DESIGN OF A SPLICED GIRDER SYSTEM TO SPAN 350 FT. THE LA-1 PROJECT

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ABSTRACT

The LA-1 project consists of 17 miles of elevated structure and includes a major navigation channel crossing over Bayou LaFourche at the Town of Leeville.

We are currently completing design of a three span channel unit 870 feet in length, with a center span of 350 feet. The spliced girder system will be the longest of it's type in the World.

A number of design and construction problems unique to the 350 foot span have been investigated, including stability during erection and handling, and ductility of the girders at ultimate strength.

Keywords: Creative Solutions, High Strength Concrete, Design Methodologies, Post-Tensioning, High Performance Concrete, Construction, Spliced Girder

INTRODUCTION

The LA-1 Project consists of 17 miles of bridge between Golden Meadows and Port Fourchon, Louisiana. The project limits are shown in *Figure 1*. The majority will be low-level trestle, utilizing top-down construction to span sensitive wetlands. However, there is a major navigable waterway midway between Golden Meadows and Port Fourchon, over Bayou LaFourche. This site requires a relatively long span, high level structure. During preliminary analysis, the design team compared a three span continuous steel plate girder alternate to a precast spliced girder alternate, both utilizing a 350 foot center span. Our preliminary cost estimates favored the spliced girder alternate by a substantial margin, and the decision to use precast concrete was further reinforced by the need to provide a durable low maintenance design in an extremely aggressive coastal environment.

The design of a spliced girder system to span 350 feet requires consideration of many factors, from segment weights and stability during handling and erection, to material requirements and ductility at ultimate strength. Key elements of the analysis and design of the system will be highlighted.

GENERAL DESIGN CONSIDERATIONS

The design of the Bayou LaFourche Channel Unit is a logical extension of previously completed successful designs utilizing a modified FBT-78 girder. These include the main channel unit of the St. George Island Bridge in the Panhandle of Florida, and the Moore Haven Bridge in south-central Florida. The channel unit of the St. George Island Bridge is a five span continuous structure with a maximum span of 260 feet. Pier segments are 12 feet deep, while drop-in and end segments are standard FBT-78 girders widened 2 inches to accommodate post-tensioning tendons. The Moore Haven Bridge is a three span continuous unit with a center span of 320 feet. Pier segments are 15 feet deep, drop-in segments are 8'-0" deep, and end segments are 6'-9" deep.

The Bayou LaFourche Channel Unit is a three span unit, with spans of 260 feet, 350 feet and 260 feet respectively. Pier segments are 150' in length, and vary in depth from 8'-6" at inflection points to 15 feet deep over the piers. The pier segment weight is minimized through use of a variable depth web. Drop-in segments are 196' in length. End segments are 183'-3" in length. End and drop-in segments are a constant 8'-6" deep. The girder used is a modification of the FBT-78. The web was thickened from 7" to 9" to accommodate post-tensioning tendons, and the resulting top flange width is 5'-2".

A comparison of segment geometry and segment weights for the three bridges referenced above is shown in *Figure 2*.

Segment weight can be a significant factor since existing precast facilities may be (and probably are) limited by the capacity of their straddle carriers, cranes and/or modes of transportation to the project site(s). Though we investigated the use of light weight aggregate, specifically Stalite® Shale aggregate, normal weight concrete is being used for the segments at the Bayou LaFourche Site.

Light weight aggregate offers two significant advantages – lighter segments for handling, transport, and erection equipment, and reduced loads on pier segments during critical erection steps prior to post-tensioning. The author believes that the advantages of light weight concrete for design and construction of long span spliced girder bridges warrant serious consideration of light weight aggregate. Higher Ultimate Creep and Shrinkage Coefficients and a lower Modulus of Elasticity are parameters that may detract from the advantages of higher strength to weight ratios. These parameters will have to be considered carefully.

High Performance Concrete has been specified for the girder segments and closure pours, i.e., high strength concrete with a 28-day compressive strength of 10,000 psi. Note that we could have used a lower strength concrete – but would have had to modify the segment geometry to do so, increasing segment weight and complicating forming requirements. The use of high strength concrete simplifies forming requirements and increases segment ductility. The St. George Island project, by comparison, used concrete with a 28-day strength of 8,500 psi.

The essential computations for the LA-1 Main Channel Unit are outlined below:

- **Sequence of Erection**
- **Time Dependent Analysis (ADAPT)**
- **Stability (During Erection)**
- **End Zone Reinforcement – Strut and Tie Model**
- **Anchor Zone Reinforcement (End Blocks)**
- **Stress Checks**
- **Strength Design**
    - **Flexure**
    - **Shear**
- **Bearing Pad Design**
- **Expansion Joint Movement**
- **Diaphragm Design**
- **Deck Design (Substructure Interaction)**
- **Camber, Deflection and Build-up**

Each of these calculations provides some insight into the problems tackled during design. Using the outline above, several key elements of the design process will be highlighted.

