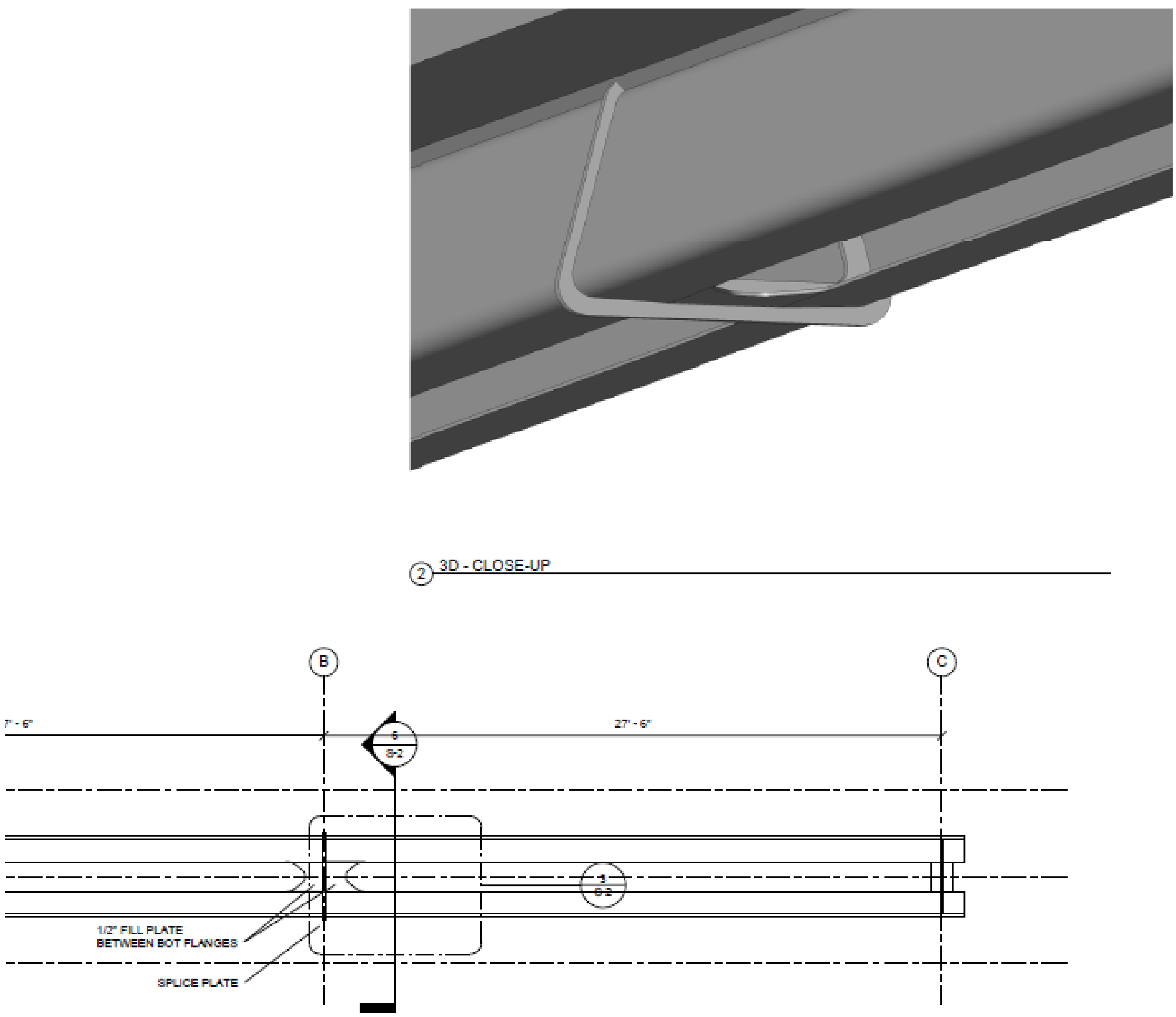


Figure 26. Fill Plate in Direct Weld Configuration.

A series of analyses was run with varying lengths of plates. For each case, an initial elastic analysis was performed to evaluate the local stress concentrations followed by a full non-linear analysis to investigate the ultimate behavior.

Both the elastic and inelastic analysis used a loading equal to three (3) times the HL-93 loading. This was done to ensure the loading was large enough to produce an inelastic response. Since the elastic result is directly proportional throughout, service load stresses can be obtained by dividing the results by a factor of three. Note that the loading applied is a full lane so the results must be adjusted by the multiple presence factor. Finally, since the loading includes the lane load, the elastic stress result is not the fatigue loading.

