Standardizing Existing Spliced Girder Systems

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Abstract:
Correlation of the span and girder spacing requirements for an existing spliced girder system can be completed using a parametric time-dependent analysis of the precast post-tensioned system. This paper outlines the steps taken to 'standardize' the span and spacing requirements for the spliced girder forms used successfully on the St. George Island, Bay St. Louis, and Biloxi Bay projects. A three span continuous spliced girder system was chosen for parametric analysis, based on it's extensive use for long span construction. The parametric analysis employed can be extended to any set of forms, for any number of spans, and provides a useful tool for preliminary design and standardization.

Keywords:
FBT-78 Girder
Spliced Girders
Parametric Analysis
Standardization

Introduction

Many tools and references are available to determine the maximum practical span and spacing of a number of standardized girder sections. Girder selection is often based on preliminary analysis using these tools, often without consideration of the inventories of available forms in the precasting plant.

However, maximum economy in design and construction of precast concrete components is often achieved when beam selection is made from the inventory of available forms from local or accessible precasters. Furthermore, rapid or emergency bridge replacement, or other scheduling factors, may dictate the use of available forms. Standardized data tables can provide a quick guide to the most efficient use of available girder forms. This paper will illustrate two projects in which girder form work usage was optimized based on schedule and availability to the precasters, and will provide a parametric analysis of the forms used to show the potential benefits of girder standardization..

Design-Build Bridge Replacements in Mississippi

A very good case for standardization involves the recent emergency design-build replacements of the Biloxi and Bay St. Louis Bridges on US 90 in Mississippi. For both projects, rapid product delivery was necessary to meet an aggressive construction schedule. Cognizant of the need to quickly design, fabricate, and deliver the replacement bridges at both of these sites, both design build teams quickly concluded that use of available girders and spliced girder forms would result in the quickest project delivery and most economical solution.

Gulf Coast Prestress had the spliced girder forms that had been used for the St. George Island Bridge available at their casting yard. At St. George Island, a five-span, continuous spliced girder system consisting of modified FBT-78 girders was used, with maximum spans of 260 feet (79.25 meters). Both the Bay St. Louis and Biloxi Design-Build projects required channel spans of 250 feet (76.20 meters), and preliminary design confirmed the suitability of the available spliced girder forms.

The Bay St. Louis Project ultimately utilized a spliced girder system with spans of 200 ft. (60.96 meters), 250 ft (76.2 meters)., and 200 ft (60.96 meters), at a girder spacing of 11.4167 ft. (3.480 meters). The Biloxi Project utilized a spliced girder system with the same span arrangement, but at a girder spacing of 11.75 ft. (3.581 meters). The St. George Island Bridge had utilized a span arrangement of 200 ft (60.96 meters), 260 ft. (79.25 meters), 250 ft. (76.20 meters), 260 ft. (79.25 meters), and 200 feet (60.96 meters), and girder spacing of 9.5 feet (2.896 meters). In every case the same spliced girder forms were used for pier, end, and drop-in segments. Figures 1 through 3 show photos of the Biloxi, Bay St. Louis, and St. George Island projects.

Girder Standardization

For the purpose of demonstrating the flexibility of an actual spliced girder system, the approximate minimum and maximum span configurations and corresponding girder spacing of the three span spliced girder system used for the projects was estimated. It was assumed that the design was in accordance with AASHTO LRFD Bridge Design Specifications, and the vehicular design load was the HL-93 vehicle. Several key parameters were fixed: The concrete compressive strength for the precast members was set at 8500 psi.(58.61 MPa). which is generally recognized as a reasonably high strength concrete which can be batched by most precasters. The deck concrete compressive strength was set to 4500 psi (31.03 MPa), and the number and size of tendons fixed based on the existing end block geometry.

The Gulf Coast Pre-Stress FBT-78 spliced girder system uses (4) 4 inch (102 mm) diameter tendons. Consequently, the maximum number of tendons and tendon diameter was limited to (4) 4 inch (102 mm) diameter tendon ducts. This precludes the need to adjust the end block size or do an extensive check of the end block anchorage, and is based on use of a 9 inch (229 mm) web thickness. [Girder forms can be further separated to achieve a thicker web, but it is not our intent here to vary this parameter.] The pier segment geometry (length and depth of the pier segment) was also fixed.

Other parameters that were fixed include the creep and shrinkage coefficients, friction coefficients, and section properties. A summary of the Input data, the parameters fixed for analysis, and the values used, are shown in Table 1.

Table 1

Input Data for Parametric Analysis (Typical Interior Girder)
Three Span Continuous Spliced FBT-78; 2 Stage Post-Tensioning

Case	Span Lengths (ft)	Girder Spacing (ft)	LLMDF	LLVDF
1	200 – 250 – 200	12.75	0.886	1.12
2	208 – 260 – 208	12.5	0.864	1.11
3	216 – 270 – 216	12.0	0.830	1.08
4	224 – 280 – 224	11.0	0.772	1.02
5	232 – 290 – 232	10.0	0.714	0.95
6	236 – 295 – 236	9.5	0.695	0.92

LLMDF – Live Load Moment Distribution Factor
LLVDF – Live Load Shear Distribution Factor
Reference Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1 of the AASHTO LRFD Bridge Design Specifications

(Table 1 Cont.)

