CE 4460 Bridge Project

Spring 2006

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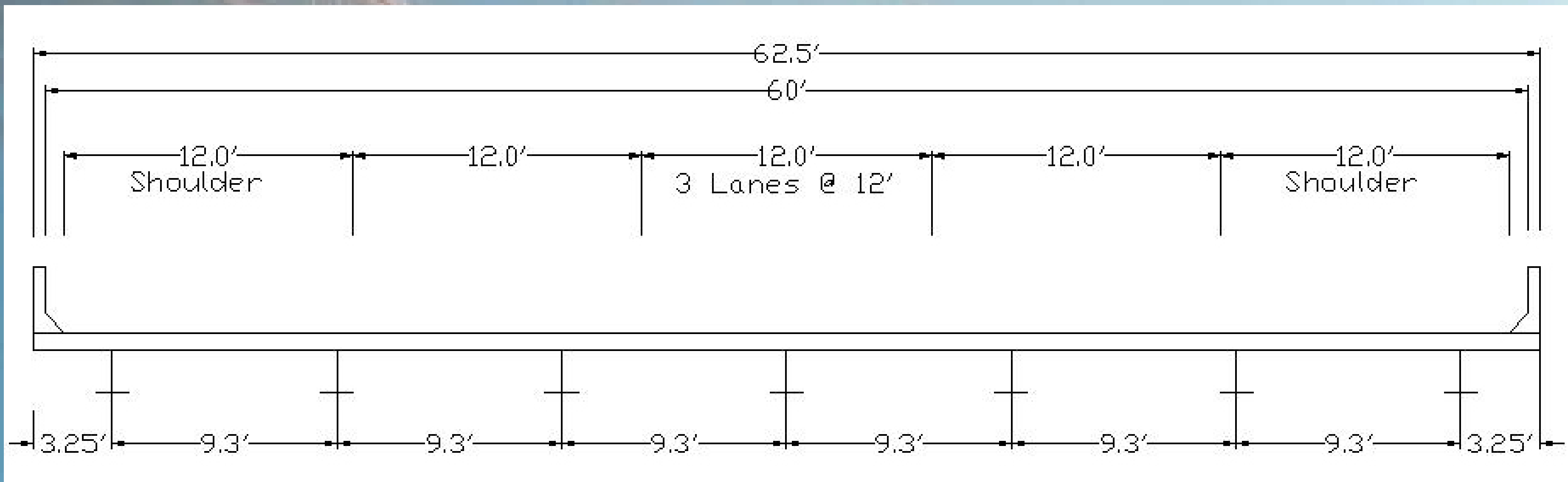
Objective:

- Design a new I-10 bridge across Lake Ponchartrain
- Design according to LRFD and AASHTO
- 4 span segment design with steel girders

Overview of Project

1. Obtain general information
2. Concrete deck design
3. Steel girder design
4. Bolted field splice design
5. Pier design
6. Pile design

General Information



- 3 Lanes and 2 Shoulders @ 12' Spacing
- 7 Steel Girders @ 9.33' Spacing
- Slab Thickness = 9"

Concrete Deck Design

Steps

- Determine Slab Thickness
- Compute Dead Load Effects
- Compute Live Load Effects
- Determine Ultimate Moments
- Design for Positive Moment in Deck
- Design for Negative Moment in Deck
- Design for Moment in Deck Overhang
- Design Longitudinal and Temperature Reinforcement

Concrete Deck Design

Determine Slab Thickness

Minimum Slab Thickness:

AASHTO 9.7.1.1 – $t_d \geq 7"$ for slabs with $t_d > 1/20\ S$

Minumum Overhang Thickness:

AASHTO 13.7.3.1.2 – $t_o \geq 8"$ for overhangs w/ barriers

Deck Top Cover:

$c_t = 2.375"$

To ensure enough room for reinforcing steel, the deck was designed as a uniform 9-inch thick slab.

Concrete Deck Design

Compute Dead Load Effects

Determined DC and DW loads for a 1' strip:

DC: Slab – 112.5 lb/ft

Barrier – Two (2) 305 concentrated loads acting at 5.8" from each edge (barrier C.O.G.)