E.4. Short Plate (Case e)

The short plate leaves a region of low stress near the middle (across the width). The cause is due to shear lag; insufficient length to transfer load. As such, more load is carried directly by the flanges. This is evidenced by the low level of stress near the center of the plate in **Figure 27** (1,698 KSI).

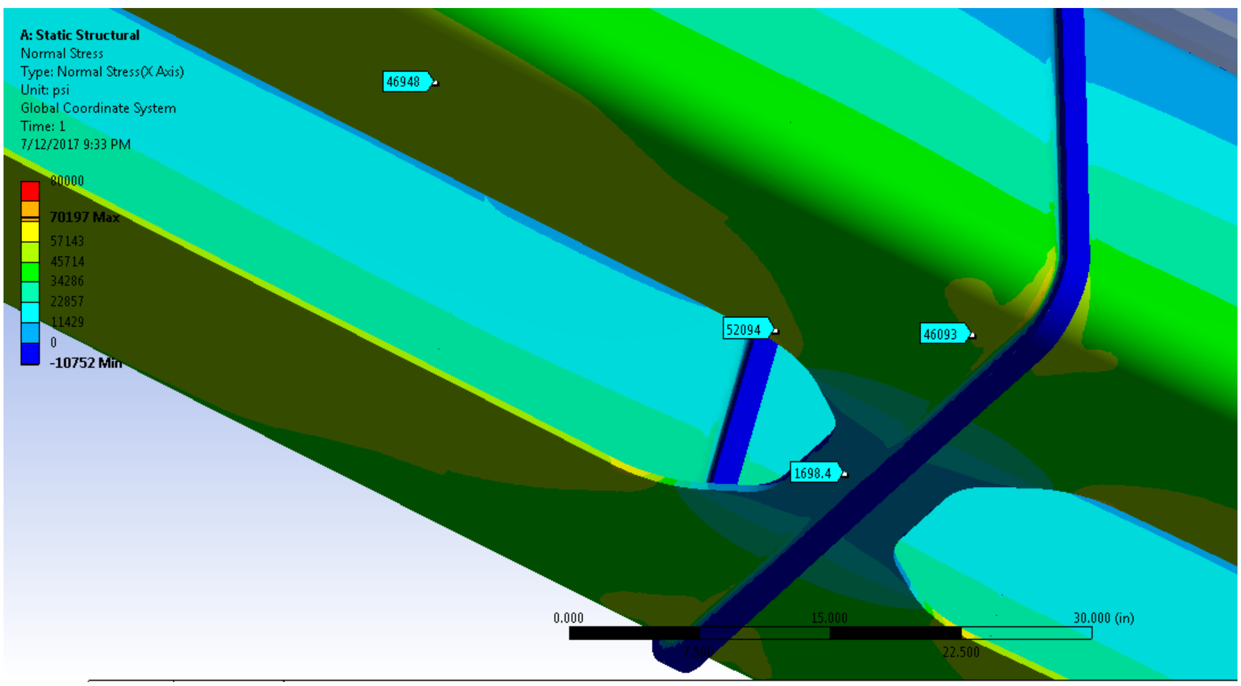


Figure 27. Short Plate (Case e)

The ineastic result is shown in **Figure 28**. This figure shows the equivalent plastic strain. The chart shows the maximum value (y-axis) versus time (x-axis). Time=1.0 corresponds to full load (3x HL-93).

