I-10 Bridge Across Lake Pontchartrain Twin Spans

CE 4460

Introduction to Bridge Design

Department of Civil & Environmental Engineering
Louisiana State University

Group 5

Megan Allain
Bryan Beyer
Paul Kocke
Anna Wheeler

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The bridge segment design outlined in this project is a simulation of the design process for the proposed bridge to replace the I-10 Lake Pontchartrain Bridge damaged by Hurricane Katrina in August 2005. The goal of this project was to model the actual bridge design process by closely adhering to the Louisiana Department of Transportation and Development (LaDOTD) specifications, the AASHTO LRFD Bridge Design code, and available LRFD design examples.

This Introduction section provides background information, details regarding the project scope and specifications, and special considerations taken for this particular design.

Project Background

The current I-10 bridge crossing Lake Pontchartrain, known locally as the "twin spans," was completed in December 1965. The $14.8 million bridge is 5.4 miles long, and is supported by prestressed concrete girders on prestressed concrete piles. It includes one 65-feet high elevated section to allow passage of maritime traffic. Along with 9.9 miles of interstate completed concurrently in 1965, the twin spans connected I-10 in New Orleans to I-59 in Slidell (*1*).

On August 29, 2006, Hurricane Katrina struck the New Orleans area as a Category 4 storm, inundating the twin spans with a storm surge of approximately 28 feet, which was unprecedented in the life of the bridge. The storm-induced flood currents in addition to the upward pressure of air trapped underneath the bridge dislodged, displaced, or submerged 435 bridge segments. Forty-seven days after the storm, the lesser-damaged eastbound span reopened to traffic. Missing or heavily damaged segments of the westbound span were replaced with a temporary hot-dipped galvanized steel truss with asphalt deck panels (*2*).

Figure 1. Damage to the Twin Spans

A replacement bridge is currently under design by the DOTD and Figg Engineering Group. While further specifications will be discussed later, the new bridge will be a two-span structure carrying three lanes of traffic each. The bridge will be either a prestressed concrete girder bridge (DOTD approach) or a segmental box girder bridge (Figg approach), and will be elevated sufficiently above the waterline to avoid failure from a Katrina-type storm surge (*3*).

Project Scope

The proposed bridge is two spans with each over 29,000 feet long. For a project of suitable length and difficulty, each group was required to choose 2-4 bridge segments and design them. To be as true as possible to the actual bridge, this project is based on a four-segment design. The proposed prestressed bridge is partitioned into four-segment continuous spans separated by finger joints and horizontally restrained by the center pile. The design section, Segments 46-49, can be seen in Figure 2 to the right, which is an excerpt from Sheet No. 107 of the preliminary design drawings issued by the DOTD. These particular segments were chosen because the level grade and absence of horizontal curves are conducive to a simpler design.

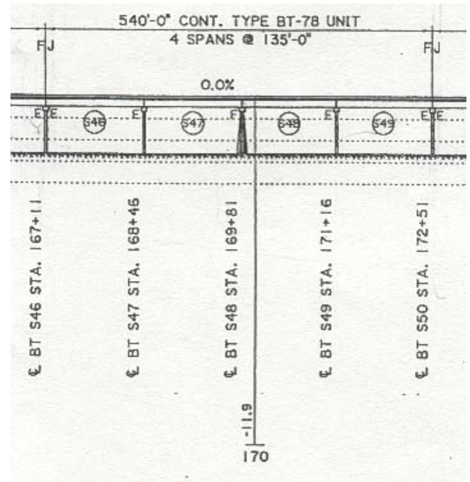


Figure 2. Design Section

Specifications

The specifications used for this project reference DOTD S.P. 450-17-0025, Revision 7 12/29/05, which are the design specifications released by the DOTD for use on the I-10 Bridge Over Lake Pontchartrain replacement project. The design methodology, design information, and design loads were followed as closely as possible.

For the deck and superstructure design, the LRFD Design Example for Steel Girder Superstructure Bridge was used as a reference. This document was prepared by Baker Engineering for the Federal Highway Administration and the National Highway Institute.

The third reference used was the AASHTO LRFD Bridge Design Specifications 3rd Edition. The version used was metric, so when the U.S. counterpart equation could not be found, the needed values were converted to metric, applied in the equation, and converted back U.S. units.

Special Considerations

By using steel girder, this project required some considerations not needed in a prestressed concrete girder design.

Bolted Field Splices

Concrete girders, whether prestressed or reinforced, have the advantage of being able to be cast onsite. Steel girders must be fabricated at a steel plant and shipped to the job site. While the Lake Pontchartrain bridge is located near large draft waterways, girder size is still a limiting factor. Field splices allow girders to be joined together midspan, but they must be rigorously designed against failure, and their design is included in this report.

Figure 3. Splice

Corrosion

Figure 4. Corrosion in Steel Beams

The main concern for steel girders placed in a marine environment is corrosion. Corrosion is the deterioration of a material's properties due to reactions with its environment (*4*). In addition to rusting caused by the wet conditions, the chlorides in the brackish water of Lake Pontchartrain can also be detrimental to the exposed steel girders. The most common preventative measures are painting and galvanization. Due to the problems associated with painting in a chloride-heavy environment and its questionable lifespan, this project will assume galvanization as the steel protection method.

The galvanization process is done in three major steps. First, prefabricated steel sections are prepared so that the cleaning chemicals and molten zinc can flow easily around and through it. All holes must be drilled and edges sanded. The second step is cleaning. Because zinc will only react with a very clean steel surface, the steel sections are dipped in a caustic mixture of cleaning chemicals to remove any surface impurities. The final step is the molten zinc bath. The thickness of the resulting zinc cover is a function of the thickness, roughness, chemistry, and design of the steel being galvanized. The size of the girder is a limiting factor in the galvanization process, which was another consideration when deciding on splice locations. However, if the kettle is too small to accommodate the entire steel girder, the galvanizer can galvanize one half of the girder, turn the girder around, and then galvanize the other half (*6*).

The lifetime of a zinc coating is dependent on the quality of the galvanization and the environment. For marine environments, the estimates of a good zinc coating range from 50 years to 100 years, with 70 years accepted as an average value (*6*).