End and Drop-In Segment Lengths Vary
End Block is 12 ft. (3.658 meters); 9 ft. (2.743 meters); 3 ft. (.914 meters Transition)

Pier Segment Depth and Length are Constants; Pier Segment Length is 115 ft. (35.152 meters); Maximum Pier Segment Depth is 12 ft. (3.658 meters).

Deck Thickness: 8 ¼ Inches (210mm); Width Varies –Dead Loads vary as Deck Width varies

(4) 16 Strand Tendons used in all Parametric Analyses
Tendon Profiles are Constant – Parabolic Profiles Proportional to the Span Length

Prestressing Strand: 0.6 Inch Dia. 270 ksi (1862 MPa) Lo Lax Strand

- **End Segment Prestress; (28) 0.6 Inch Dia. Strands for 200 ft. End Span, (40) 0.6 Inch Dia. Strands for End Spans in Excess of 200 ft.**
- **Drop-In Segment Prestress; (40) 0.6 Inch Dia. Strands for Segments**
- **Pier Segment; (34) 0.6 Inch Strand – (24) Upper & (10) Lower Strands for all Segments**

Post Tensioning Tendons:
- **0.6 Inch Dia. 270 ksi (1862 MPa) Lo Lax Strand**
- **4 Inch (102mm) Dia. Tendon Ducts; (16) Strand Capacity**
- **Angular Friction (μ) = 0.15 (Pounds/Inch)**
- **Wobble Friction (K) =0.00020/12 (Pounds/Inch)**

Concrete Properties
- **Deck f'c = 4500 psi (31 MPa)**
- **Girders f'c = 8500 psi (58.6 MPa)**

Concrete Creep and Shrinkage: ACI 209
- **Creep Cr = 2.0 Ult. Creep Strain Coeff.**
- **Shrinkage Sh = 0.0004 Ult. Shrinkage Strain Coeff.**

Section Properties	Area in.^{2}	Moment of Inertia - in.^{4}	Y_t in.	Y_b in.
Typical Section	1260.9	1,014,886	37.79	40.21
End Block	2329.8	1,291,195	36.71	41.29
Pier Segment (@ Centerline)	3305.0	6,449,280	80.28	63.72

A common template for parametric study of spliced girders can be adapted to virtually any existing spliced girder system, and hence a 'standardized' data base of available forms quantified for future use.

Establishing the maximum practical span that can be accommodated using the FBT-78 spliced girder system is perhaps the most important parameter to be determined. Consequently this is the primary focus of the parametric study. Common sense dictates that to satisfy the governing design load requirements, girder spacing lessens as span lengths increase. Girder spacing and span length are effectively interdependent.

The term minimum span is a misnomer, because minimizing the span length of a system is seldom cost-effective. However, the pier segment and drop-in forms can be used (exclusive of end segments) to construct moderately long main spans with flanking spans as short as 65 feet (19.812 meters). The design is based on construction of end blocks immediately adjoining the pier segment in the flanking spans, and anchoring the end spans to counter balance the main span fixed end moments. This system indicates a potential use for deep pier segments where a long main span is desired, but there is little need or space for flanking spans.

For the purpose of analysis two key parameters were varied: the segment lengths, which influences span length, and the girder spacing. It is assumed that the span lengths will be balanced (or nearly so) for optimum moment distribution between flanking and main spans. I.e., as the channel/main span increase, the flanking span lengths are adjusted to maintain a ratio of 0.8 between flanking span and channel span The prestresing force, girder post-tensioning – both the trajectory and total pull, and shear reinforcement can be adjusted as required to satisfy strength and serviceability requirements.

Adapting these restrictions, it is possible to quickly run a series of time-dependent analyses to establish the maximum span for a given girder spacing. By adjusting the girder spacing and re-running the series of analyses for each spacing, the maximum span length and relationship of span to girder spacing can be established.

Analysis of the spliced girder system using the parameters outlined above is readily done using the time dependent analysis program ADAPT. The only input that needs to be varied from run to run consists of the joint spacing (nodal coordinates), construction dead loads (deck and build-up weights), and live loads (live-load distribution factors).

Output can be checked graphically for stress violations, and strength at various critical points checked against enveloped moments and shears on the composite section. A relatively simple MATHCAD template can be used for strength calculations. Results were computed at 10,000 days, representative of the state of stress after creep and shrinkage losses have essentially run their course. A girder spacing of 9 ft - 6 inches (2.896 meters) was used for the St. George Island project, while 11 ft.- 9 inches (3.581 meters) was used for the Biloxi project. For the purpose of this paper, a range of girder spacing from 9 ft.- 6 inches (2.896 meters) to 12 ft.- 9 inches (3.886 meters) was used, and approximate span limits established for girder spacing of 9 ft. - 6 inches, 10 ft., 11 ft., 12 ft, 12 ft - 6 inches, and 12 ft.- 9 inches (2.896 meters, 3.048 meters, 3.353 meters, 3.658 meters, 3.810 meters, and 3.886 meters. Flexural and shear results are summarized in Tables 2 and 3.