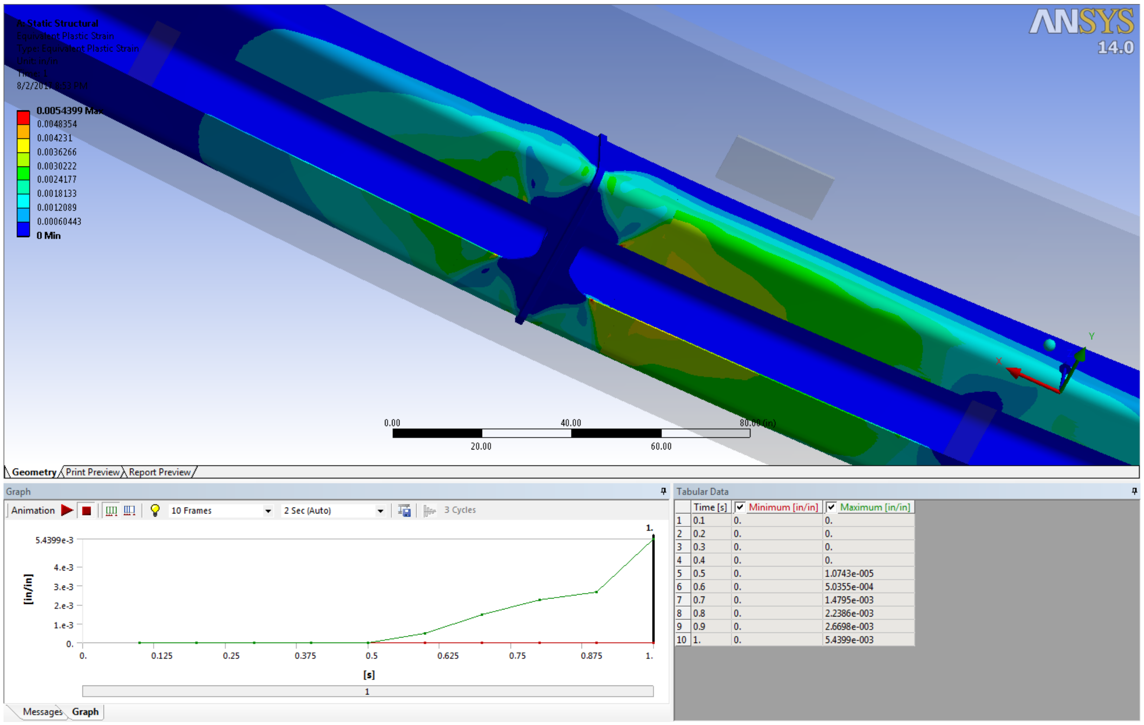


Figure 28. Short Plate (Case e) - Inelastic

E.5. Medium Plate (Case f)

As the filler plate is made longer as shown in **Figure 29**, more load is dragged into the filler plate and the stress near the middle increases. The load carried by the flange decreases. Note that the stress near the middle of the plate has increased to 11,876 ksi (from 1,698 ksi).

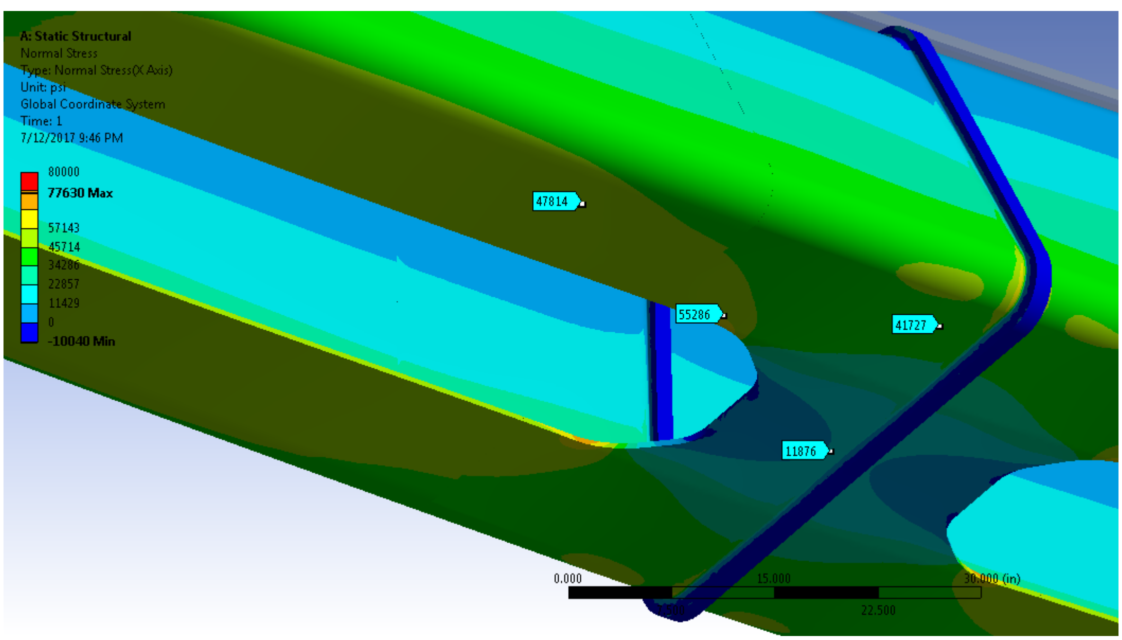


Figure 29. Medium Plate (Case f)

Inelatic analysis shows a similar pattern with a slightly larger strain, 7.9x10-3 verus 5.5x10-3 as shown in **Figure 30**.