DESIGN DRAWINGS

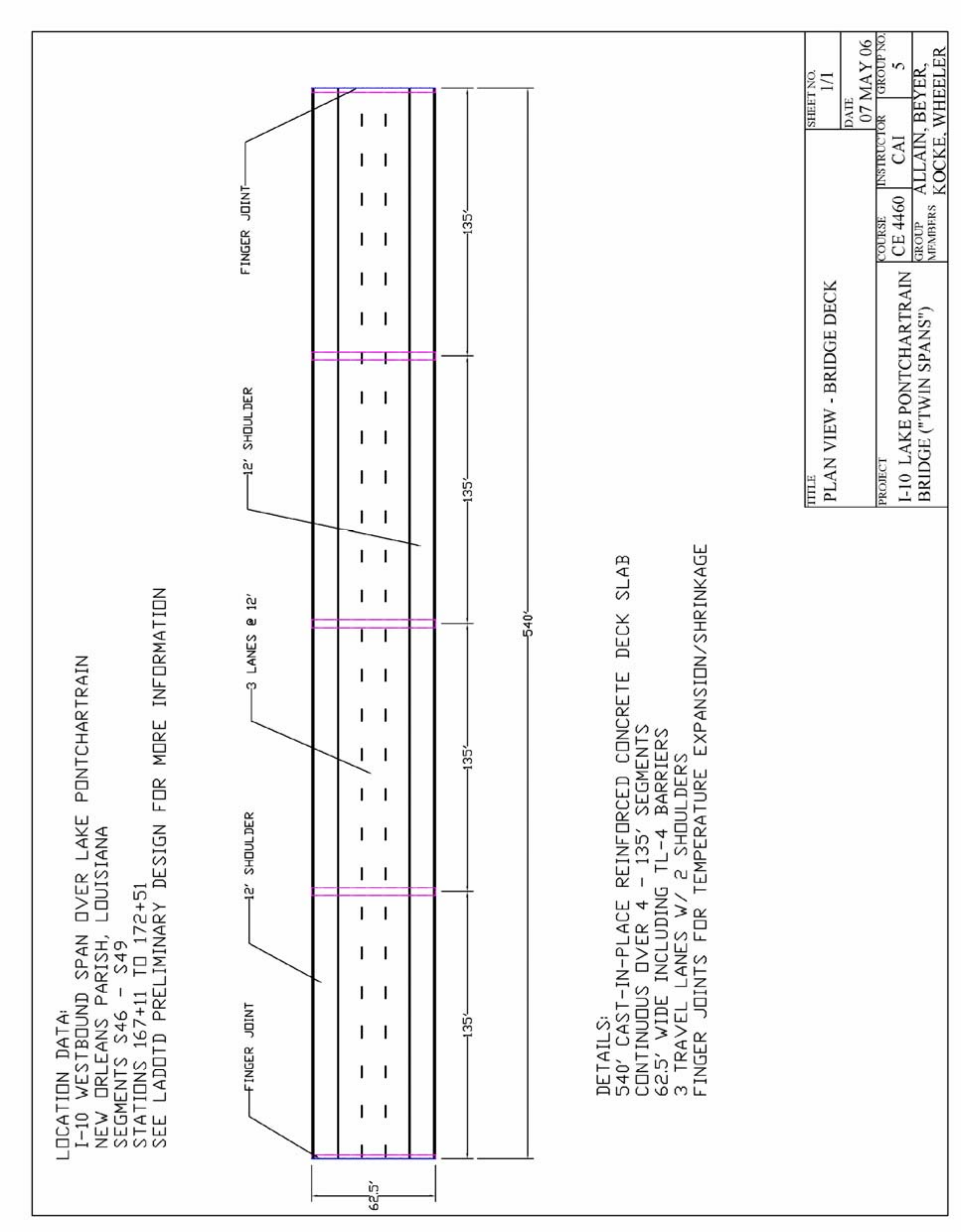


Figure 5. Plan View

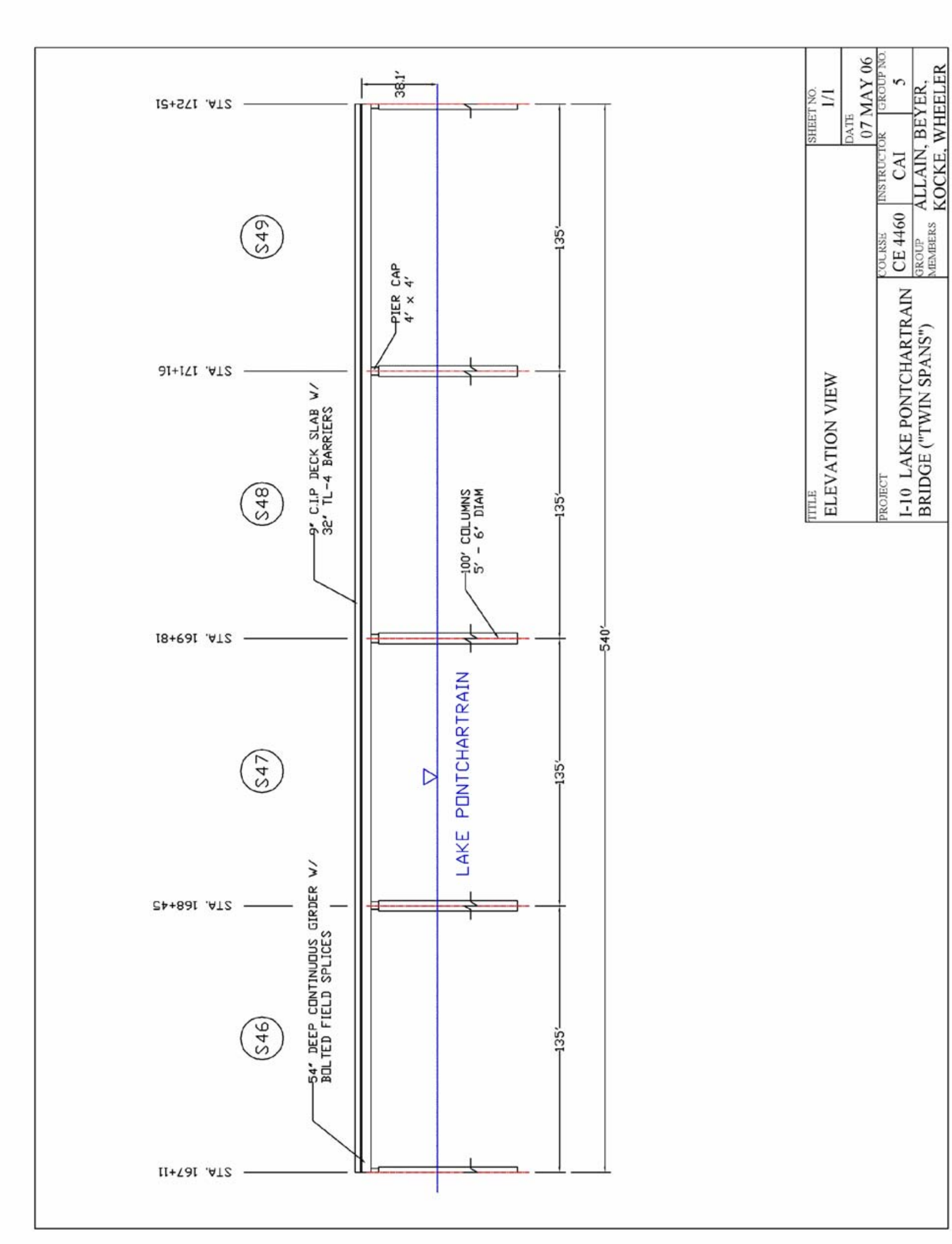


Figure 6. Elevation View

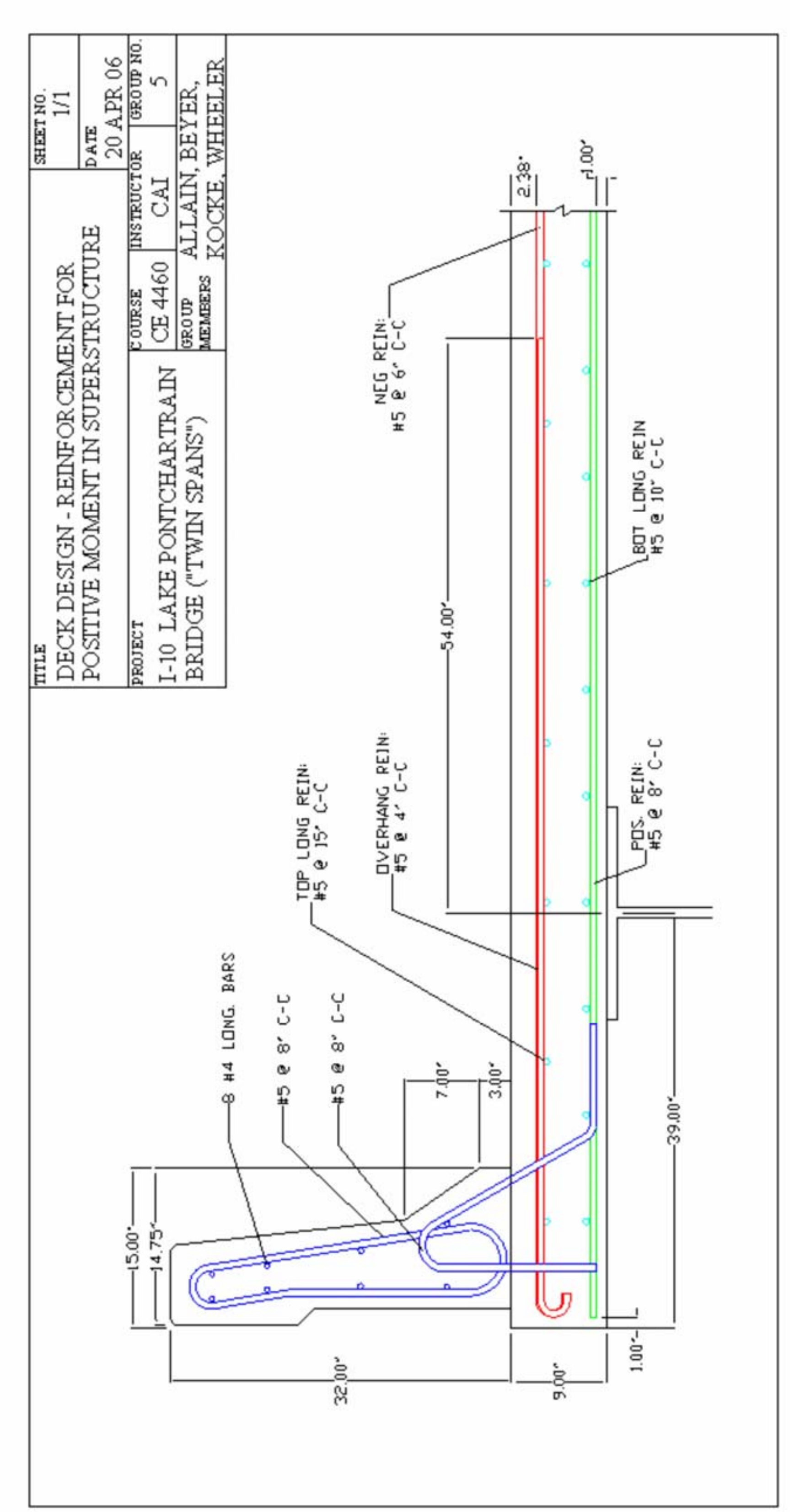


Figure 7. Bridge Deck Design – Positive Moment in Superstructure

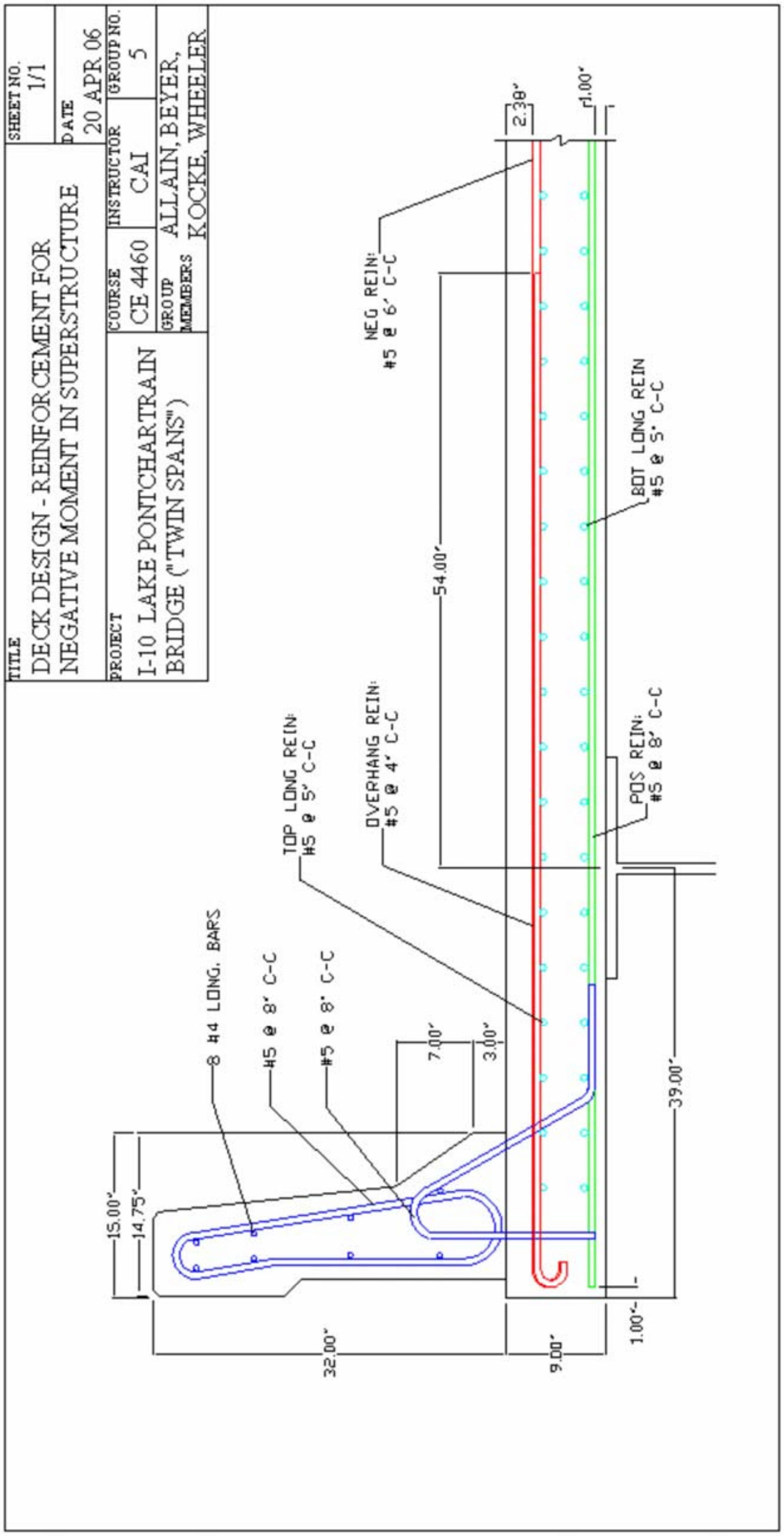