Table 2

Flexural Results
Three-Span Continuous Spliced FBT-78 Typical Interior Girder
2 Stage Post-Tensioning

Span Lengths (ft)		Girder Spa. (ft)	Demand to Capacity Ratios Strength Limit State						Extreme Fiber Stress Service Limit State	
End Span	Main Span		M_{up} k-in.	ΦM_p	Ratio	M_{un} k-in.	ΦM_n	Ratio	σ_b (ksi)	σ_t (ksi)
200	250	12.75	247,330	380,500	0.65	431,720	591,900	0.73	-.450	0.126
208	260	12.5	302,480	436,800	0.69	492,810	591,900	0.83	-.604	0.087
216	270	12.0	277,170	436,800	0.63	464,930	591,900	0.79	-.443	0.146
224	280	11.0	248,090	436,800	0.57	438,450	591,900	0.74	-.322	0.044
232	290	10.0	281,500	436,800	0.64	467,560	591,900	0.79	-.045	0.147
236	295	9.5	280,940	436,800	0.64	469,850	591,900	0.79	0.005	0.163

The maximum positive moment on the composite girder at the critical section in the end span is M_{up}. The maximum negative moment on the composite section over the interior piers is M_{un}. The corresponding extreme fiber stresses are the bottom stress in the prestressed end segment at the location of the maximum positive moment and the top stress in the deck over the pier at the location of the maximum negative moment. Note that the cracking (rupture) stress in 4.5 ksi (31.03 MPa) concrete is approximately .500 ksi (3.45 kPa) indicating that the maximum tensile stresses in the deck remain well within reason. Figures 4 and 5 show the factored and enveloped moments and shears on the composite section of the 232 ft. – 290 ft. – 232 ft. spliced girder system.

Figures 6 through 10 show the working stresses at critical stages of erection for the same parametric analysis. The results indicate no overstresses occur during erection.

Neither the ultimate strength nor service load stresses were exceeded. In fact, there was considerable safety factor at the maximum factored moment. However, checking the ultimate strength at only two critical points is not adequate for final design, and the 4th edition of the AASHTO LRFD Bridge Specifications incorporates a floating phi factor which will come into play when checking ultimate strength over the length of the tapered pier segment (consuming as much as 33% of the calculated nominal strength). Service load stresses are dependent on parameters that can vary considerably; for example, concrete cylinder strengths, creep and shrinkage parameters, tendon friction coefficients, and thermal gradients, etc. Also, only deck, build-up, barrier, and future wearing surface loads were considered; utility loads, interior diaphragm weight, or other dead loads may be required in final design.

Table 3

Shear Results
Three-Span Continuous Spliced FBT-78 Typical Interior Girder
2 Stage Post-Tensioning

Span Lengths (ft)		Girder		Demand to Capacity Ratios		
		Spacing (ft)		Strength Limit State		
End	Main		Vuend	ΦVnend	Vuint	Φ Vnint
Span	Span		(kips)		(kips)	
200	250	12.75	598	1090	877.1	1908
208	260	12.5	603.6	1090	884.2	1908
216	270	12	599.5	1090	881.6	1908
224	280	11	567.3	1090	844.8	1908
232	290	10	575	1090	852.2	1908
236	295	9.5	566.1	1090	841	1908

ΦV_n is calculated in accordance with the AASHTO LRFD Bridge Design Specifications Equation 5.8.3.3-2 (neglecting contribution of the tendon). The depth of the section for shear is taken as 0.72h, where h is the depth of the composite member. V_{uend} is the maximum factored shear at the exterior support, and V_{uint} is the maximum factored shear at the interior support.

Available shear capacity was in excess of the maximum calculated shears at the end and interior supports.

It is note worthy that the amount of prestress in the girders did not have to be raised to satisfy ultimate strength; what worked for the Bay St. Louis project with a maximum span of 250 ft. satisfied ultimate strength requirements in the 290 ft. unit in the parametric analysis. However, the prestress needed to provide positive camber in end segments under dead load alone had to be raised to prevent sag at release of prestress.

There are relative advantages of using existing forms vs. designing a new system optimized for a specific span: First, time is saved; it takes at least 2 months to procure new forms, and they cannot be purchased until the new design is finalized. Money is also saved. At $250.00 a linear foot, which is roughly today's cost for a line of spliced girders fabricated erected and stressed, it is very often less expensive to have an additional line of girders than to purchase new forms and upgrade existing stressing beds. It is not unreasonable to expect the cost of new forms and ancillary upgrades to be in the neighborhood of $250,000.00. An additional girder line may also provide a more redundant, serviceable design.

For main spans of 270 ft., 260 ft., and 250 ft. the girder spacing is varied from 12 ft to 12 ft - 9 inches, reflecting the author's belief that 12 ft. - 9 inches girder spacing is probably about the maximum which should be used. It is not based on strength limitations, but reflects the author's judgment and experience – though stretching the girder spacing may well prove to be viable.

Main spans of as little as 200 ft. may be practical and efficient using the system shown above, but the author would not recommend increasing the girder spacing much beyond 12 ft. - 9 inches without careful consideration.