ERECTION SEQUENCE

A detailed sequence of erection is a necessary precursor to the time-dependent analysis. Erection requirements will influence the type of temporary shoring system used to assemble the segments prior to post tensioning, and the shoring system will dictate the loads to be carried by the precast elements. Prior to first stage post-tensioning the pier segment must carry the weight of the drop-in. This stage of erection is critical, since the pier segment must be reinforced/prestressed to handle the cantilevered load imposed by the drop-ins, or temporary shoring must be designed to relieve the load on the pier segment. As segments become longer and heavier, the problem is aggravated. The shoring system modeled for analysis of the Bayou LaFourche spliced girder system is shown in *Figure 3*. The prestress in the pier segment is sufficient for the weight of the drop-in segment, combined with anticipated construction loads. These loads include strongbacks, erection bracing, and a nominal 20 psf construction load. The flexural demand to capacity ratio is 1.0 at this stage of construction, using the temporary shoring shown in *Figure 3*.

The St. George Island shoring system, shown in *Figure 4*, has a couple of advantages over the shoring proposed in *Figure 3*: First, the cantilever is reduced, improving the flexural demand to capacity ratio of the pier segment, and the shoring is located on the pile caps, eliminating the need to drive and extract temporary piling beneath the superstructure.

TIME DEPENDENT ANALYSIS

Time dependent analysis can be done using any of a number of commercially available software packages. ADAPT was used for analysis of the Bayou LaFourche channel unit. The erection analysis is divided into two sections: The first contains the geometry of the completed structure, including all members, tendons, temporary supports, stays, and associated material and section properties – any structural member that is ever used in the construction of the bridge is included. Creep and shrinkage coefficients, and the associated creep and shrinkage model used to accumulate long term effects are also specified. The second section contains the erection steps in chronological order, from casting the segments and release of initial prestress, to erection of the segments onto shoring towers and piers (complete with strongbacks and tiedowns) to casting of the deck, stressing of the tendons, and removal of temporary shoring members. Time intervals between each step are input as well, and events must be staged accurately (e.g., three days between segment casting and release of prestress, etc.). Support conditions are revised as necessary with each stage of construction.

The erection analysis provides all loads and reactions, shears and moments, fiber stresses and estimates of camber and deflection for each step of construction, and throughout the service life of the bridge.

Subsequent to running the erection analysis, a live load analysis is performed. Results of the live load analysis are combined with the results of the erection analysis, usually at the beginning of the structures service life, and at a point in time (generally 10,000 days from start of construction) when all long term losses have accumulated.

Ideally, the time dependent analysis combines a realistic assessment of the precaster's fabrication schedule, with a reasonable construction schedule proposed for erection of the bridge, and foreknowledge of.the temporary shoring system likely to be used during erection.

STABILITY

Stability of the girder segments for handling and erection becomes more critical as members become longer. To provide stability during handling and erection of the drop-in and end segments of the Bayou LaFourche channel unit, we elected not to try and move the pick points inward. End stresses quickly become unmanageable. Instead an erection truss has been sized to inhibit bowing, and stiffen the top flange. See *Figure 5*. It can be used in the casting yard and in the field for erection. It is not necessary to provide each segment with it's own truss, only a sufficient number to allow handling until segments can be cross-braced to each other, and clamped to the pier segments. The erection truss was designed to satisfy Equation 5.2.2 presented in the PCI Design Handbook, by increasing the lateral stiffness of the member. Satisfaction of Equation 5.2.2 guards against lateral roll (Chapter Five – Product Handling and Erection Bridge), a phenomena known to occur when direct sunlight causes the girder to bow laterally, causing the c.g. of the member to shift.

To inhibit drop-in and end segments from rolling once they are attached to the pier segments via strongbacks, clamps are attached at the base of the web to prevent out-of-plane rotations. Critical lateral buckling loads/moments increase substantially when these boundary conditions are imposed. Note that the Author does not recommend that the strongbacks attached to the drop-ins and end segments bear on crush blocks or elastomeric pads to distribute the bearing reactions. Strongbacks as clamped to the segments using prestressed rods. Should the blocks crush or elastomeric pads deform (plastic flow), the clamping force will be relieved, and the strongbacks will be ineffective in preventing incipient roll.

The Sequence of Erection requires that cross frames be installed as soon possible, i.e, as soon as there is a pair of girders positioned on their bearings or clamped to the pier segments. Lifting frames can be removed as soon as cross frames are installed. The cross frames have been designed for permanent installation. They are nested between precast girders a minimum of 65 feet over water, and covered by the deck. Consequently, with a proper painting system they should not be highly susceptible to corrosion.