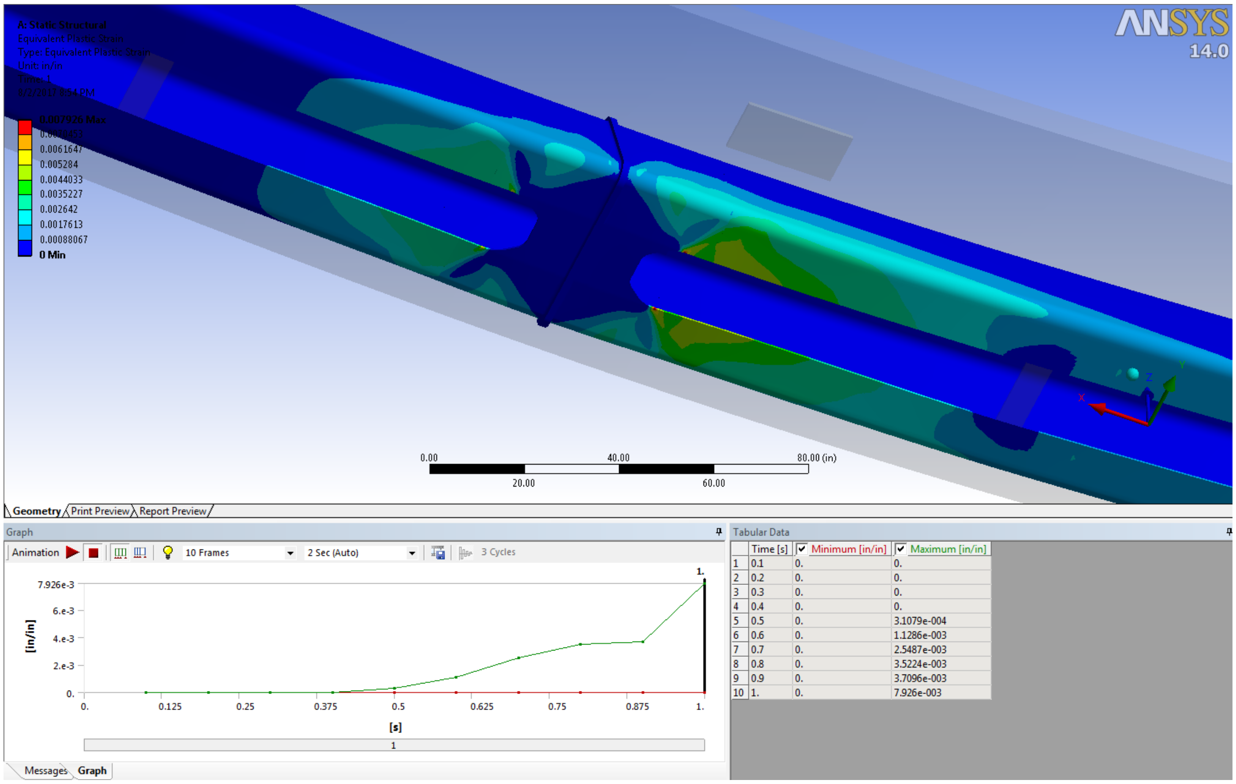


Figure 30. Medium Plate (Case f) - Inelastic

E.6. Long Plate (Case g)

With a very long fill plate as shown in **Figure 31**, the stress becomes more uniform across the width. However, since there is more load being dragged into the filler plate, the stress at the terminus of the filler plate increases (stress concentration).