Figures 11 and 12 show the end block and pier segment details. Figure 13 shows the typical span geometry used for this analysis. For a three span unit with main span of 290 ft., the end and drop-in segment lengths are approximately equal, at 175 ft. in length. Stretching FBT-78 segment lengths much beyond 175 ft. requires careful consideration of handling and transport stresses and stability.

Note that to maximize the span capability of the spliced girder system, temporary shoring towers remain in place until deck pours have been completed and the girders are ready for second stage post-tensioning.

Conclusion

In summary, the girder sections and forms used for the St. George Island, Bay St. Louis, and Biloxi projects appear to be well suited to a wide range of spans, from 200' to 290'. Similarly, forms which have been used on other projects, and are currently inventoried at other locations, may prove equally versatile. The procedure outlined above can be applied to any set of forms, to better quantify the flexibility and limitations of the systems and provide an additional tool for rapid precast construction.

Precasters, Contractors, Owners and Engineers all benefit from quantitative knowledge of the flexibility and limitations of girder forms locally available to them. The use of beam standardization can be extended to the spliced girder systems currently in use for major waterway crossings. Knowledge of the span range and girder spacing requirements for spliced girder shapes currently within the inventories of local Precast Plants is something that can reasonably be obtained using the same design tools used for project specific design. This knowledge has an immediate and practical value to all of the stakeholders listed above.

Figure 1
This is the St. George Island Bridge
Located in the Florida Panhandle

Figure 2
This is U.S. 90 Biloxi Bay Bridge
Located in Biloxi, MS

Figure 3
This is the U.S. 90 Bay St. Louis Bridge
Located between Pass Christian and Bay St Louis, MS

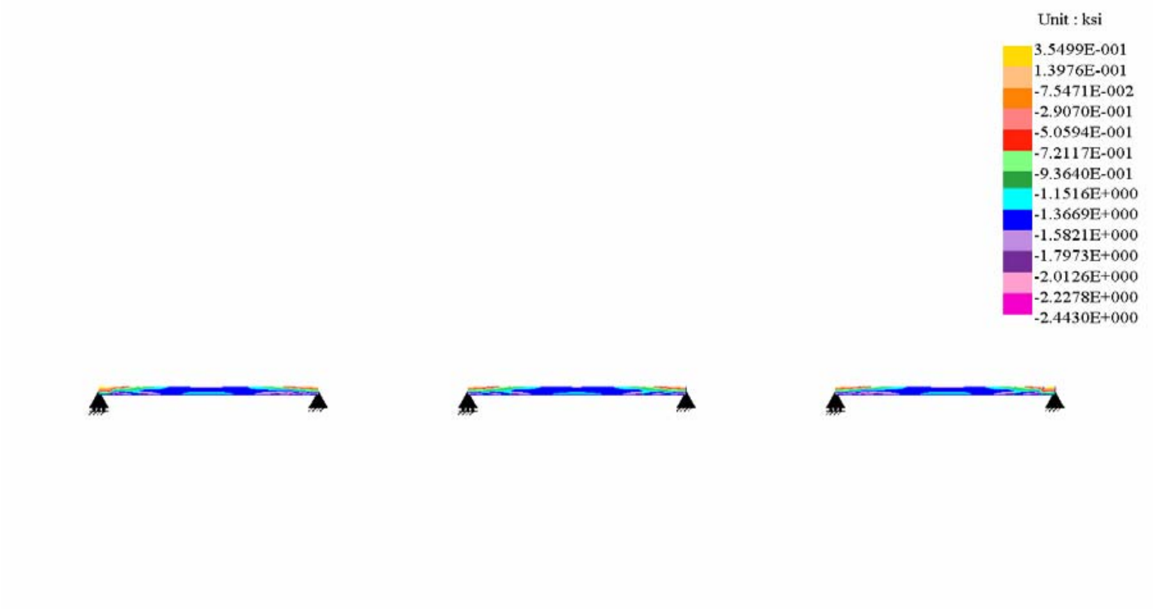

Figure 6
Day 4 End and Drop-In Segment Stresses (In Storage)
Note: 1 ksi = 6.895 MPa