Figure 8. Bridge Deck Design – Negative Moment in Superstructure

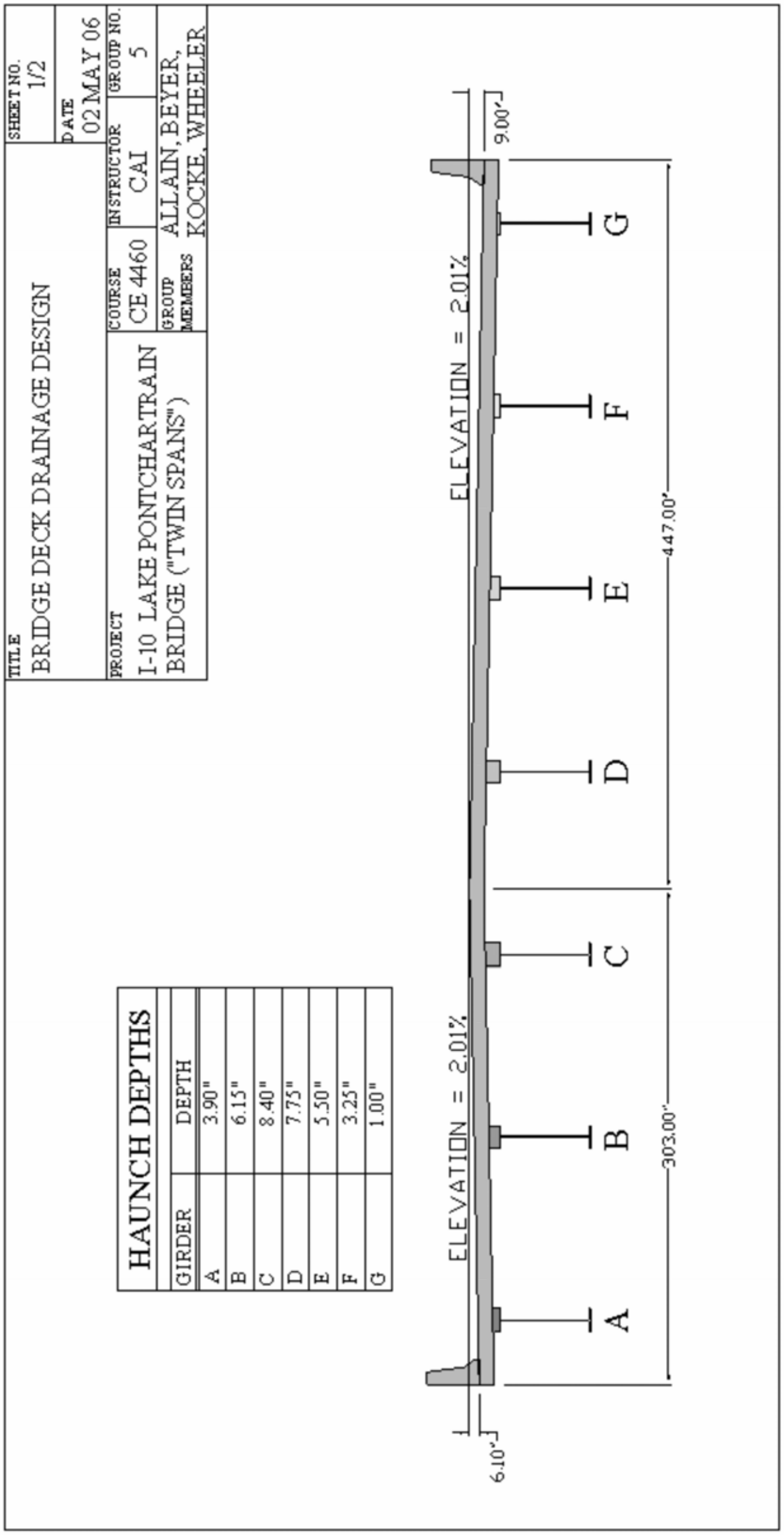


Figure 9. Bridge Deck Drainage Design

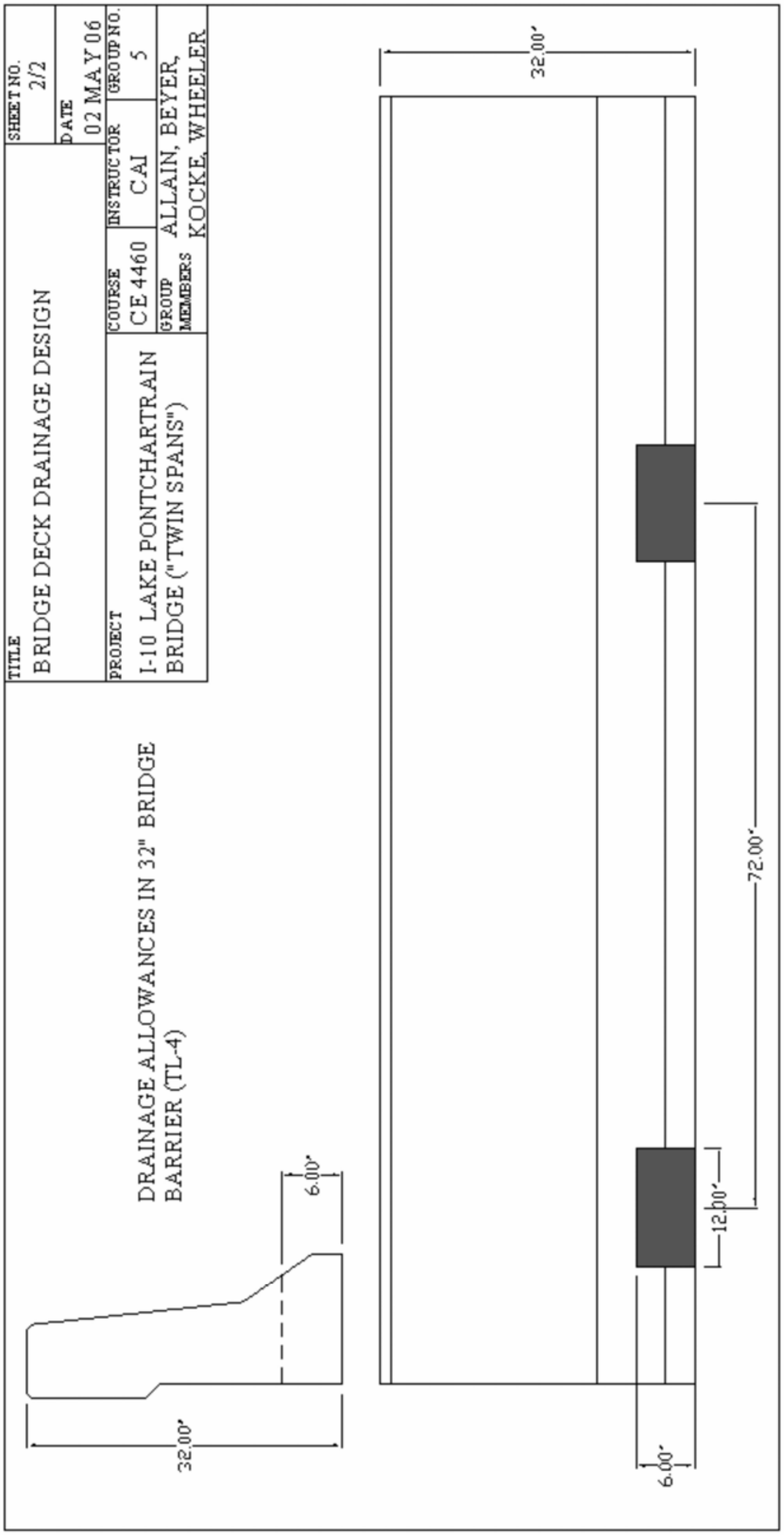


Figure 10. Bridge Deck Barrier Drainage Design

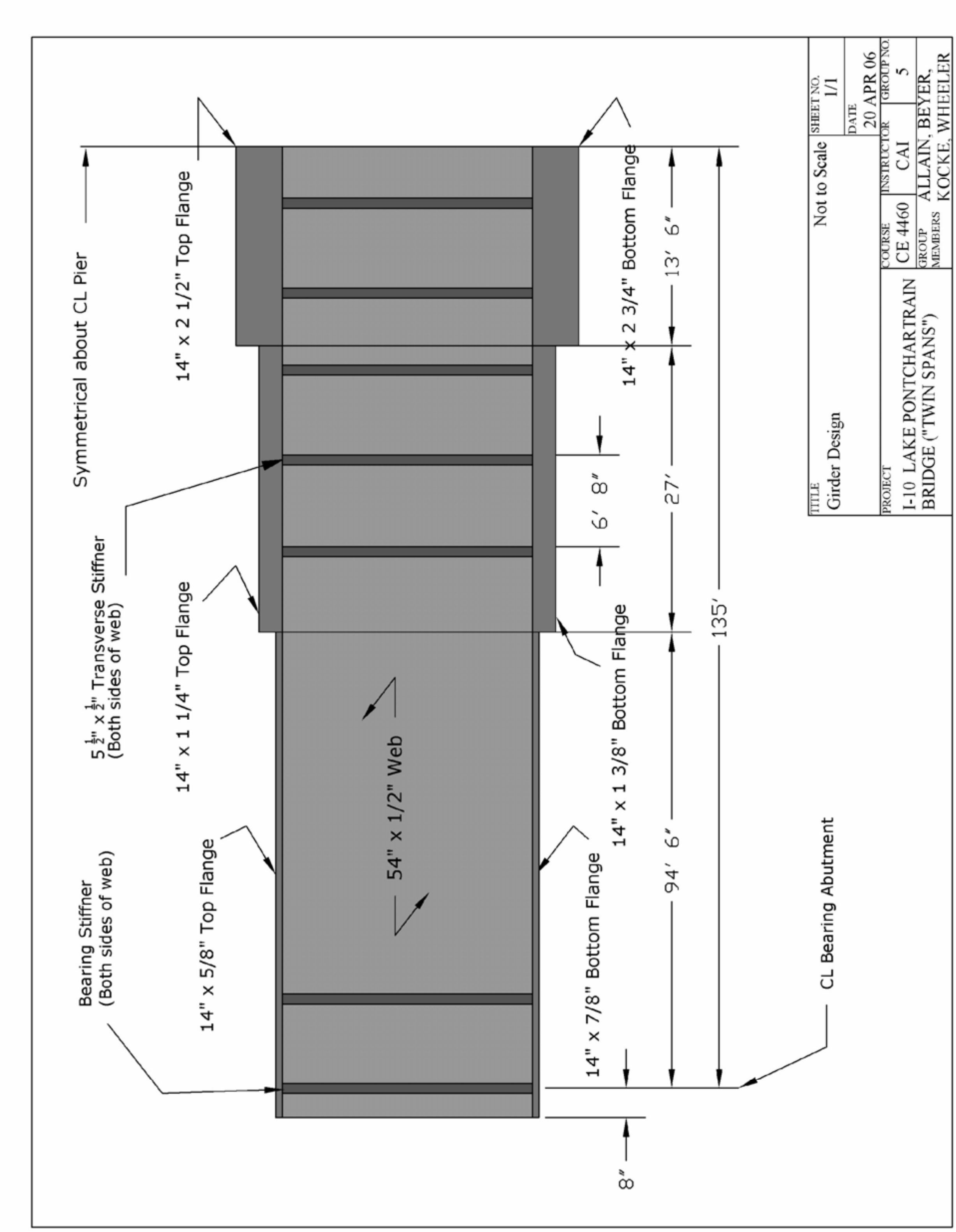


Figure 11. Girder Design

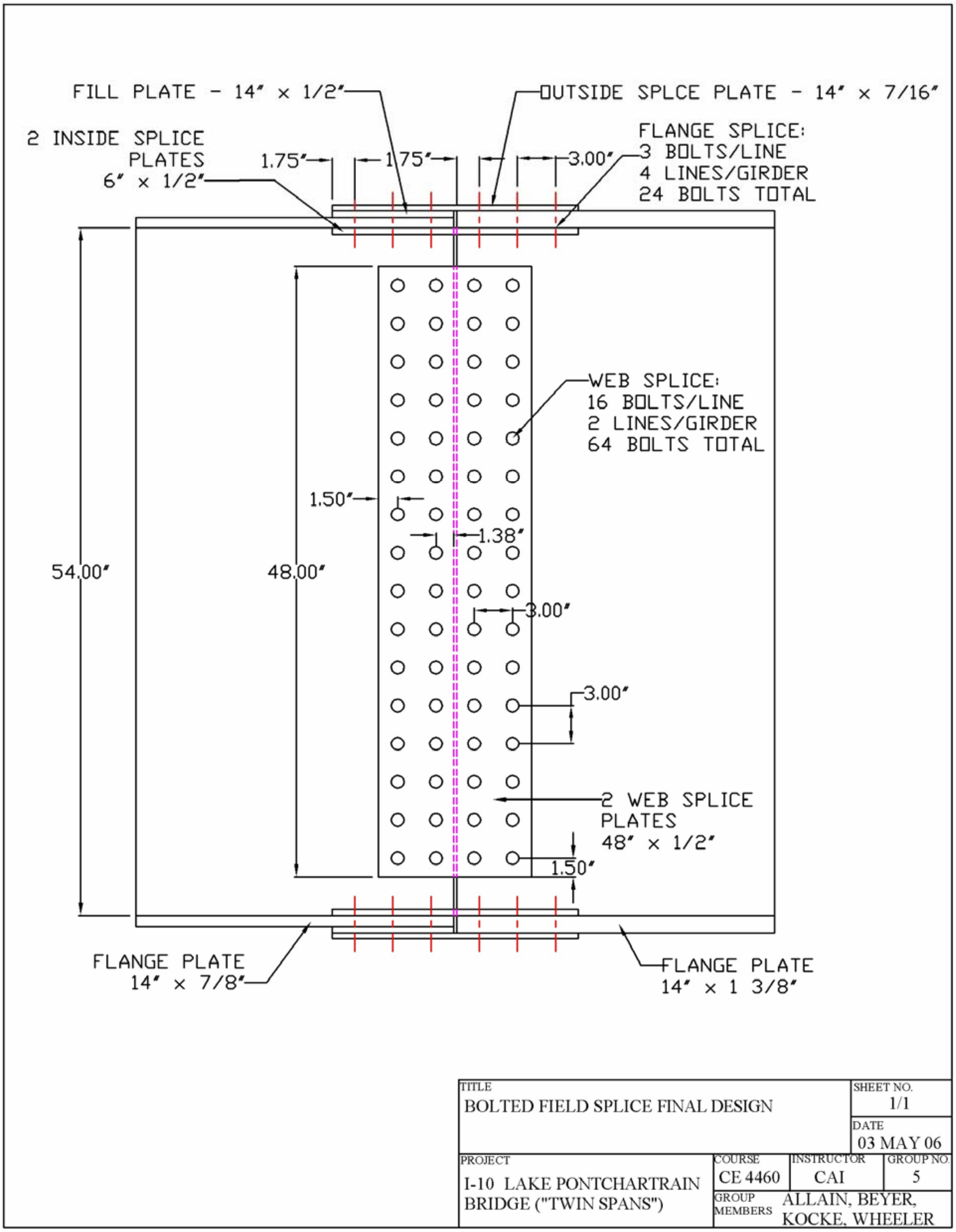