One last note: The Sequence of Erection requires that pier diaphragms be cast and post-tensioned, and cross bracing installed between pier segments before drop-in segments are placed. This is intended to stiffen the assembly against racking from wind and incidental construction loads.

END ZONE REINFORCEMENT

The stress distribution in the ends of prestressed members is a superposition of the prestress force as it is developed and distributed to the girder over the transfer length of the strand, and the bearing reactions distributed to the girder in the same reach of the beam. The resulting disturbed region is about as long as the girder is deep (St Venant Theory). Strut and Tie modeling of this region is appropriate.

The choice of strut and tie mechanism is not arbitrary. Though it is possible to devise a number of strut and tie mechanisms to resist applied loads, a strut and tie model that follows the path of principle compressive and tensile stresses will provide the 'best fit'.
A simple finite element model of a simple span girder can provide both the direction and magnitude of principle tension and compression. Orienting struts along the compressive principle stresses and ties along the path of principle tension, it is possible to size tensile reinforcement to resist applied loads, and quantify the compression in the struts.

The flows of principle tensile and compressive stresses in a simply supported concrete girder are shown in *Figure 6*. The top figures show the action of prestress on the member, while the bottom figures show the action of gravity loads.

A relatively simple wheel and spoke strut and tie model is appropriate for analysis of the gravity forces to be resisted by mild steel reinforcement. The struts are chords around the rim, and tension ties are spokes that converge to a point on the centroid of the prestressing. The force distribution shows that the tension has both horizontal and vertical components. At the top of the girder the strut is virtually horizontal, equal and opposite to the internal compressive stress resultant. At the base of the girder the strut is virtually vertical - equal and opposite to the bearing reaction.

Superimpose on the gravity model the edge and bursting tension forces due to the applied prestress. Size end zone rebar, both longitudinal and transverse bars, for the resulting forces/stresses. Equations for approximate stress analysis for applied prestress forces (edge tension and bursting forces) are contained in Sections 5.10.9.3 through 5.10.9.6 of the LRFD Code. Though intended for post-tensioned anchorages, they consider the same forces that develop in prestressed components. Alternatively a finite element model will provide the direction and magnitude of principle compressive and tensile stresses. A strut and tie model can be fashioned that properly distributes the flow of principle stresses from the applied prestress.

ANCHOR ZONE REINFORCEMENT

The details of tendon anchorage, as detailed for the Bayou Lafourche Channel Unit, require that first stage post-tensioning be completed before the girders in the adjacent channel units are set on their permanent bearings. The anchor zone details are essentially identical to those used on the St. George Island project. On the St. George Island project four of the five adjacent girders were set prior to first stage post-tensioning, but were offset from the permanent bearings. This provided a working platform for pulling tendons through the girders. Once the first stage tendons are pulled and stressed, the adjacent girders can be relocated to their permanent bearings. Second stage post-tensioning is done from the deck of the adjacent spans.

The convenience of pulling all tendons from the top of the girders introduces anchorage details that are significantly more complicated, and in the Author's opinion generally unwarranted.

The equations contained in Sections 5.10.9.3 through 5.10.9.6 of the LRFD Code were used to size the reinforcement in the anchor zone. Implementing the equations will not be discussed at length. However, it is necessary to review the stresses in the anchor zone for each step of the post-tensioning process: Review the state of stress after the first tendon is stressed, the second and so on. The highest bursting tension occurs when the second or third tendon is pulled. Edge forces are also critical for an intermediate step.

Tendon grouting has become a critical step in the post-tensioning process. The State of Florida has inspected a number of their segmental bridges, and found that a large percentage of the tendon anchorages have experienced rapid corrosion due to water and air intrusion into voids present in the tension anchorages. They found that many strands corroded so severely in the first 5 to 10 years of service that they were essentially eaten away. They have implemented a multi-step process for grouting, post-grouting anchorage inspection, and subsequent protection of the anchorage to remedy the problem. No distinction between segmental boxes and spliced girders was made.

From my perspective segmental boxes are much more at risk to corrosion problems than spliced girders, due in part to the lower level of redundancy in a segmental box superstructure (one box with six to eight tendons vs. four or five girders with three or four tendons each). Tendons in spliced girders are bonded from one end to the other, whereas many segmental bridges use unbonded external tendons (span by span construction). If a tendon breaks in an externally post-tensioned box, the strength provided by that tendon is lost the full length of the tendon. The sole function of grout in the externally post-tensioned box is protection of the strand.

STRESS CHECKS

The graphic post-processor in ADAPT (time dependent analysis) provides a step by step (each step of erection) display of flexural stress in the spliced girder system. Stress checks provide a very useful tool for review of the analysis and design. In general, if stresses remain within allowable limits, strength design will be satisfied. They also provide a graphic display of the time dependent effects of differential shrinkage and creep induced camber.