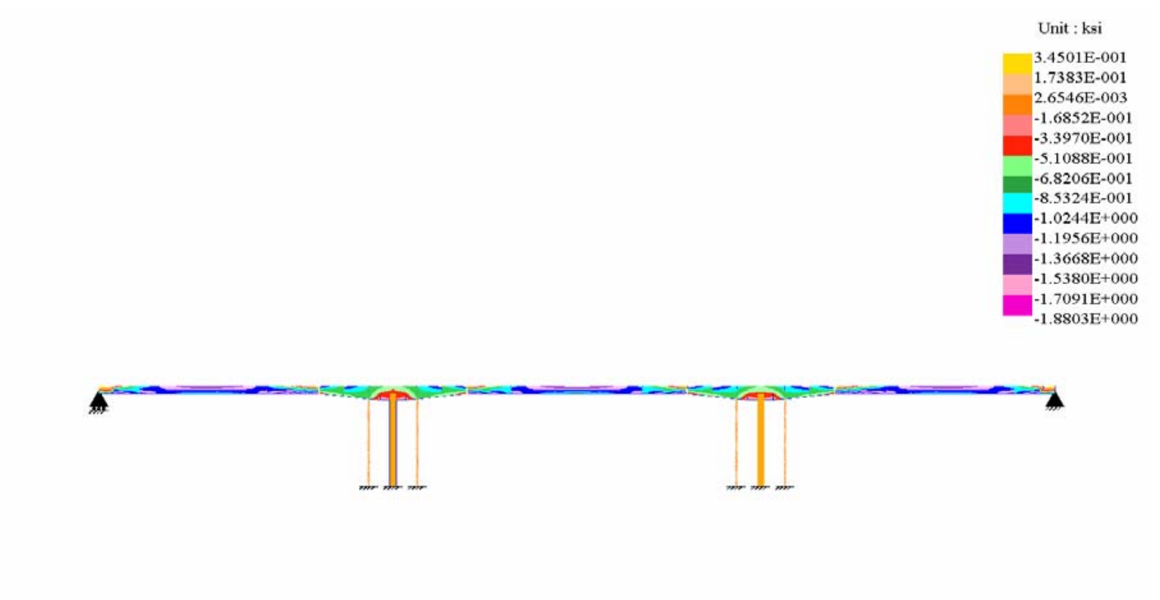

Figure 7
Day 120; End and Drop-In Segments Suspended on Pier Segments
(Immediately Prior to First Stage Post-Tensioning)
Note: 1 ksi = 6.895 MPa

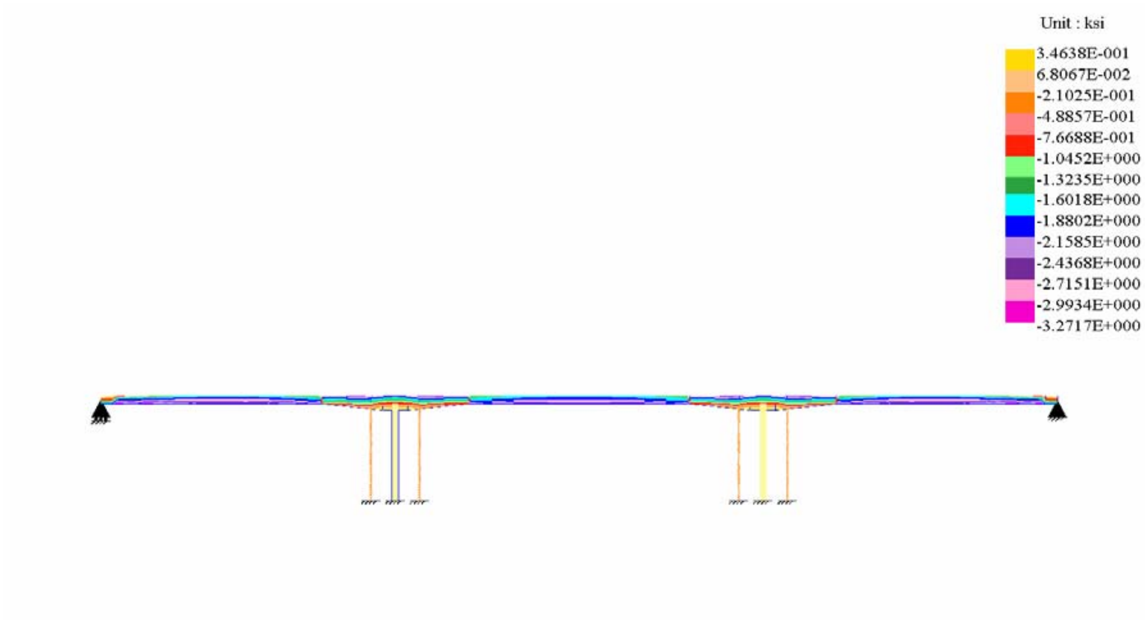

Figure 8
Stresses Immediately Following First Stage Post-Tensioning
(Shoring Towers Remain Through Deck Pouring Sequence)
Note: 1 ksi = 6.895 MPa