Figure 12. Field Splice Design

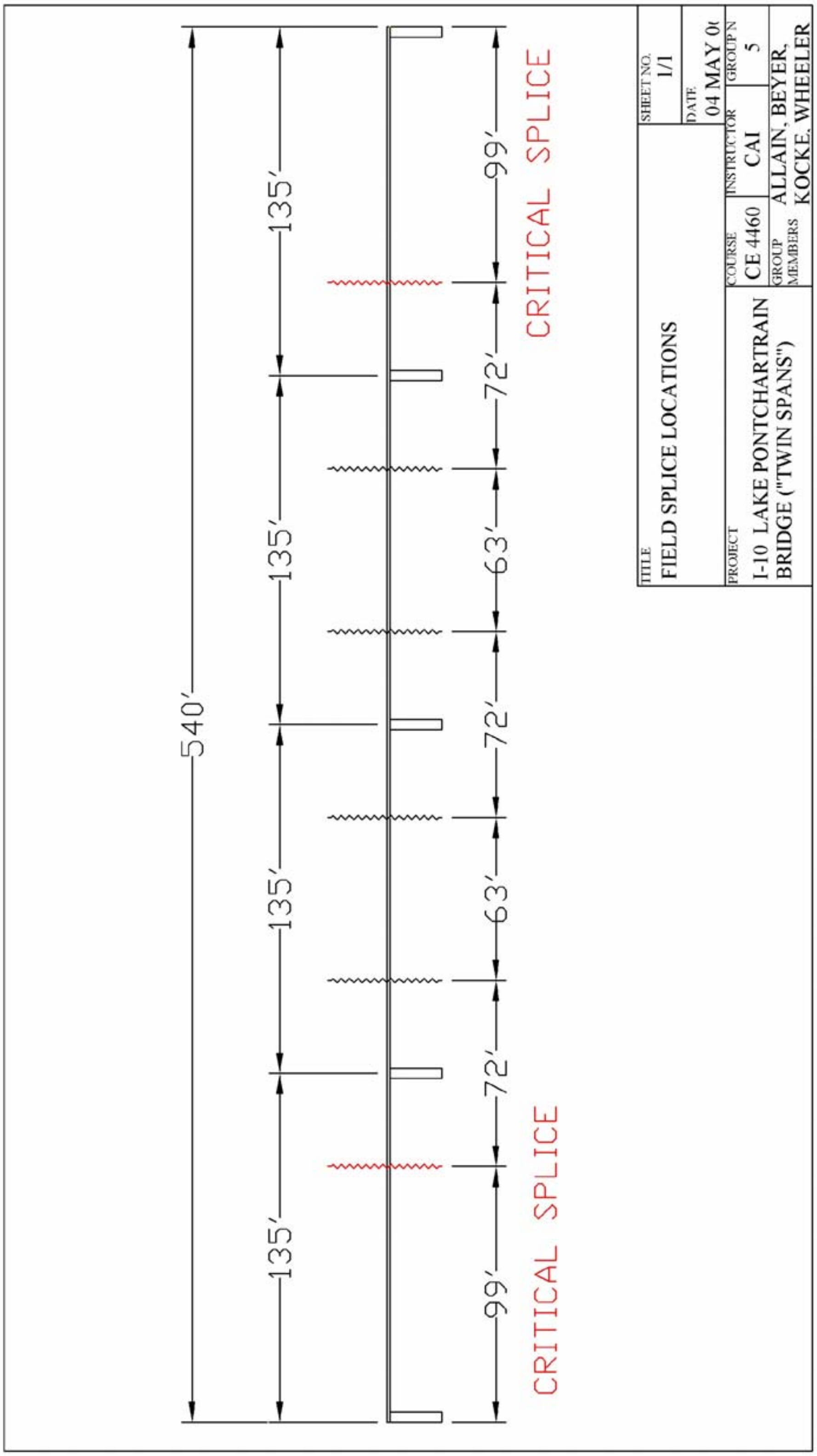


Figure 13. Field Splice Locations

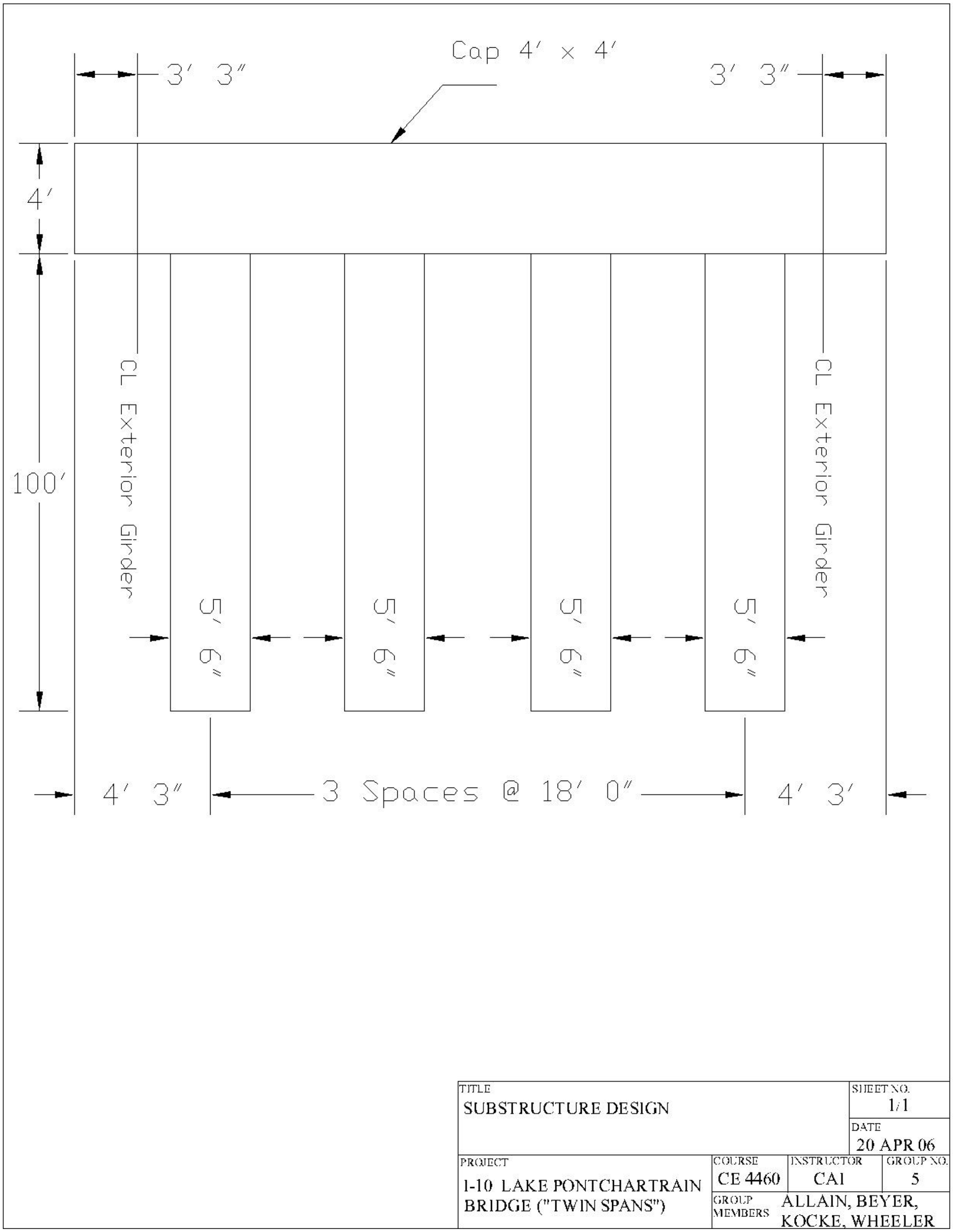


Figure 14. Substructure Design

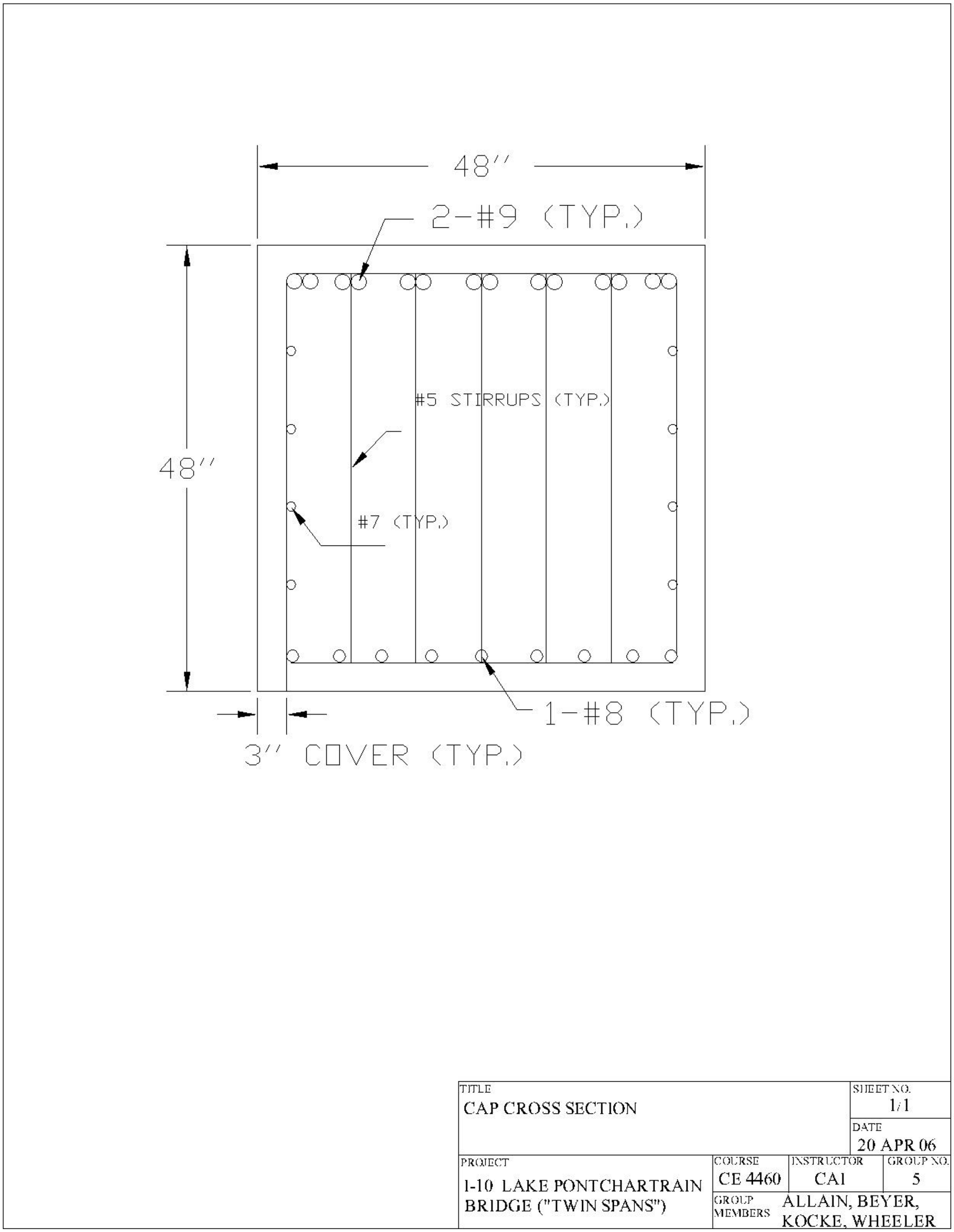


Figure 15. Cap Cross Section

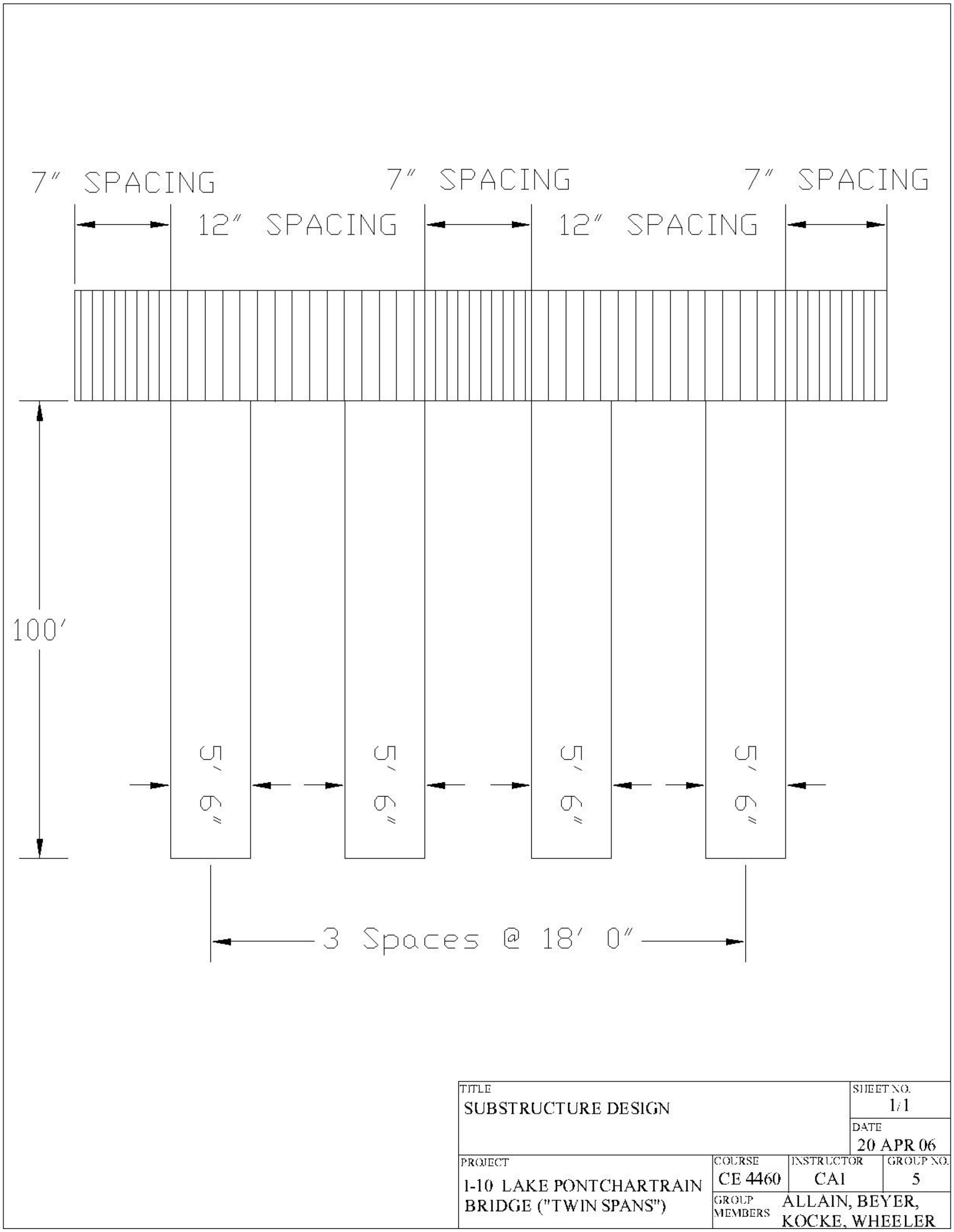
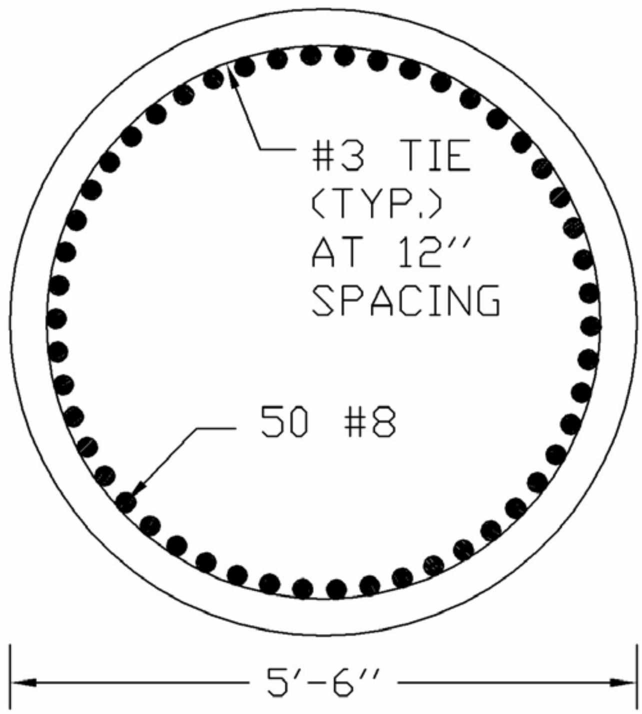