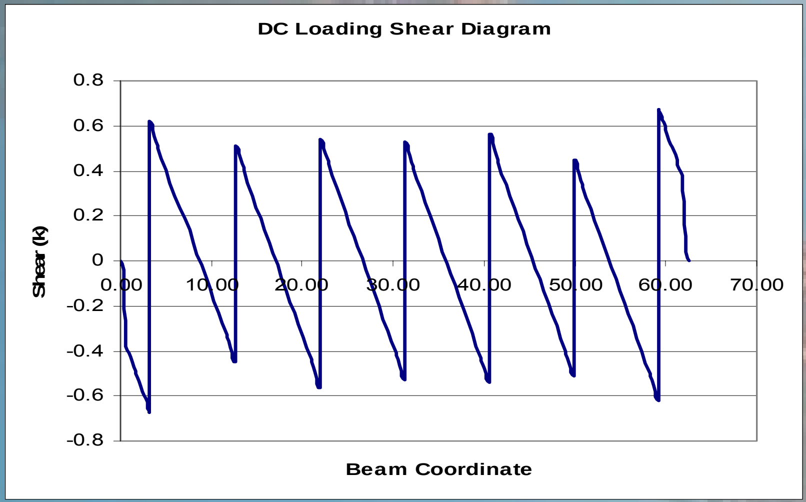
DW: Future Wearing Surface (FWS) – 30 lb/ft

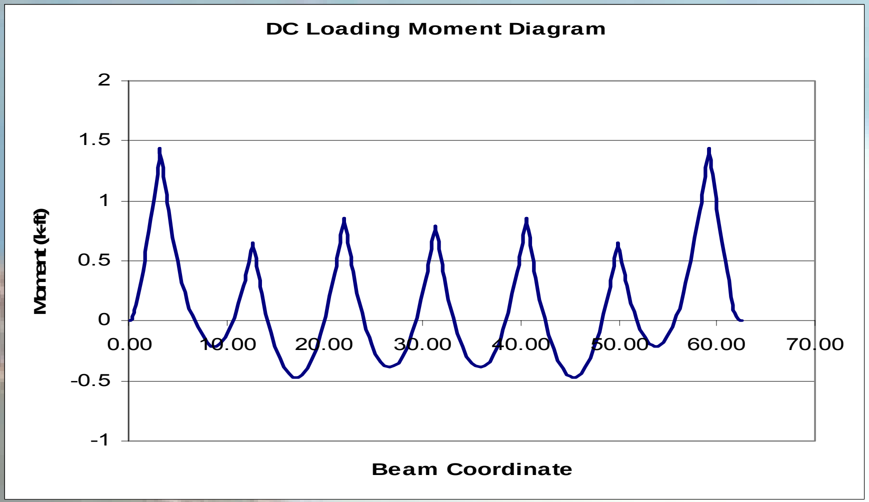
Ran STAAD.Pro

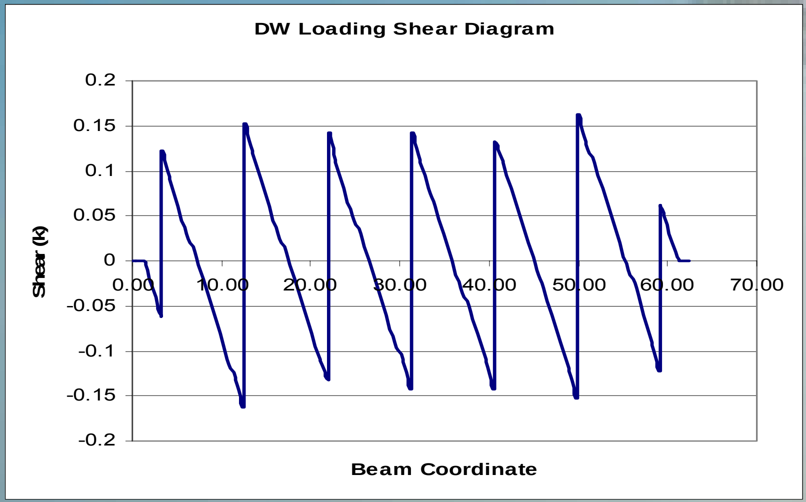
Modeled deck as a continuous indeterminate beam

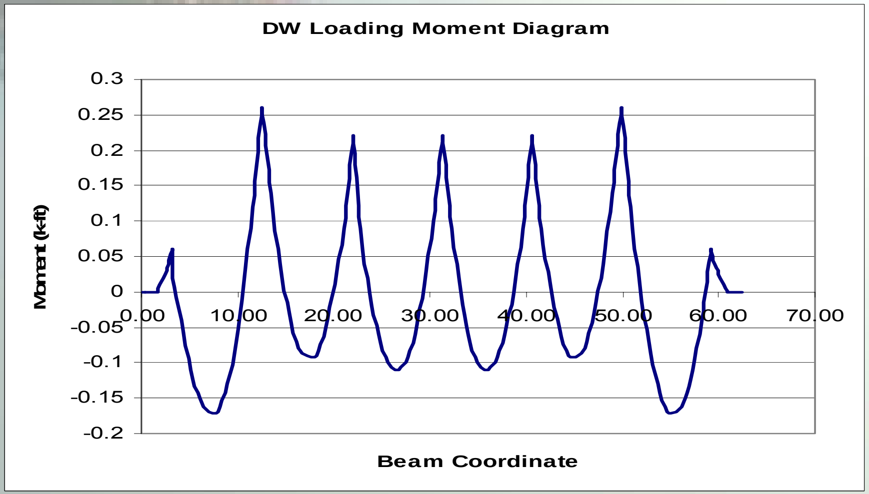
Concrete Deck Design

Compute Dead Load Effects









Concrete Deck Design

Compute Live Load Effects

- AASHTO Table A4-1 gives values for maximum live loads.