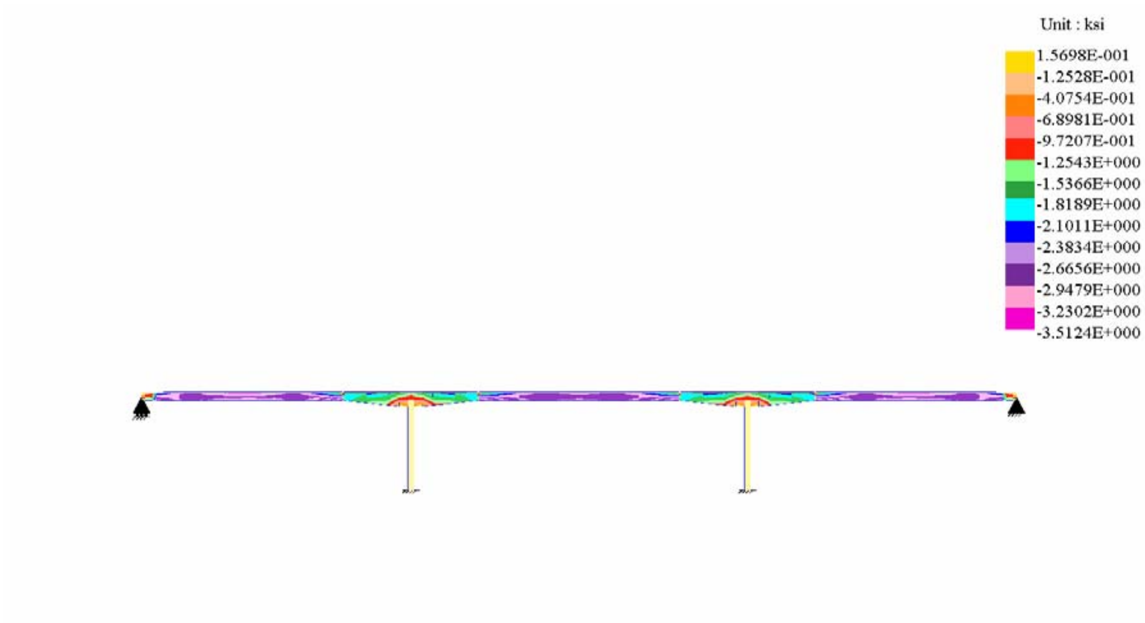

Figure 9
Stresses Immediately After Second Stage Post-Tensioning
(Deck Cast and Temporary Shoring Removed)
Note: 1 ksi = 6.895 MPa

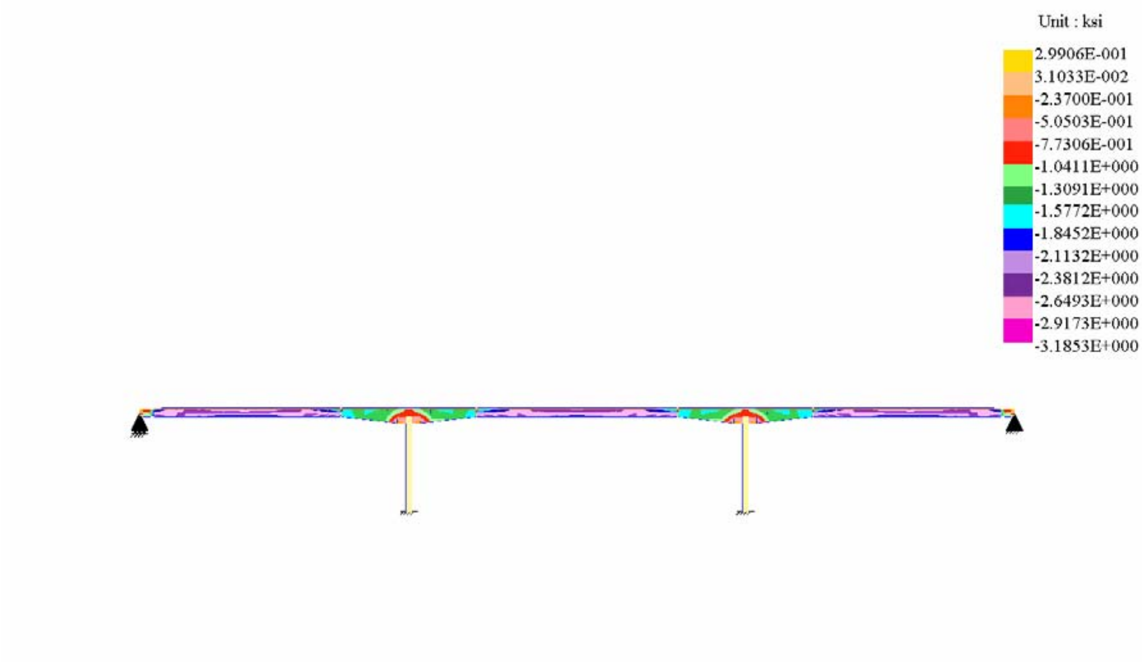

Figure 10
End of Service Life Dead Load Stresses
(10,000 Days)
Note: 1 ksi = 6.895 MPa