Figure 16. Cap Shear Reinforcement



TITLE PILE DESIGN--CROSS SECTION			SHEET NO. 1/1 DATE 7 MAY 06
PROJECT I-10 LAKE PONTCHARTRAIN BRIDGE ("TWIN SPANS")	COURSE CE 4460	INSTRUCTOR CAI	GROUP NO 5
	GROUP MEMBERS	ALLAIN, BEYER, KOCKE, WHEELER	

Figure 17. Pile Design

DECK DESIGN

The proposed bridge deck is a monolithic cast-in-place concrete slab. It is a uniform 9 inches thick and is 62.5 feet wide edge to edge. It is supported by 7 girders spaced at 9.33 feet with an overhang of 3.25 feet on either side. Each overhang supports a cast-in-place concrete F-shape barrier 32 inches high and rated TL-4. Drainage is provided by a cross slope of 2.01 percent, crowning between the inside travel lane and the middle travel lane, and drainage slots spaced at six feet in the concrete barriers at deck level.

This section reiterates the final design of the bridge deck and the more important steps of the process. The complete design process can be found in Appendix A.

Final Design

Reinforcement for Positive Moment in Substructure

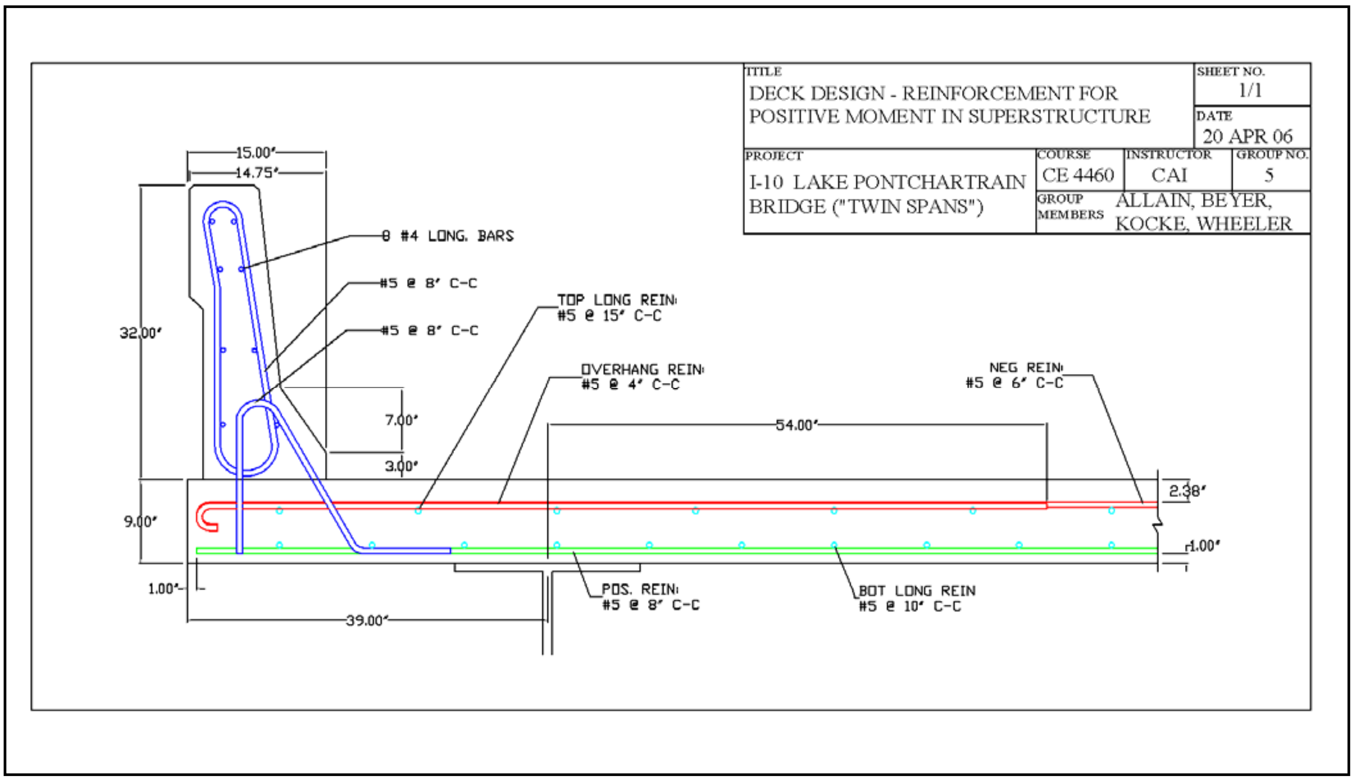


Figure 18. Deck Reinforcement for Positive Moment Substructure

Reinforcement for Negative Moment in Substructure

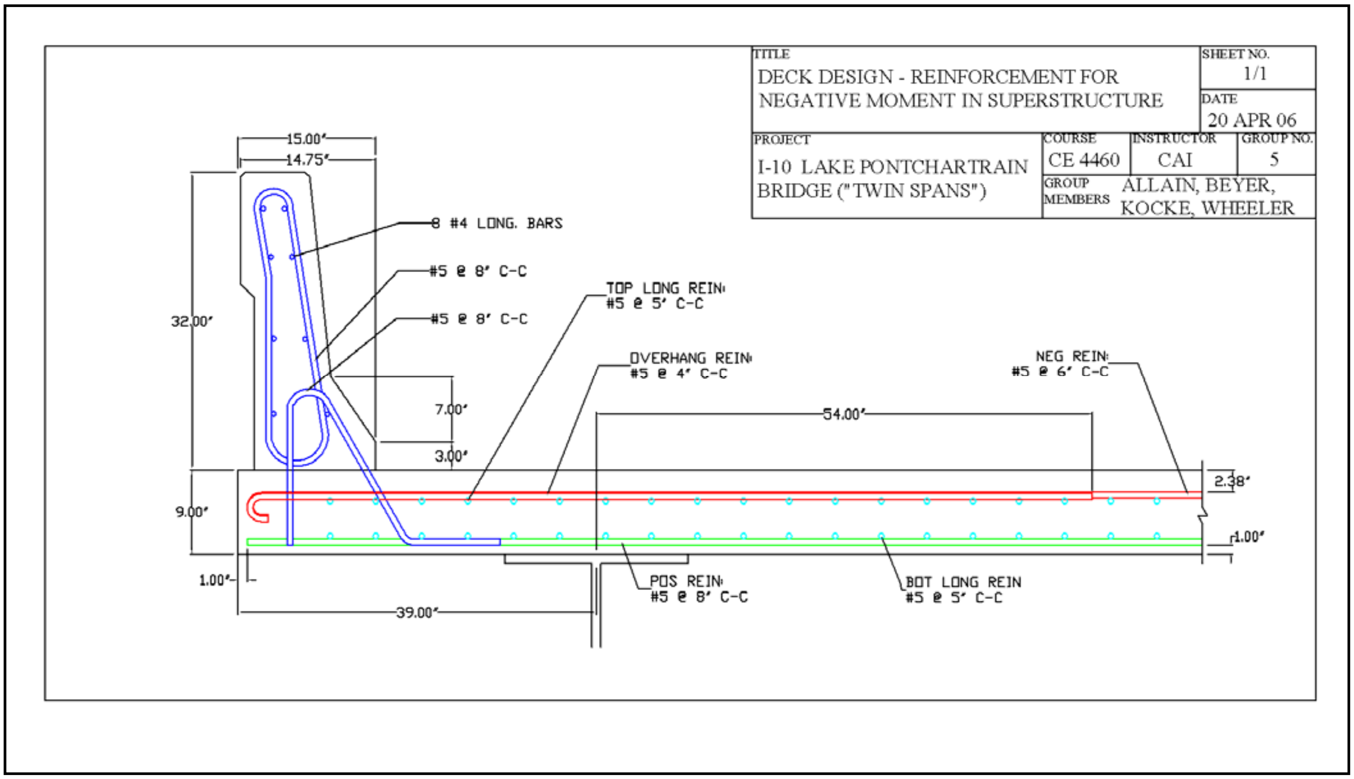


Figure 19. Deck Reinforcement for Negative Moment Substructure

When the bridge girders are subjected to negative moment (such as when crossing a pier), the flexure in the top flanges of the girders places the bridge deck under tensile stresses in the direction of travel, or perpendicular to the primary reinforcement. More longitudinal reinforcement is needed in the bridge deck for these areas to control cracking.

Design Criteria

Clear Width:	60 ft	Design Specifications, Section 3
Total Width:	62.5 ft	Design Specifications, Section 3
Number of Lanes:	3, w/ 2-12 ft shoulders	Design Specifications, Section 3
Girder Spacing:	S = 9 ft 4 in	
Number of Girders:	N = 7	
Overhang Length:	d_{OH} = 3 ft 3 in	
Deck Top Cover:	c_t = 2.375 in	Design Specifications, Section 5
Deck Bottom Cover:	c_b = 1 in	Design Specifications, Section 5
Concrete Density:	γ_c = 150 pcf	Design spec for $f'_c < 8500$ psi
Concrete Strength:	f'_c = 4000 psi	Assumed
Steel Strength:	f_y = 60 ksi, epoxy coated	Design Specifications, Section 5
Steel Density:	γ_{st} = 490 pcf	Design Specifications, Section 2

FWS Density:	γ_{FWS} = 144 pcf	Design Specifications, Section 2
FWS Thickness:	t_{FWS} = 2.5 in	Assumed from FHWA SBD Ex.
*Integral Wearing Surface**:	t_{IWS} = 0.5 in	AASHTO 5.12.3

*included in top cover, but does not contribute to deck strength.

Barrier Properties

Source:	FHWA Bridge Rail Guide
Type:	F-Shape
Height:	h_{bar} = 32 in
Test Level:	TL-4
Cross-Sectional Area:	A_{bar} = 292.7 in^2
Weight per foot:	w_{bar} = 305 lb/ft
Width at base:	W_{bar} = 14.75 in
Distance from barrier face to deck edge:	d_{bf} = 15 in
Drainage Considerations:	6-in x 1-ft slots @ 6 ft C-C

Slab Thickness

To determine the design slab thickness, the minimum slab thicknesses were determined from the AASHTO code. The minimum slab thickness for non-overhang slab section, according to AASHTO 9.7.1.1, is 7 inches for decks in which the slab thickness is greater than 1/20 the girder spacing. For overhang sections, AASHTO 13.7.3.1.2 stipulates that the slab thickness must be at least 8 inches.

Standard practice is to use a deck thickness of about 8 to 8.5 inches to allow room for the reinforcement. The design specifications for this project mandate a top cover thickness of 2.375 inches, which is greater than the standard 2-inch top cover. To accommodate this increased cover, this design uses a slab thickness of 9 inches. Being that this figure exceeds the minimum thickness for the overhang, the slab was designed using a uniform thickness of 9 inches for the entire bridge deck.

Dead Load Effects

The dead load effects for the deck design include the self weight of the slab and the barrier and the anticipated weight from the future wearing surface (FWS).

The bridge deck can be modeled as a one-way slab because the distance between the lateral supports (i.e., the girders) is much less than the distance between the longitudinal supports. When the deck is viewed as a one-foot wide beam in the lateral cross section, it can be analyzed as an indeterminate continuous beam supported by the seven girders.