An example of the utility of stress checks is seen in *Figures 7 through 9*.

Handling stresses can be visualized in *Figure 7.* The Bayou LaFourche pier segment is shown with two different sets of support conditions: The left pier segment supports (pick points) are 34 feet apart, and the right pier segment is shown with support points 63 feet apart. It is apparent that there is a limit to the spread between pick points on the pier segment. We set the limit at 60 feet apart to prevent tensile cracks in the bottom of the segment during handling. We also reinforced the bottom of the segment with sufficient mild steel rebar to arrest cracking in the event that impact/shock loading caused the stresses to spike to Modulus of Rupture.

Erection Stresses on the pier segment are shown in *Figures 8 and 9.* The pier segment is sitting on the pier in *Figure 8,* supporting only segment's self weight. Prestress in the top flange results in tension in the bottom of the segment. In *Figure 9* the pier segment is supporting the weight of the drop-in segment prior to first stage post-tensioning. There is a complete stress reversal in the segment, and the top flange stresses are approaching the tensile strength of the concrete.

STRENGTH DESIGN

The 15 foot deep pier segment presents several unique design problems. To minimize the segment weight the web depth is variable and bottom flange depth is constant. Over the pier the bottom flange abruptly transitions to a much larger section. This is not done to satisfy flexural stress or strength design, but to provide a more massive section directly over the pier bearings (in a disturbed region with high stress/strain gradients). It also provides additional section to resist the potential transfer of ship impact from the pier to the superstructure. As it turns out, it also helps ensure ductile behavior.

Flexural strength calculations in the negative moment region indicate that the concrete strength is critical in deep narrow haunched sections. To ensure that the failure mode is ductile yielding of the tensile steel rather than crushing of the concrete, the ratio c/d must be less than .46 (c/d is the ratio of two dimensions: the neutral axis to extreme compression fiber (c) divided by the tensile reinforcement centroid to the extreme compression fiber (d)). To ensure that this criterion is satisfied, high strength concrete can be specified. Alternatively, the bottom flange depth has to be increased. We elected to use high performance/high strength concrete. The section is lighter, and other benefits are derived from use of high performance concrete (durability, tensile strength, etc.) The section remains ductile since the flexural steel will begin to yield if the moment exceeds the nominal strength of the section over the piers. As the steel yields, moment will be shed to the positive bending region (moment redistribution). This is not to say we designed for moment redistribution, we did not. But if overloaded the system should remain ductile.

Near the closure pour it is practically impossible to impose the condition that c/d be less than .46 (for negative bending), but a healthy reserve capacity exists, and stress redistribution over the pier ensures continued ductility of the girder system, since well before the negative moment increases over the closure pour the inflection point will shift towards the pier. *Figure 10* provides a summary of the demand to capacity ratios throughout the pier segments, as well as the c/d ratios along the length of the segment.

Shear design provides no real surprises or special design considerations. Parabolic post-tensioning tendons, aside from precompressing the section (thus reducing principle tension everywhere) carry a considerable amount of the gravity loads. The equivalent uniform load $\omega = 8F\delta/l^2$ due to the prestress force F can be deducted from the shear carried by the concrete. This is a sizeable reduction of the shear force on the section. One reason the Author avoids terminating tendons at the top of the girder and away from the bearings is to carry the load borne by the tendons back to the support.

CAMBER, DEFLECTION AND BUILD-UP

This is probably not the most interesting aspect of design, but it cannot be overlooked. The camber diagram for the Channel Unit is shown in *Figure 11*. The diagram not only tabulates build-ups and provides deflection estimates to facilitate setting the deck screeds, it provides essential information for setting the elevations of the temporary shoring tower bearings. Once the closure pours are made the ability to modify the girder geometry is severely limited. The beam seat elevations are critical.

There are five segments on each continuous girder line, and each segment is set so that the end points, after adjustment for camber and deflection, will fall on the vertical curve (shifted down to account for the thickness of the deck and build-up). Note that the girder segments for the Bayou LaFourche channel unit are tied down to the temporary shoring tower, but will rise approximately 1 ½ inches after first stage post-tensioning. This rise is anticipated when setting the temporary shoring tower elevation.

The deflection adjustment tabulated on the plans is the deflection anticipated to occur due to placement of the deck and subsequent second stage post-tensioning. It is the deflection that occurs subsequent to the 1 ½ inch rise after release of the tie downs.

Invariably, the camber adjustments tabulated on the diagram will have to revised. Measured camber in deep bulb tees is almost always considerably less than the estimated/calculated camber. The deviation will affect the tabulated build-ups, and can cause the required temporary shoring tower beam seat elevations to change. It is necessary to note on the plans that the camber measured in the segments delivered to the field must be checked against the camber estimated in the table, and adjustments made prior to setting girder segments.