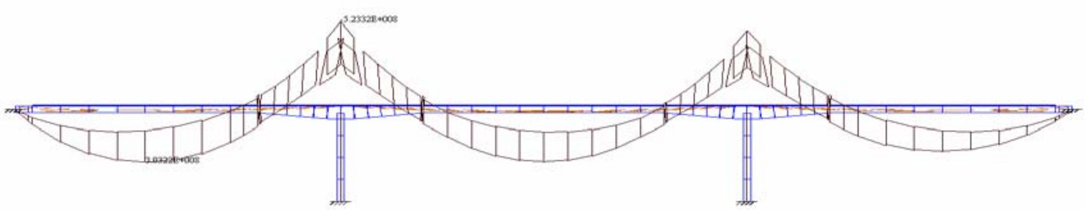

Figure 4
Peak Factored Moments
Strength
Design

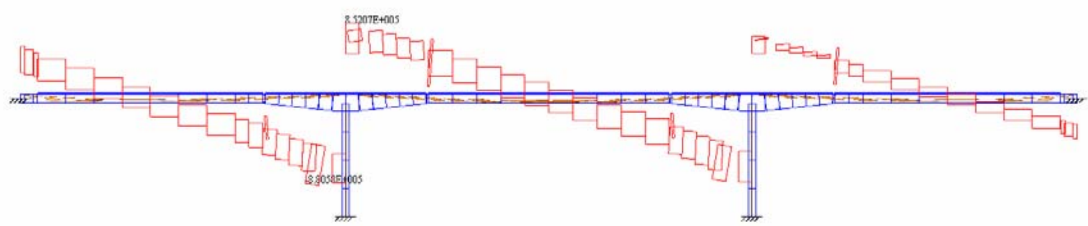

Figure 5
Peak Factored Shears
Strength Design

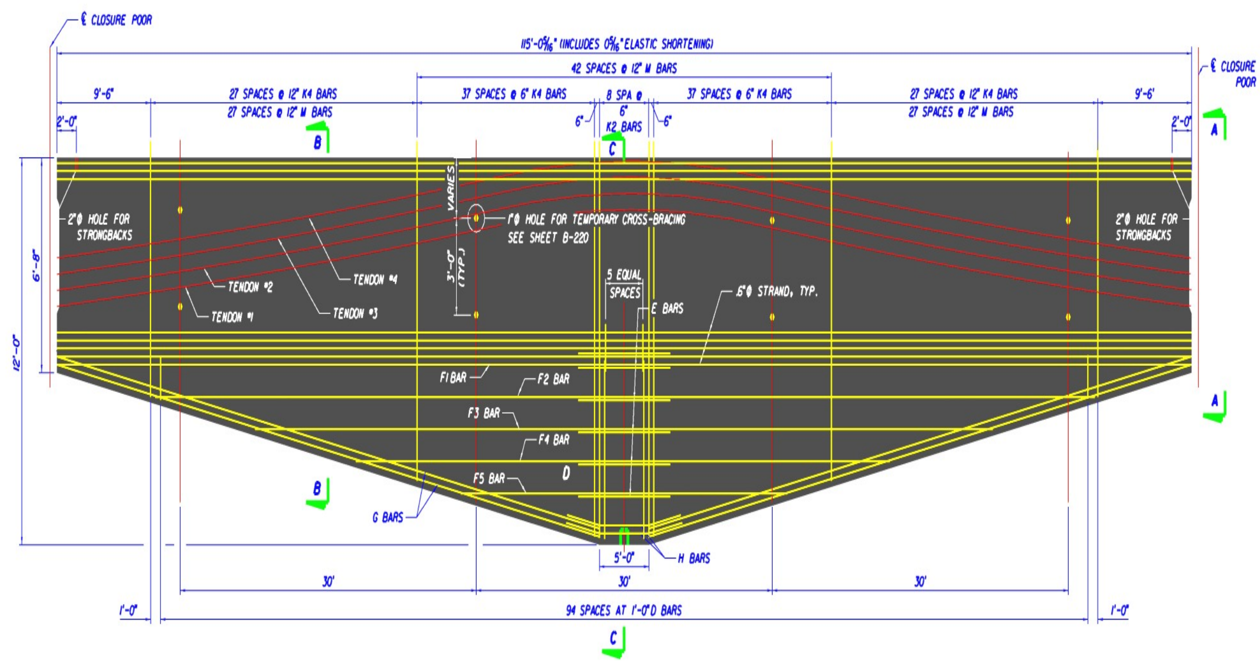

Figure 11
Pier Segment Forms