Staad.pro was utilized to analyze this beam. The slab dead weight was modeled as a uniform load of 112.5 pounds per foot acting on the whole beam. The two barrier weights (one for each side) were modeled as 305-pound concentrated loads acting at the centers of gravity of the barriers. The FWS load was modeled as a uniform load of 30 pounds per foot acting between the faces of the barriers.

After the Staad.pro analysis was complete, the proper load factors were applied. The load factors were taken from AASHTO Table 3.4.1-2. To achieve a conservative design and without further information, the maximum load factors were chosen for this design.

Figure 20 below displays the shear and moment diagrams for DC and DW loading.

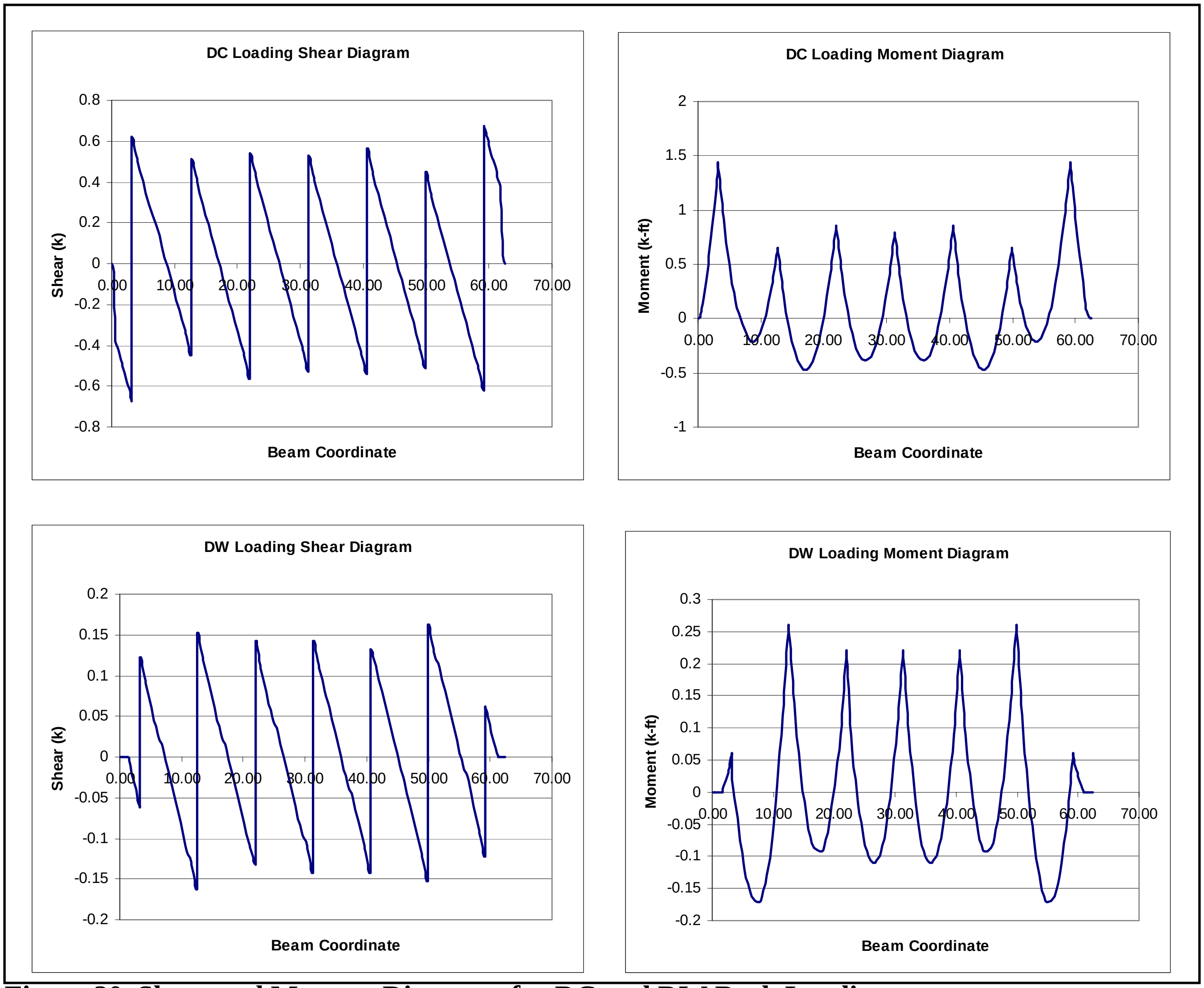


Figure 20. Shear and Moment Diagrams for DC and DW Deck Loading

Live Load Effects

The AASHTO code makes a simplifying allowance for live load effects on bridge decks. AASHTO Table A4-1 allows the designer to assume a maximum positive and negative live load moments given the girder spacing. Using linear interpolation, the live load effects, already adjusted for dynamic loading, can be calculated.

Positive Moment Design

After a bar size assumption, the effective depth was found. This calculation includes a half inch allowance for the integral wearing surface (IWS). The IWS is not included in the structural calculations because it is assumed that it will wear off as the bridge ages. The positive moment design is No. 5 bars @ 8 inch spacing.

Negative Moment Design

The design for negative moment followed the same procedure as the positive moment design. Being that the compression block for negative moment is on the bottom of the slab, opposite the IWS, no allowance for the IWS was needed in calculating effective depth. The original negative moment design was identical to the positive moment design, using No. 5 bars @ 8 inch spacing, but, when checked, cracking under the service limit state controlled, and the spacing had to be reduced to 6 inches.

Overhang Design

Unlike the moment design for the interior deck sections, which dealt only with strength and service limit states, the overhang design required investigation of extreme limit states. The extreme limit state accounts for the loading applied during a vehicle collision with the concrete barrier. The maximum moment due to the extreme event was determined to be the maximum moment about the base of the barrier due to the worst collision the barrier was designed to withstand. This information was included with the other barrier properties.

The extreme limit state had to be checked for the bridge deck below the inside barrier face and at the design section in the overhang during vehicle collision. This was done using Staad.pro by applying a concentrated moment at the barrier base and analyzing the idealized 1-foot wide bridge deck. Vertical collision forces do not control according to AASHTO A13.4.1, so this load case did not need to be checked.

The overhang had to be further checked for strength and service limit states, but the extreme limit state ultimately controlled. The reinforcement needed for the overhang is No. 5 bars @ 4 inch spacing. The overhang development length was computed using Staad.pro and by graphing the moment diagram in Bay 1. The graph below depicts the moment diagram from the edge of the deck to the Girder B, the first interior girder. The red line indicates the negative moment capacity of the interior primary reinforcement. The point of intersection of the two lines was calculated to be 6 feet from the edge of the deck. Adding a development length of 21 inches (AASHTO 5.7.3.4), the additional reinforcement for the overhang should end at 93 inches from the edge of the deck, or 4.5 feet inside of Girder A.

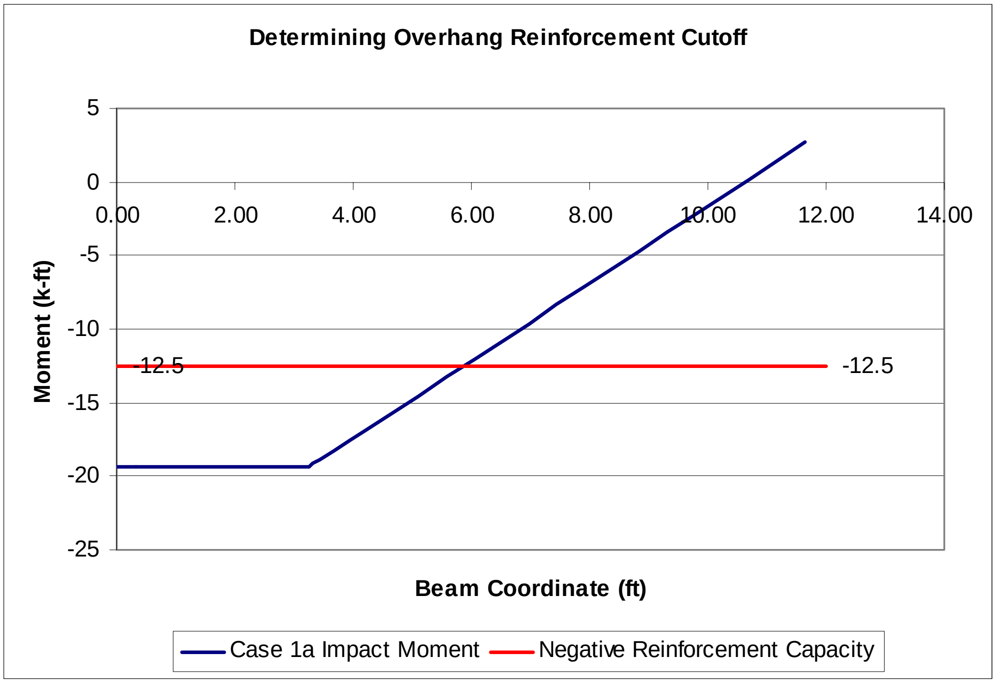


Figure 21. Moment Capacity for Overhang Reinforcement Cutoff Design

GIRDER DESIGN

The proposed bridge girder design is steel girders composed of built-up welded plates. A trial girder size was selected with a web depth of 54” and 1/2” thickness. For the positive moment region of the girder, a plates size of 14” x 5/8” was used for the top flange while the bottom flange measures 14” x 7/8”. For the top flange in the negative region of the girder, the dimensions are 14” x 2 ½” and the bottom girder has 14” x 2 ¾” measurements. A larger flange area is required in the negative moment region because the maximum moment occurs at the pier which is subjected to a negative moment. Thus, more flange area is required to resist the additional flexure. Based upon the trial girder, the section properties and dead load effects will be computed and compared to the applied loads in order to determine if the trial girder is adequate.

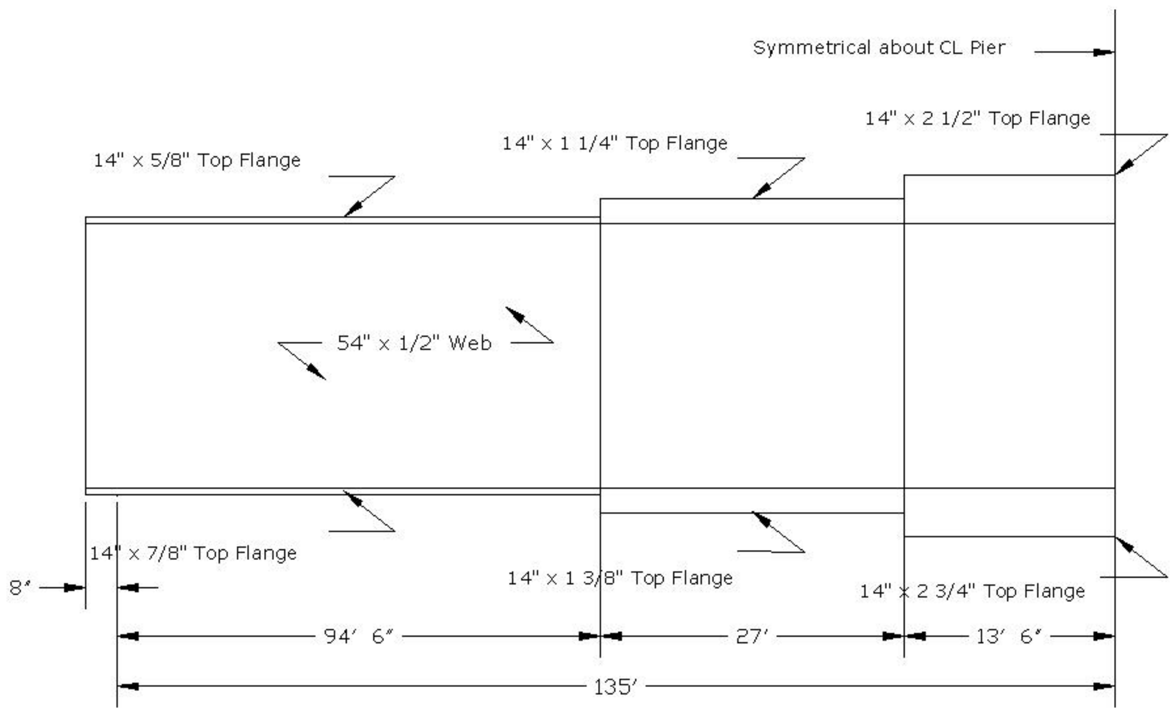


Figure 22. Preliminary Girder Design

Design Criteria

Table 1. General Design Criteria

Number of Spans	4
Span Length	135’
Skew Angle	0°
Number of Girders	7
Girder Spacing	9.33’
Deck Overhang	3.25’
Cross-frame Spacing	15’
Web Yield Strength	50 ksi
Flange Yield Strength	50 ksi
Concrete Strength	4.0 ksi
Reinforcement Strength	60 ksi

Table 2. General Design Criteria (2)

Total Deck Thickness	9”
Effective Deck Thickness	8.5”
Total Overhang Thickness	9”
Effective Overhang Thickness	6.31”
Steel Density	0.490 kcf
Concrete Density	0.150 kcf
Additional Miscellaneous Dead Load (per Girder)	0.015 k/ft
Deck Form Weight	0.015 k/ft
Parapet Weight	0.305 k/ft
Future Wearing Surface Weight	0.140 kcf
Future Wearing Surface Thickness	2.5”
Deck Width	62.5’
Roadway Width	60’

Section Properties

Because the girder is composite, the cross-sectional properties must be computed for both the positive and negative moment region. For the time being, only the dead loads will be considered to act upon the girder for computation. Tables 3 and 4 below show the different properties of each composite element of the girder for the positive and negative moment region.