STRETCHING THE SYSTEM FURTHER – IS IT PRACTICAL?

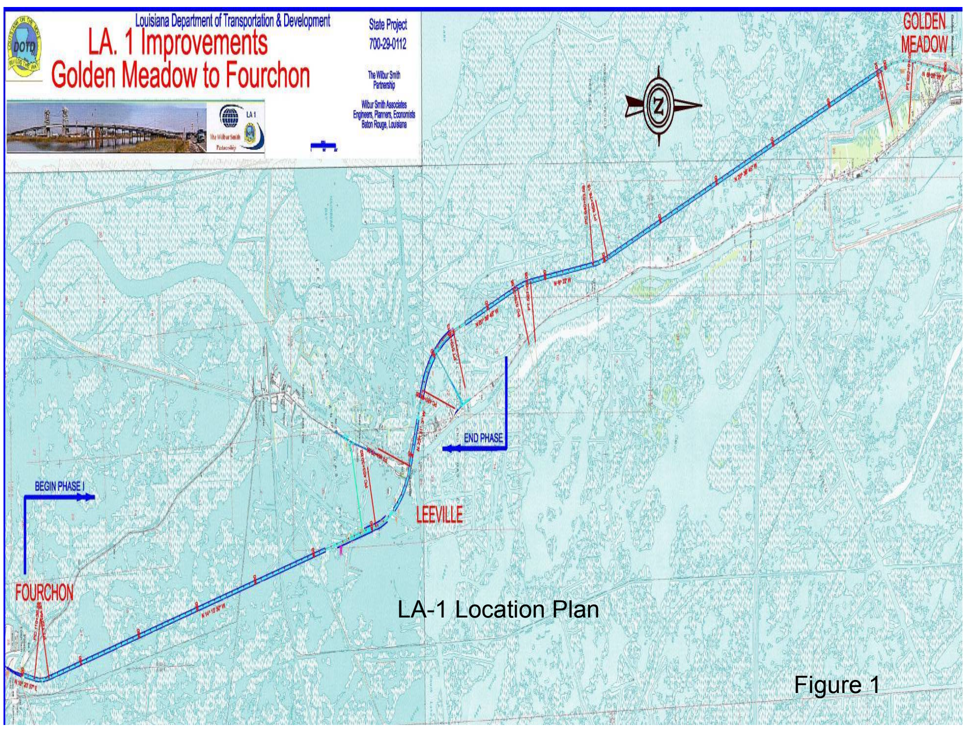
The Author believes that a 400 foot span is feasible - and practical - using segments no heavier than those sized for the 350 foot unit. The approach to design and segment fabrication however, involves a significant departure from current practice. Stretching the length of the pier segments may be the key.

At present the forming, handling and transport of the pier segments is the greatest impediment to use of the system away from navigable waterways, and to increasing spliced girder spans over waterways. The pier segment is generally the critical segment in terms of it's weight, depth, and strength requirements during erection.

To span 400 feet, the pier segment will likely need to be lengthened to 196 or 200 feet – similar to the length of the drop-in segment on the Bayou Fourche project. At this length, and assuming the depth will need to be 15 feet or greater, the segment becomes unwieldy if precast in a single pour, and prestressed 15 to 20 feet up in the air (the top flange is prestressed).

A hybrid, composite precast/steel girder system appears to circumvent many of the problems we currently deal with: large amounts of prestress in the top flange, heavy deep members increasingly difficult to transport, and large moments due to cantilevered end and drop-in segments that are increasingly heavy themselves.

The hybrid composite system proposed by the Author consists of casting a standard precast section, and attaching it to a steel haunch in the field. The completed structure is shown in *Figure 12*. The fabrication details are proprietary, but analysis indicates that the system may not only allow longer spans, but conceivably will allow use of the system for shorter spans in areas where access is currently limited by segment weight or height restrictions.