& Typical Segment Reinforcement

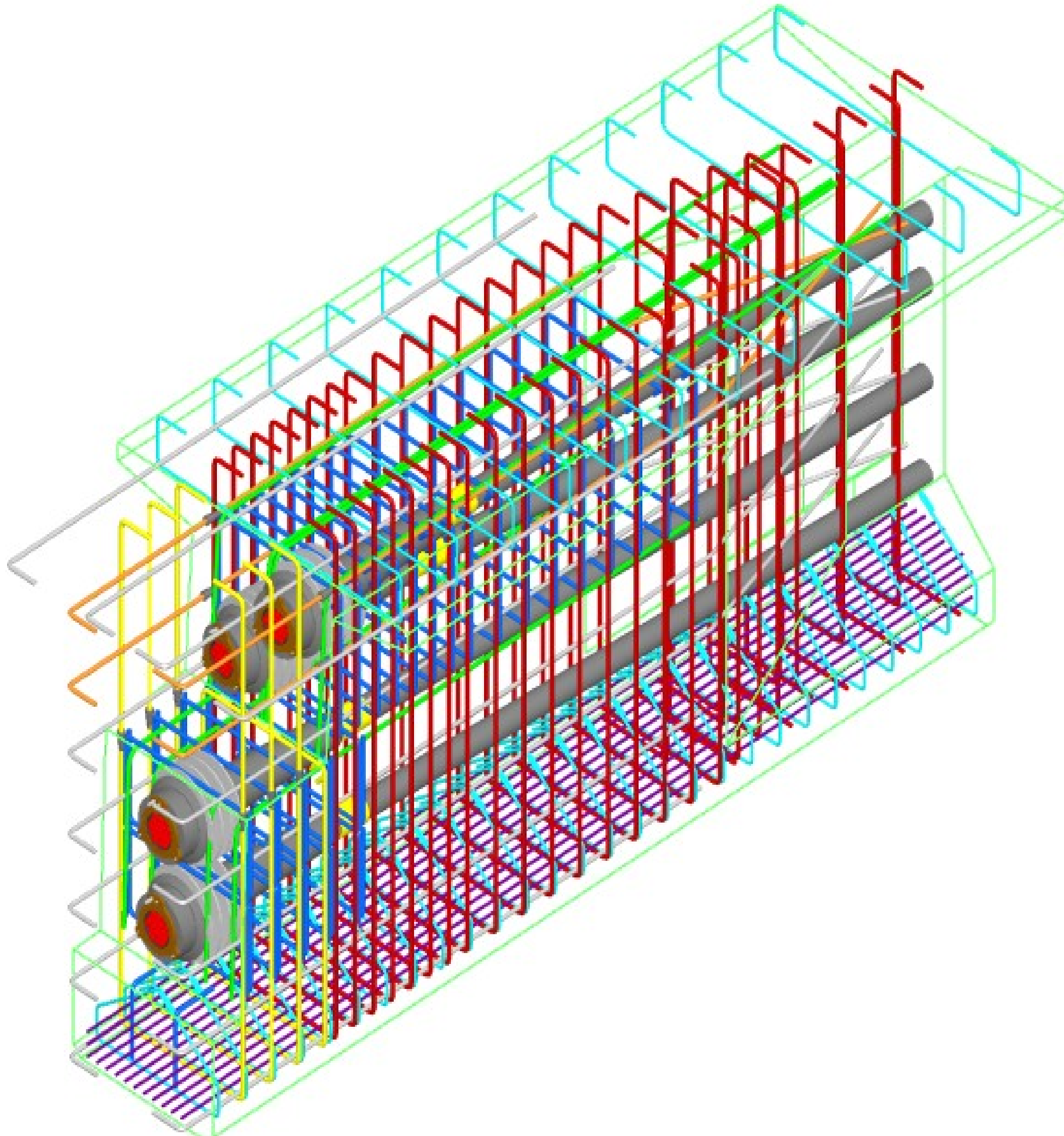

Figure 12
End Block Tendon Anchorage

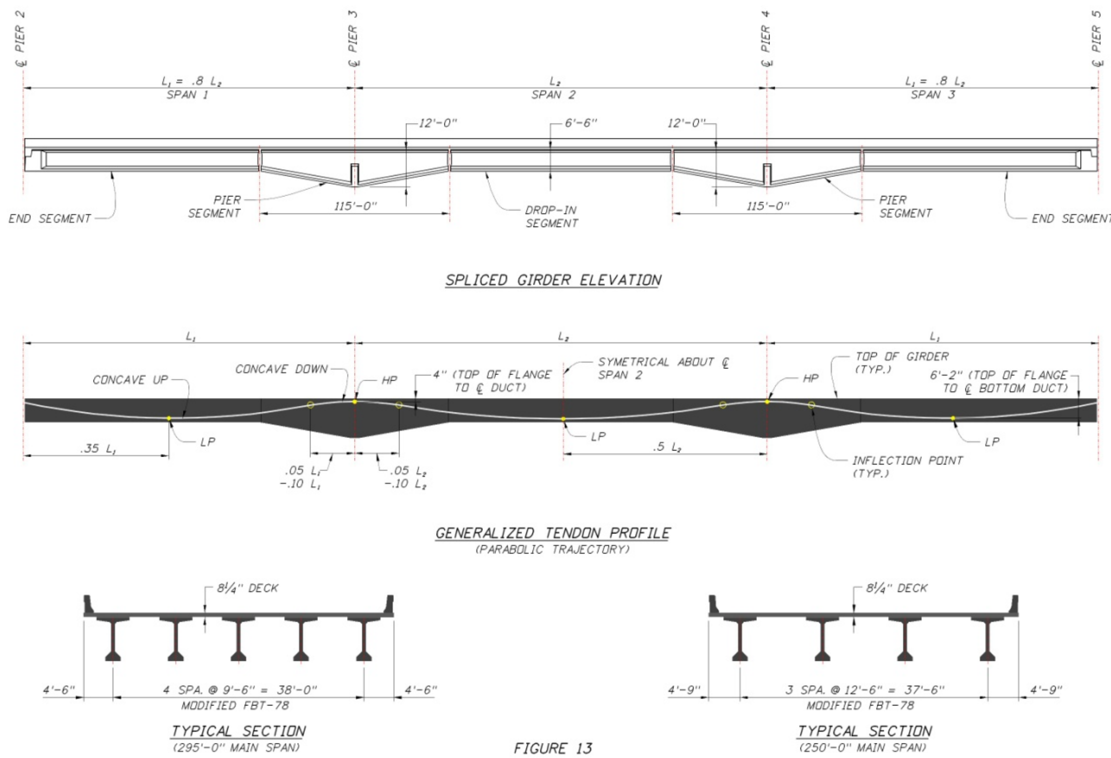

Figure 13
Typical Span Geometry
For Parametric Analysis

Authors

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