Table 3. Sectional Properties: Positive Moment Region

Positive Moment Region Section Properties						
Section	Area A (in2)	Centroid d (in)	A*d (in3)	Io (in4)	A*y2 (in4)	Itotal (in4)
Girder only						
Top flange	10.5	55.125	578.81	0.5	8382.4	8382.9
Web	27	27.875	752.63	6561	110.5	6671.5
Bottom flange	12.25	0.438	5.3655	0.8	7912	7912.7
Total	49.75	26.87	1336.8	6562.3	16404.9	22967.1
Composite (3n):						
Girder	49.75	26.87	1336.8	6562.3	11072.6	17634.9
Slab	40.5	60.115	2434.7	273.38	70726.1	70999.5
Total	90.25	41.789	3771.5	6835.7	81798.7	88634.4
Composite (n):						
Girder	90.25	26.87	2425.1	22967	1559.4	24526.5
Slab	121.5	60.115	7304	820.13	256489	257309
Total	211.75	45.946	9729	447008	258048	281835
Section	ybotgdr (in)	ytopgdr (in)	ytopslab (in)	Sbotgdr (in3)	Stopgdr (in3)	Stopslab (in3)
Girder only	26.87	28.23	-	854.73	813.6	-
Composite (3n):	41.789	13.311	-	2121	6658.8	-
Composite (n):	45.946	9.1542	-	6134.1	30787.6	-

Table 4. Sectional Properties: Negative Moment Region

Negative Moment Region Section Properties						
Section	Area A (in2)	Centroid d (in)	A*D (in3)	Io (in4)	A*y2 (in4)	Itotal (in4)
Girder only						
Top flange	35	58	2030	18.2	30009.7	30027.9
Web	27	29.75	803.3	6561	28.7	6589.7
Bottom flange	38.5	1.375	52.9	24.3	28784.7	28809
Total	100.5	28.718	2886.2	6603.5	58823.1	65426.6
Composite (3n):						
Girder	100.5	28.718	2886.2	65427	58823.1	65426.6
Slab	40.5	63.2	2559.6	273.38	60413.4	60686.8
Total	141	38.622	5445.8	65700	119237	126113
Composite (n):						
Girder	100.5	28.718	2886.2	65427	8081.82	73508.4
Slab	121.5	63.2	7678.8	820.13	275173	275993
Total	222	47.59	10565	66247	283255	349502
Composite (deck reinforcement only)						
Girder	100.5	28.718	2886.2	65427	16072.3	81498.9
Deck reinfor.	23.16	61.96	1435	0	77997.5	77997.5
Total	123.66	34.944	4321.2	65427	94069.8	159496
Section	ybotgdr (in)	ytopgdr (in)	ytopslab (in)	Sbotgdr (in3)	Stopgdr (in3)	Stopslab (in3)
Girder only	28.718	30.532	-	2278.2	2142.89	-
Composite (3n):	38.622	20.628	-	3265.3	6113.82	-
Composite (n):	47.59	11.66	-	7344	29974.2	-
Composite (rebar)	34.944	24.306	-	4564.4	6561.97	-

Load Effects

1. Dead Load

The girder must be designed to resist the dead load components, consisting of both the composite and noncomposite sections. The following table lists the different dead loaf components and the type of load factor to be used.

Table 5. Dead Load Components

Dead Load Components		
Resisted by	Type of Load Factor	
	DC	DW
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Miscellaneous dead load (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface

Because the of the different flange sizes along the length of the steel girder, the dead load per unit length varies. The dead load per unit length for various bridge properties include

$DL_{deck} = 1.05$ k/ft
$DL_{deckforms} = 0.122$ k/ft
$DL_{misc} = 0.015$ k/ft
$DL_{par} = 0.167$ k/ft
$DL_{fws} = 0.292$ k/ft

2. Live Load

For a live load consisting of a HL-93 truck, the live load effects were computed by performing an analysis and using the lever rule. The following table displays the results.

Table 6. Live Load Distribution Factors

Live Load Distribution Factors				
Interior Girder	g(m,1)	0.428	g(m, 2)	0.503
	g(v, 1)	0.733	g(v, 2)	0.906
Exterior Girder	g(m,1)	0.975	g(m, 2)	0.568
	g(v, 1)	0.892	g(v, 2)	0.838

The girder must be designed around the effects of the maximum loads that it undergoes. In order to determine these loads, Staad.pro was used to analyze the structure. Once the analysis was complete, the maximum moments and shear were read off of the printout. (See Appendix #) The data retrieved is tabulated in Table 7.

Table 7. Maximum Loads

Maximum Loads			
Load case	(+) Moment (k-ft)	(-) Moment (k-ft)	Shear (k)
DC Loading	2053.2	2855.4	120
DW Loading	252.4	351	14.75
Max Truck/Tandem	1720.3	979.3	52.31
Lane Load	897.6	1248.3	52.45

Once the maximum loads were determined, it was necessary to factor the loads for the following Limit states: Strength I, Service II, and Fatigue. This was done for both the positive and negative region of the girder and is tabulated below.

Table 8. Factored Loads for Positive Moment

Combined Effects at Location of Max. Positive Moment			
Summary of Unfactored Loads			
Loading	Moment (k-ft)	f(botgdr) ksi	f(topgdr) ksi
Noncomposite DL	2053.2	28.83	30.28
FWS DL	252.4	1.43	0.45
LL - HL - 93	1720.3	3.37	0.67
Lane Load	897.6	1.76	0.35
Summary of Factored Loads			
Limit State	Moment (k-ft)	f(botgdr) ksi	f(topgdr) ksi
Strength I	6051.059	44.06	39.71
Service II	5491	36.49	31.98
Fatigue	1753	3.43	0.68

Table 9. Factored Loads for Negative Moment

Combined Effects at Location of Max. Negative Moment			
Summary of Unfactored Loads			
Loading	Moment (k-ft)	f(botgdr) ksi	f(topgdr) ksi
Noncomposite DL	2855.4	15.04	15.99
FWS DL	351	1.29	0.69
LL - HL - 93	979.3	1.60	0.39
Lane Load	1248.3	2.04	0.50
Summary of Factored Loads			
Limit State	Moment (k-ft)	f(botgdr) ksi	f(topgdr) ksi
Strength I	6582.75	24.80	22.02
Service II	5757	20.50	17.70
Fatigue	1447	2.36	0.58

Table 10. Factored for Shear

Combined Effects at Location of Max. Shear	
Loading	Shear (kips)
Noncomposite DL	120
FWS DL	14.75
LL - HL - 93	52.31
LL - Lane Load	52.45
Summary of Factored Loads	
Limit State	Shear (kips)
Strength I	282.68
Service II	256.77
Fatigue	68.62

Limit States

After the factored loads were determined, the rest of the design process is to check the trial girder for the different limit states and the adequacy of the girder. The following is a list of the checks that were performed. All calculations can be found in Appendix B.

Design Check: Positive Moment

- Section Proportions
- Compute Plastic Moment
- Determine Whether Section is Compact or Non-compact
- Check Strength I Limit State
- Check Shear
- Check Fatigue and Fracture limit state
- Check Service Limit State
- Check Constructability
- Check Lateral Torsional Buckling

Design Check: Negative Moment

- Section Proportions
- Compute Plastic Moment
- Determine Whether Section is Compact or Non-compact
- Check Strength I Limit State
- Check Shear
- Check Fatigue and Fracture Limit State
- Check Service Limit State
- Check Constructability
- Check Wind Effects on Flanges

Because all of the checks listed above were compliant, the trial girder is adequate and thus, selected to be the girder design for the new bridge. See below for the nominal dimensions.

Final Girder Design

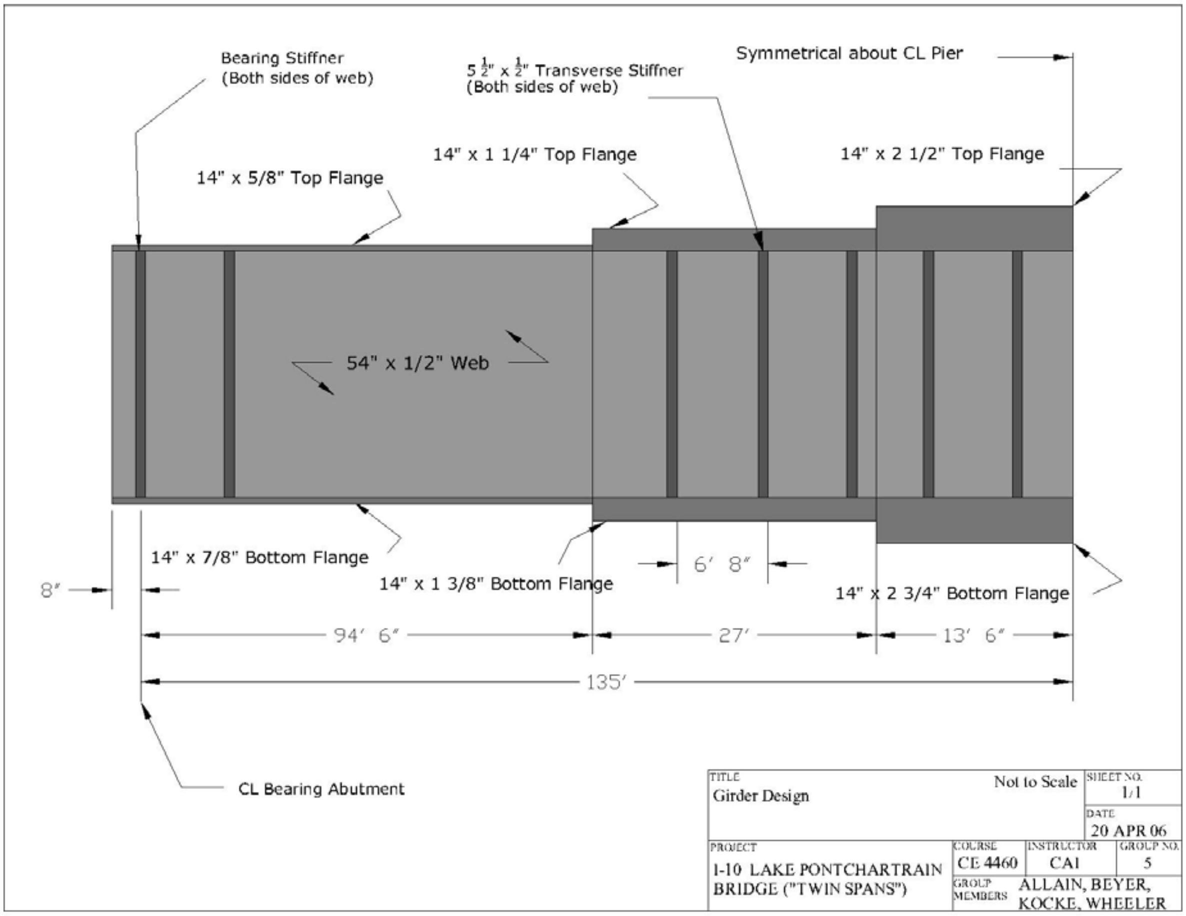


Figure 23. Final Girder Design

BOLTED FIELD SPLICE DESIGN

This section recounts the steps of the bolted field splice design process and marks the highlights. For complete calculations, see Appendix C.

Identify Field Splice Locations

There are three conditions that generally control the locations of field splices.

- Shipping restrictions on girder size.
- Splices near the point of dead load contraflexure.
- Splices should be located where the total moment is relatively small.

The first condition is a minor concern in this case. Being that the project is located in Lake Pontchartrain, which is near to and accessible by a number of maritime routes, the girders can be barged in and do not need to be put on a truck.

The other two conditions were already considered in the girder design. When the girder flanges are reduced is approximately where the moment contraflexure is. Therefore, the splices will be places at these locations.

There are six splices in all, or three symmetrical pairs. The outside splices will suffer the highest loads, so they were chosen as the design splice.