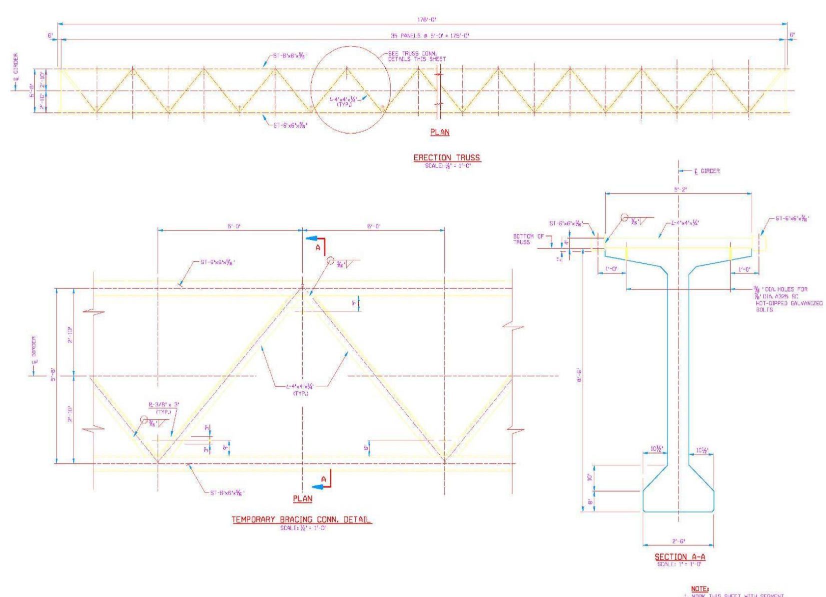


LA-1 Location Plan

Figure 1

Three Span Channel Unit at Leeville, LA.
350 ft. Channel Span

Figure 1



Erection Truss

Figure 2

Comparison of Splice Girder Designs

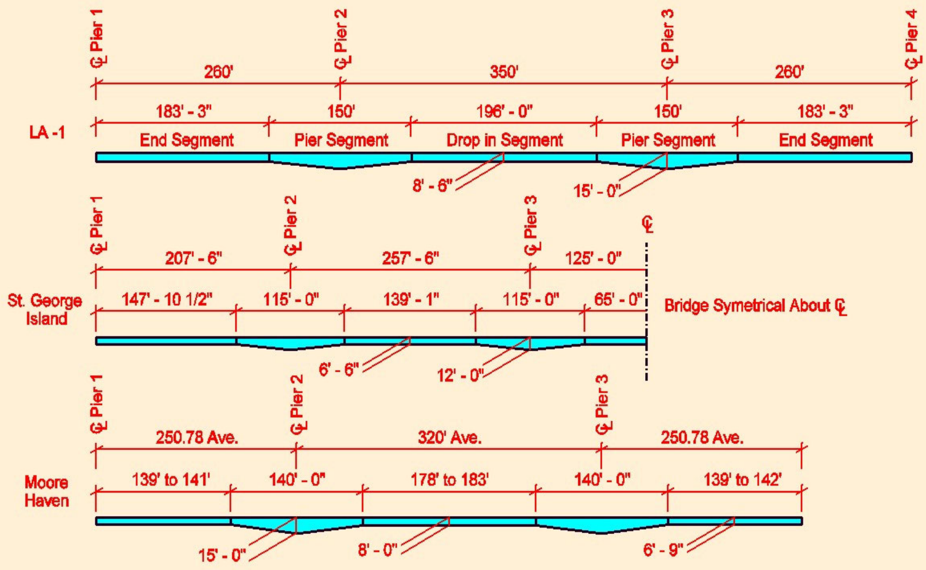


Figure 2

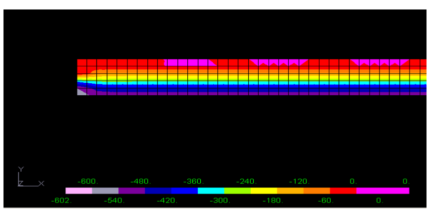
Comparison of Girder Segment Weights			
	St, George Island	Moore Haven	LA-1
End Segment	207.4 K	207.12 K	292.6 K
Drop - in Segment	184.5 K 174.5 K	248.4 K	292.2 K
Pier Segment	225.1 K	238.5 K	325 K

St George Island & LA-1 use a 9" Web;
Moore Haven uses an 8" Web

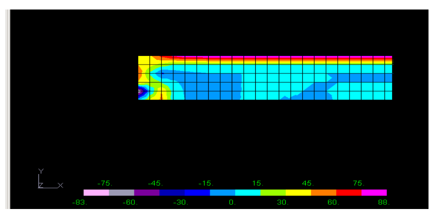
Figure 2


Figure 3
Temporary Shoring
(Moore Haven)

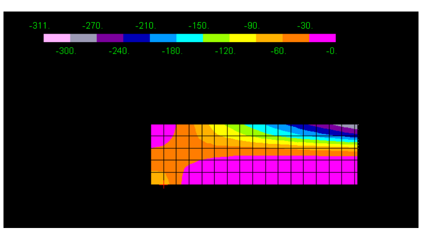
Figure 4



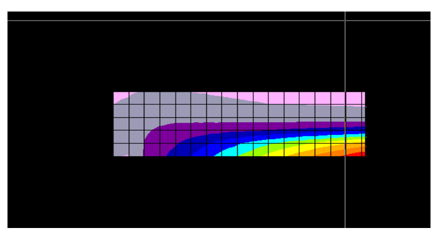
Principle Compressive Stress due to Prestress