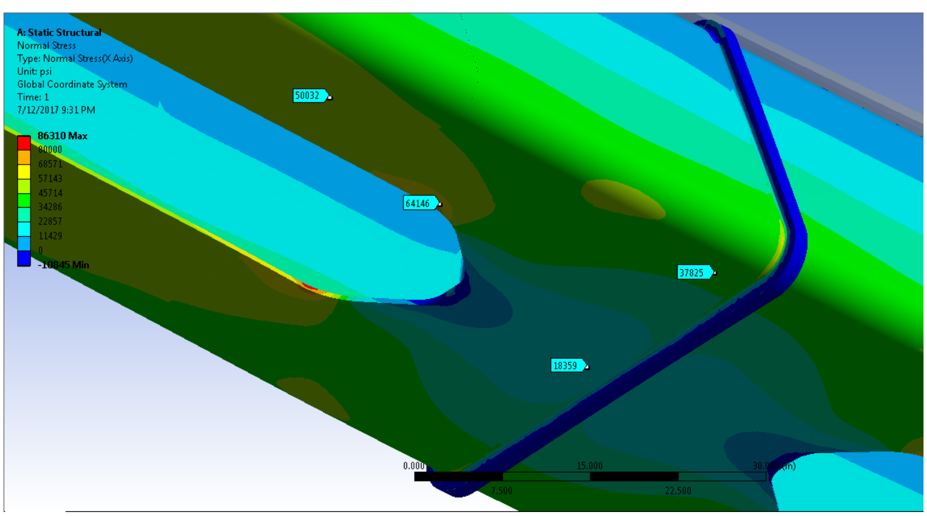


Figure 31. Long Plate (Case g)

Inelastic analysis again shown an increase in plastic strain over the shorter plates as shown in **Figure 32**.

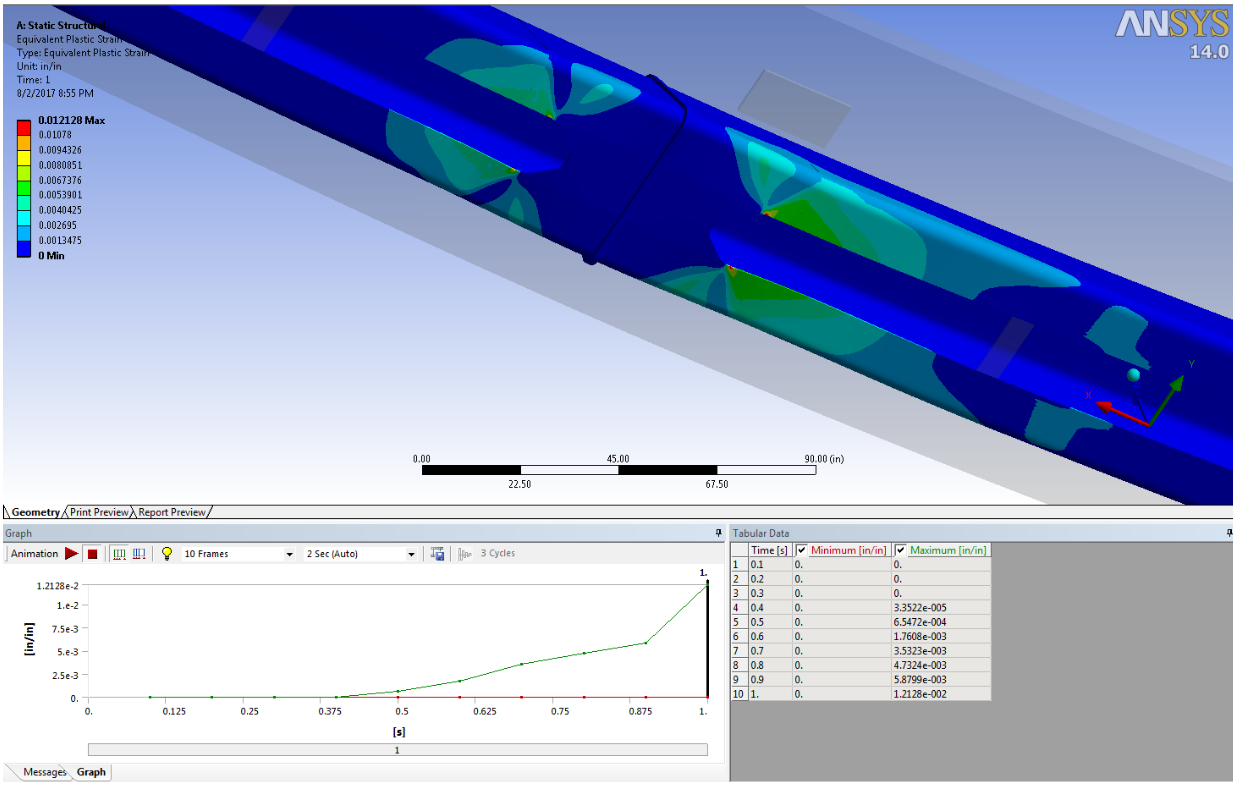


Figure 32. Long Plate (Case g) – Inelastic

F. Conclusion

Based on numerous analysis that were carried out, decision was made to construct two test specimen, one using flange plate and another using direct welding, utilizing full penetration weld. The two test specimens are shown in **Figure 33**. The specimen on left uses filler plate and flange plat. Test specimen on the right uses direct welding, without any filler plate and full penetration weld.