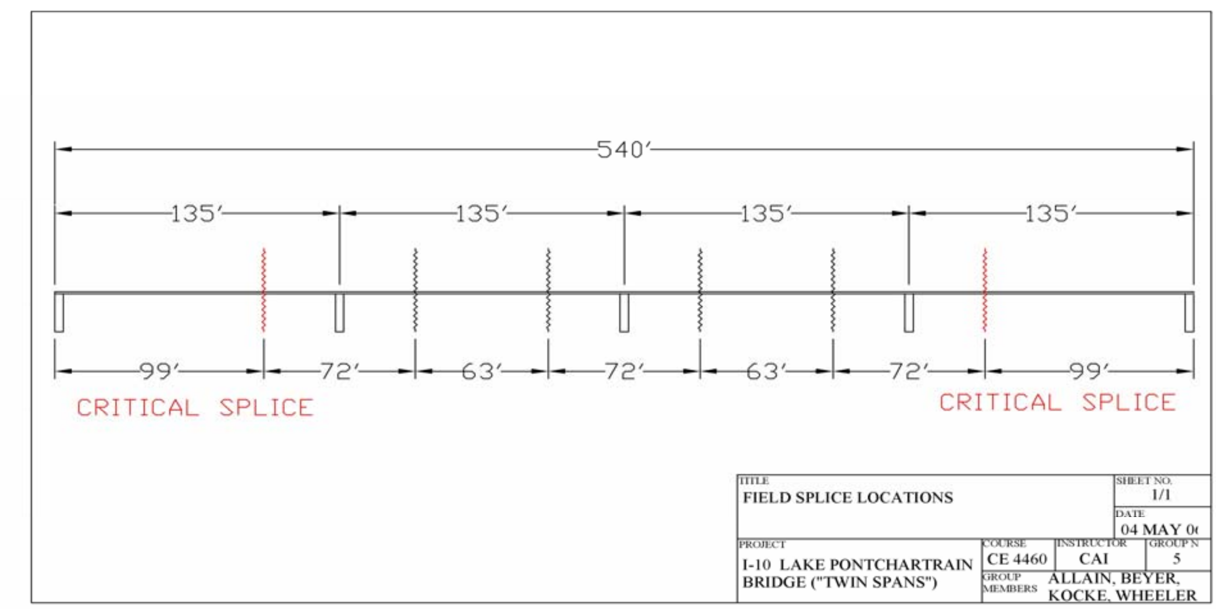


Figure 24. Splice Locations

Compute Girder Moments at Splice

The moments due to dead load, future wearing surface, truck loads, and lane loads were computed at the splice location using STAAD.Pro. The controlling positive moment limit state was found to be Strength I at 3477.3 k-ft, and the controlling negative moment limit state was found to be Fatigue at -246 k-ft.

Compute Flange Splice Design Loads

The flange design loads were computed using the section properties of the flanges and the composite and noncomposite girder shapes. These computations were already done in the Girder Design section of this project. The stresses in the bottom and top flanges were calculated to be 34.8 ksi and 25.2 ksi, respectively. These were both less than the minimum design load of 37.5 ksi, so 37.5 ksi was used in the flange splice design.

Design Flange Splices

The following limit states were checked.

Splice Plates:
- Yielding
- Fracture
- Compression
- Block Shear

Flange Bolts:
- Shear
- Minimum Spacing
- Maximum Spacing
- End Spacing
- Edge Distance

Compute Web Splice Design Loads

Using the same STAAD.Pro printout, the controlling ultimate shear load, V_u, was found to be 242.9 k under the Strength I limit state. Web moments due to the applied load and the eccentricity of the shear force also had to be considered. The total moment, M_T, was equal to 336.2 k-ft, and the associated axial force, H_w = 667.1 k.

Design Web Splice

The following limit states were checked.

Splice Plates:

- Shear Yielding
- Block Shear
- Fracture
- Flexural Yielding

Web Bolts:

- Minimum Spacing
- Maximum Spacing
- Edge Distance
- Shear
- Vertical Moment on the Extreme Bolt
- Horizontal Moment on the Extreme Bolt

Final Field Splice Design

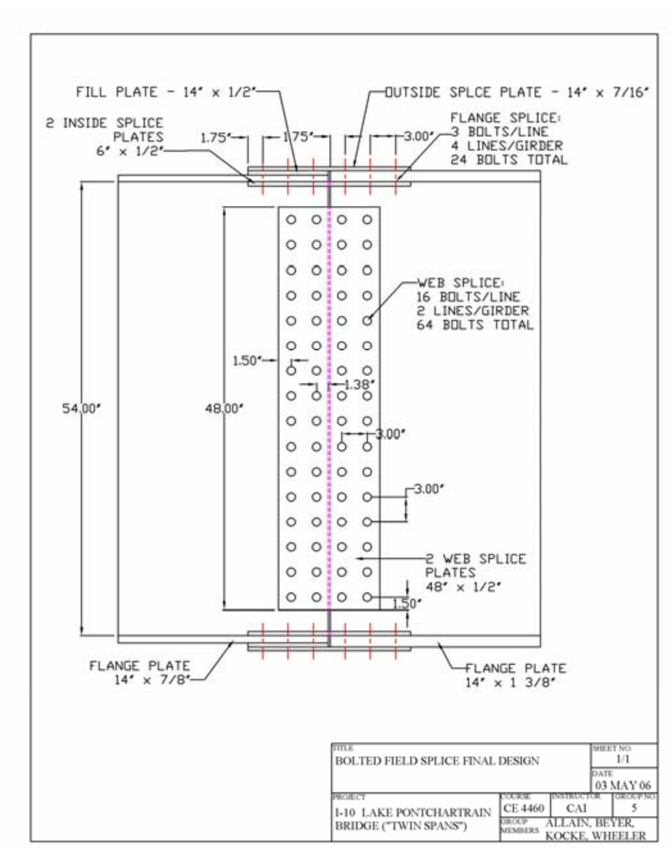


Figure 25. Final Field Splice Design

SUBSTRUCTURE DESIGN

The proposed width of the pile cap is 62.5 feet. It is reinforced concrete and the piles are also taken to be reinforced concrete. One of the first steps is to compile force effects on the substructure taking into account the forces caused by wind, water, scour, temperature, shrinkage, collisions, and braking forces. This was followed by analysis of the structure and a compilation of load combinations and then design of the pile cap and piles. The final design for the substructure consisted of a pile cap with the dimensions of 4’ x 4’ x 62.5’ and four piles with 5.5’ diameters and a length of 100’.

Wind

Wind forces occur in both the longitudinal and transverse directions and are assumed to act only at an angle of 0 degrees. They are assumed negligible on the pile cap, but are taken into account as a moment acting on the piles.

Water

There are both hydrostatic and dynamic forces acting on the structure. The dynamic forces are assumed to be negligible and the hydrostatic forces balance.

Scour

There are three types of scour possible: from lateral shifting of the channel, erosion of the riverbed, or localized scour from substructure restriction of flow. It is assumed that only the latter exists for the project and that it is accounted for in the length of the pilings by taking the preliminary scour depth to be 5’.

Temperature and Shrinkage

Due to the symmetry of the bridge, it is assumed that there are no forces on the intermediate bent due to temperature expansion or shrinkage of the superstructure.

Ship Collision

Ship collisions are assumed to act only on the pile cap and not on the pilings. The forces for collisions are provided in the project specifications. The design specifications state that the substructure should be able to withstand an impact force from an oversize tanker. However, this seems overdesigned. Due to the weak soils in Lake Pontchartrain, the proposed design cannot withstand this force, but was able to withstand impact forces from an empty barge.

Braking Force

The specifications state that the breaking force used is to be the greater of 25% of the axle weight of the design truck or 5% of the design truck plus lane load. The braking force acts as a moment on the columns, but has a negligible effect on the design of the pier cap.

Bearings

The bearings used are elastomeric steel reinforced bearing pads that are 2" thick and 24" x 24" centered on the pile cap.

PILE CAP DESIGN

Maximum loads on the cap are given below:

Table 11. Maximum Loads

	Location*	Unfactored Responses			Str-I
		DC	DW	LL+IM	
Max Pos M (k-ft)	at 9' from JT3	503	63	215	1099.5
Max Neg M (k-ft)	at JT 2	-539	-67	-231	-1178.5
Max Shear (k)	at JT 3	323	40.1	138	705.4

*where location is measured from the end of the cap and JT 3 is the center pier cap

Braking force was considered to be negligible for the pier cap design. Live loads on the superstructure were obtained from the girder live load analysis to obtain the maximum unfactored live load reactions for the interior and exterior girder lines.

Table 12. Pile Cap Design Criteria

Concrete Strength	4 ksi
$\beta 1$	0.85
Reinforcement Strength	60 ksi
Cap Width	4 ft.
Cap Depth	4 ft.
Number of stirrup legs	6
Stirrup diameter (#5 bars)	0.625 in.
Stirrup area (per leg)	0.31 in.^{2}
Stirrup spacing along cap	varies
Cover (column and cap)	3 in.

Pile Cap Design Steps:

- Flexural resistance
- Maximum positive moment—bottom steel

Limits for reinforcement
Flexural reinforcement
Service load applied steel stress

- Maximum negative moment—top steel
  Limits for reinforcement
  Flexural reinforcement
  Service load applied steel stress
- Check for minimum temperature and shrinkage steel
- Skin reinforcement—used 7 and 12 inch spacing
- Maximum shear

PILE DESIGN

The reinforced pre-cast piles were designed as columns, that is, it was assumed that there were no soil forces acting along the length of the pile. The dimensions were determined using the maximum slenderness ratio. The diameter was 5'-6'' and the length of the pile was 100'. Due to time constraints, battered pilings were not considered. The final design for the piles included 50 #8 bars as longitudinal reinforcement with 3" cover.

To check the pile for ultimate loads, the ultimate moment, M_u, and the ultimate axial force, P_u, were computed using the Strength V limit state. To obtain the column interaction diagram, the program Kader Column was used. The program outputs were put in an Excel spreadsheet and the column interaction curve was drawn for the nominal moment and design moment. By plotting M_u and P_u, it was determined that the pile could withstand the ultimate loads. The column interaction diagram can be found in Figure 26 below. For all calculations for the cap and pile design, see Appendix E.

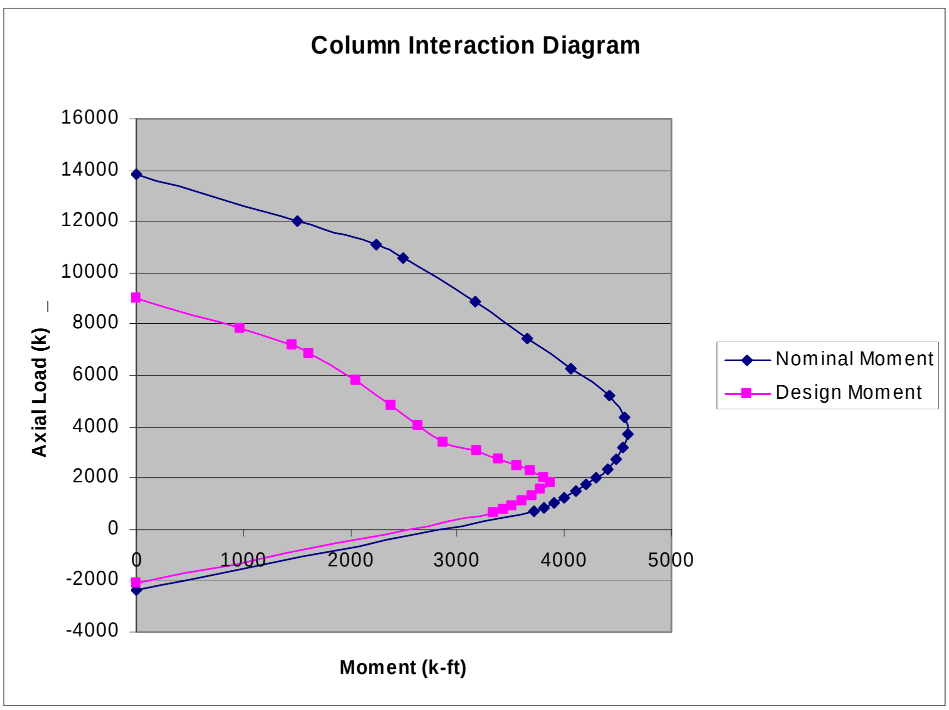


Figure 26. Column Interaction Diagram

REFERENCES

1. Interstate Facts. (2006). Federal Highway Administration. Retrieved April 25, 2006 from: http://www.fhwa.dot.gov/interstate/previousfacts.cfm.

2. "News in DOTD: DOTD to reopen westbound lanes of I-10 'twin span' bridge Friday." (Jan 2006). Louisiana Department of Transportation and Development. Retrieved April 25, 2006 from: http://www.dotd.louisiana.gov/press/pressrelease.asp?nRelease=584.

3. D'andrea, Artur. (Apr 6, 2006) "I-10 Bridge over Lake Pontchartrain." Presentation.

4. Corrosion. (2006). Wikipedia.org. Retrieved may 7, 2006 from: http://en.wikipedia.org/wiki/Corrosion.

5. Chong, Shuang-Ling. (2004). "Preventing Corrosion in Steel Bridges." Federal Highway Administration. Retrieved May 7, 2006 from: www.tfhrc.gov/ pubrds/04sep/08.htm.

6. Frequently Asked Questions. RustSucks.com. Retrieved May 7, 2006 from: http://www.rustsucks.com/faq.html.

This page intentionally left blank.