Principle Tensile Stress due to Prestress



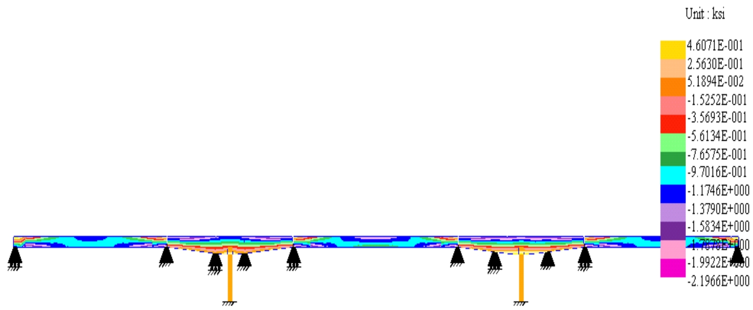
Principle Compressive Stress due to Gravity Loads



Principle Tensile Stress due to Gravity Loads

Figure 6

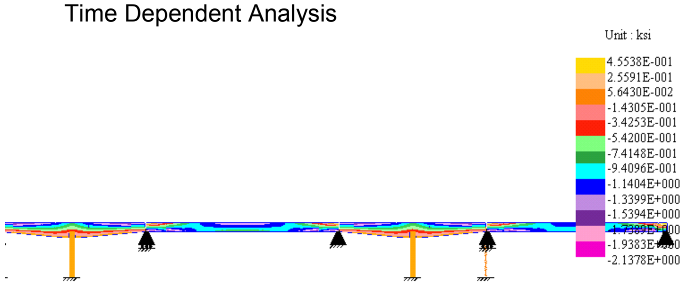
Time Dependent Analysis



Pier Segment Handling Stresses

Figure 7

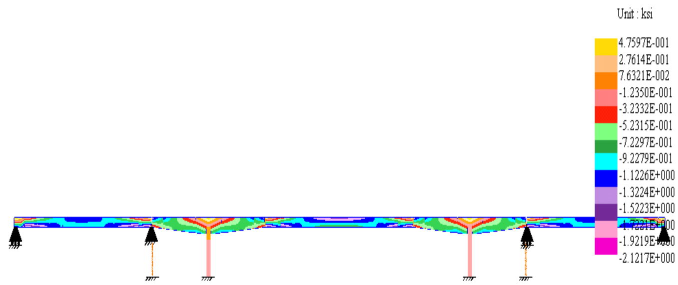
Time Dependent Analysis



Pier Segment Stress
Pier Segments resting on Permanent Bearings
Prior to setting Drop-In Segments

Figure 8

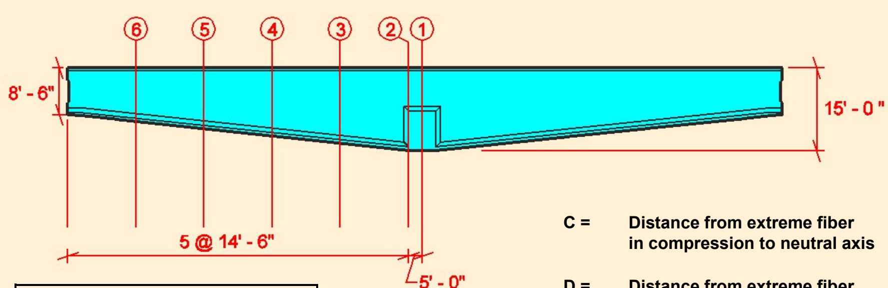
Time Dependent Analysis



Pier Segment Stress at
Erection of Drop-In Segment

Figure 9

Strength and Ductility



C = **Distance from extreme fiber in compression to neutral axis**

D = **Distance from extreme fiber in compression to centroid of tensile reinforcement**

C/D > 0.46 Brittle Failure by crushing of the concrete

Pier Strength and Ductility

Section	M_u	$ØM_n$	M_u / $ØM_n$	C/D
1	53710	56160	0.956	0.166
2	51380	53060	0.968	0.342
3	39340	45370	0.867	0.387
4	29010	36450	0.796	0.457
5	20130	26110	0.771	0.553
6	13170	17510	0.752	0.694