The full penetration weld could be used when the two section being joined have similar cross sections, allowing use of full penetration weld. Otherwise connection with flange plate and filler plate is an option for splicing two girders and achieving longer lengths.

These two test specimens, during the next phase of the study will be tested under static, cyclic and ultimate load tests as shown in **Figure 34** for test setup. A third test specimen using bolted type connection may also be fabricated, if needed. Results will then be used to complete the development of the design provisions for extending the maximum length of the Folded Plate Steel Bridge system to about 100 ft.

Figure 33. Specimens for Future Experimental Work.

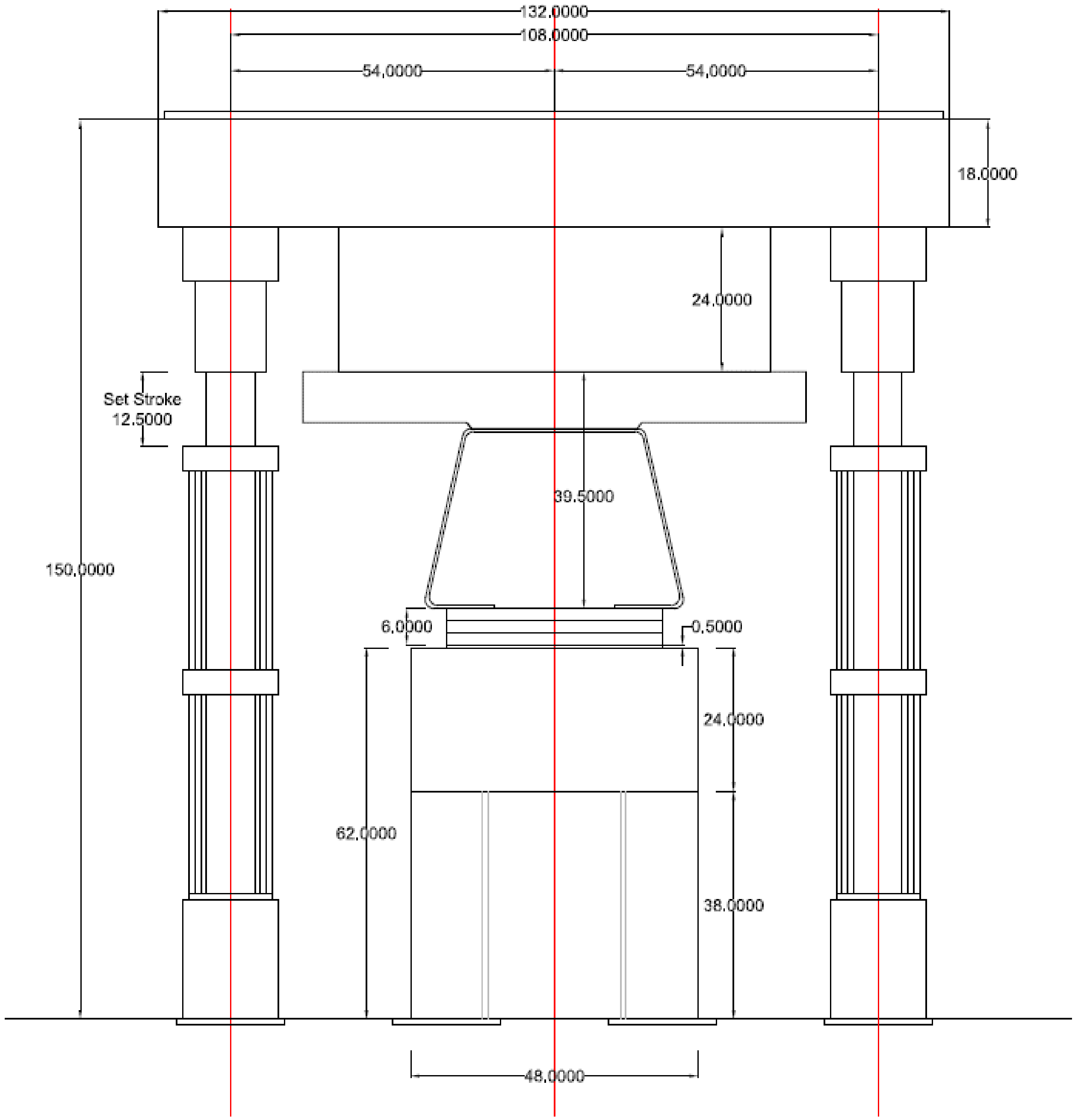


Figure 34. Test Setup for Future Experimental Work.