Moments are in Kip-Ft

Figure 10

Camber Diagram
Figure 12

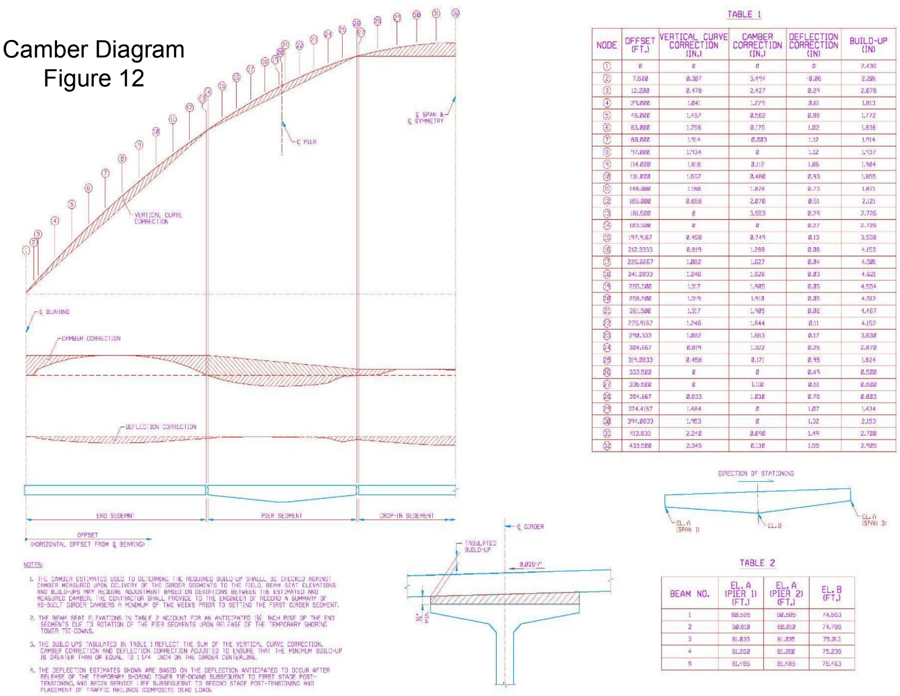
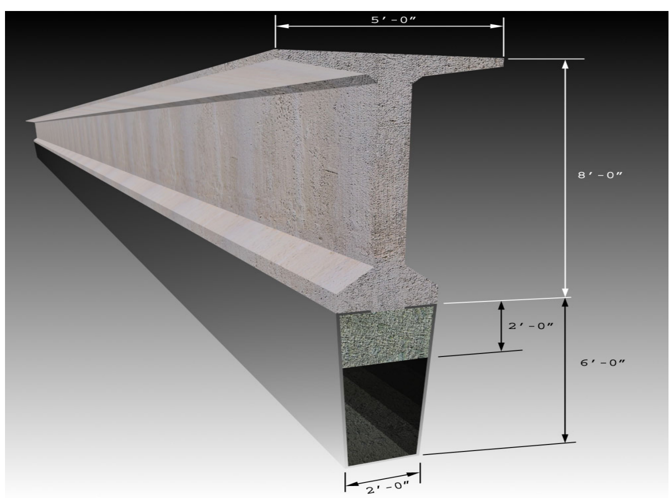


TABLE 1

NODE	OFFSET (FT.)	VERTICAL CURVE CORRECTION (IN.)	CAMBER CORRECTION (IN.)	DEFLECTION CORRECTION (IN)	BUILD-UP (IN)
1	0	0	0	0	2.436
2	7.520	0.307	3.494	-0.06	2.201
3	12.200	0.478	2.427	0.29	2.078
4	29.000	1.041	1.279	0.61	1.813
5	46.000	1.457	0.562	0.86	1.772
6	63.000	1.758	0.175	1.02	1.836
7	80.000	1.914	-0.003	1.12	1.914
8	97.000	1.934	0	1.12	1.937
9	114.000	1.818	0.112	1.06	1.904
10	131.000	1.567	0.480	0.93	1.855
11	148.000	1.180	1.078	0.73	1.871
12	165.000	0.658	2.070	0.51	2.121
13	181.500	0	3.553	0.29	2.726
14	183.500	0	0	0.27	2.726
15	197.9167	0.458	0.749	0.13	3.558
16	212.3333	0.819	1.288	0.08	4.153
17	226.6667	1.082	1.627	0.04	4.501
18	241.0833	1.248	1.826	0.03	4.621
19	255.500	1.317	1.905	0.05	4.554
20	258.500	1.319	1.910	0.05	4.512
21	261.500	1.317	1.905	0.06	4.467
22	275.9167	1.248	1.844	0.11	4.152
23	290.333	1.082	1.663	0.17	3.630
24	304.667	0.819	1.322	0.25	2.870
25	319.0833	0.458	0.171	0.95	1.824
26	333.500	0	0	0.49	0.520
27	336.500	0	1.110	0.51	0.520
28	354.667	0.833	1.030	0.78	0.003
29	374.4167	1.464	0	1.07	1.434
30	394.0833	1.953	0	1.32	2.153
31	413.833	2.240	0.090	1.49	2.720
32	433.500	2.345	0.130	1.55	2.905

TABLE 2

BEAM NO.	EL. A (PIER 1) (FT.)	EL. A (PIER 2) (FT.)	EL. B (FT.)
1	80.585	80.585	74.563
2	80.810	80.810	74.788
3	81.035	81.035	75.013
4	81.260	81.260	75.238
5	81.485	81.485	75.463



Hybrid Alternate

Figure 12