S (mm)	+M (N-mm)	-M (N-mm)
2800	29020	31050
2900	29910	32490

- After linear interpolation and conversion,
  - $+M_{max}$ = 6.61 k-ft/ft
  - $-M_{max}$ = -7.13 k-ft/ft
- Includes dynamic load allowance (IM)

Concrete Deck Design

Determine Ultimate Moments

- Using Strength I,

  $M_u = \gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + \gamma_{LL}M_{LL+IM}$,

  where $\gamma_{DC} = 1.25$, $\gamma_{DW} = 1.50$, and $\gamma_{LL} = 1.75$

- $M_u^+ = 12.4$ k-ft/ft

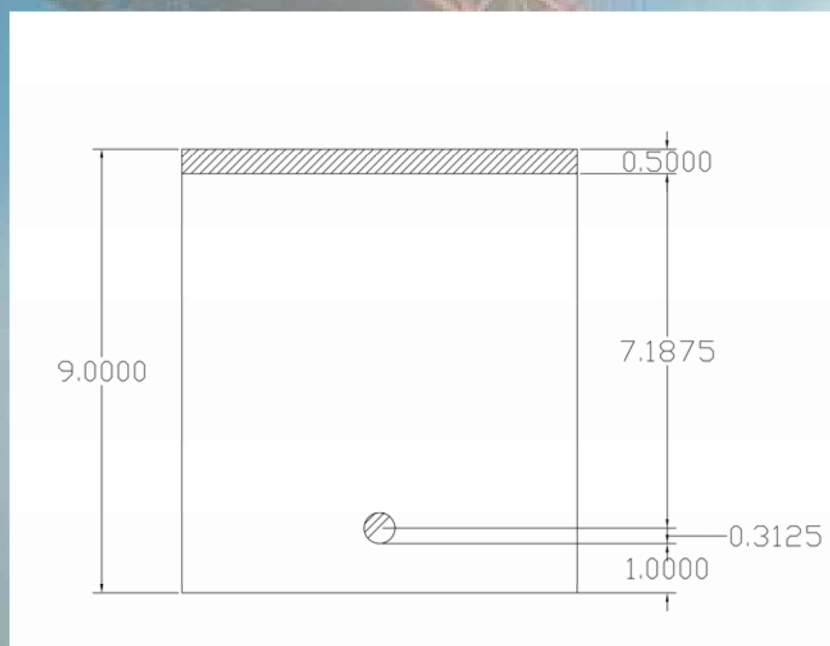
- $M_u^- = -15.0$ k-ft/ft

- The simplifying assumption that the maximum dead load and live load moments occur at the same location. This assumption yields simpler calculations and a conservative design.

Concrete Deck Design

Design for Positive Moment in Deck

- Assume bar size. (#5 bars)
- Effective depth: $d_e = t_d - c_b - .5d_b - IWS = 7.19"$
  - IWS = Integral Wearing Surface (allowance for wear)



Concrete Deck Design

Design for Positive Moment in Deck

- Solve for steel ratio required.
- Find the area of the reinforcing steel.
- Determine the spacing between bars. (s_b = 9.1", use 8")
- Check for maximum reinforcement. (OK)
- Check for cracking under the Service Limit State. (OK)
  - Assume severe exposure conditions (i.e., Lake Pontchartrain)
  - Find the transformed moment of inertia.
  - Calculate stresses.

Concrete Deck Design

Design for Negative Moment in Deck

- Assume bar size. (#5 bars)
- Effective depth: $d_e = t_d - c_t - .5d_b = 6.31"$
- Solve for steel ratio required.
- Find the area of the reinforcing steel.
- Determine the spacing between bars. ($s_b = 9.1"$, use 8")
- Check for maximum reinforcement. (OK)
- Check for Cracking Under the Service L.S. (not OK. . .)

Concrete Deck Design

Design for Negative Moment in Deck

- Redesign for cracking (reduce spacing to 6”)
- Recheck for maximum reinforcement. (OK)
- Recheck for Cracking Under the Service L.S. (OK)

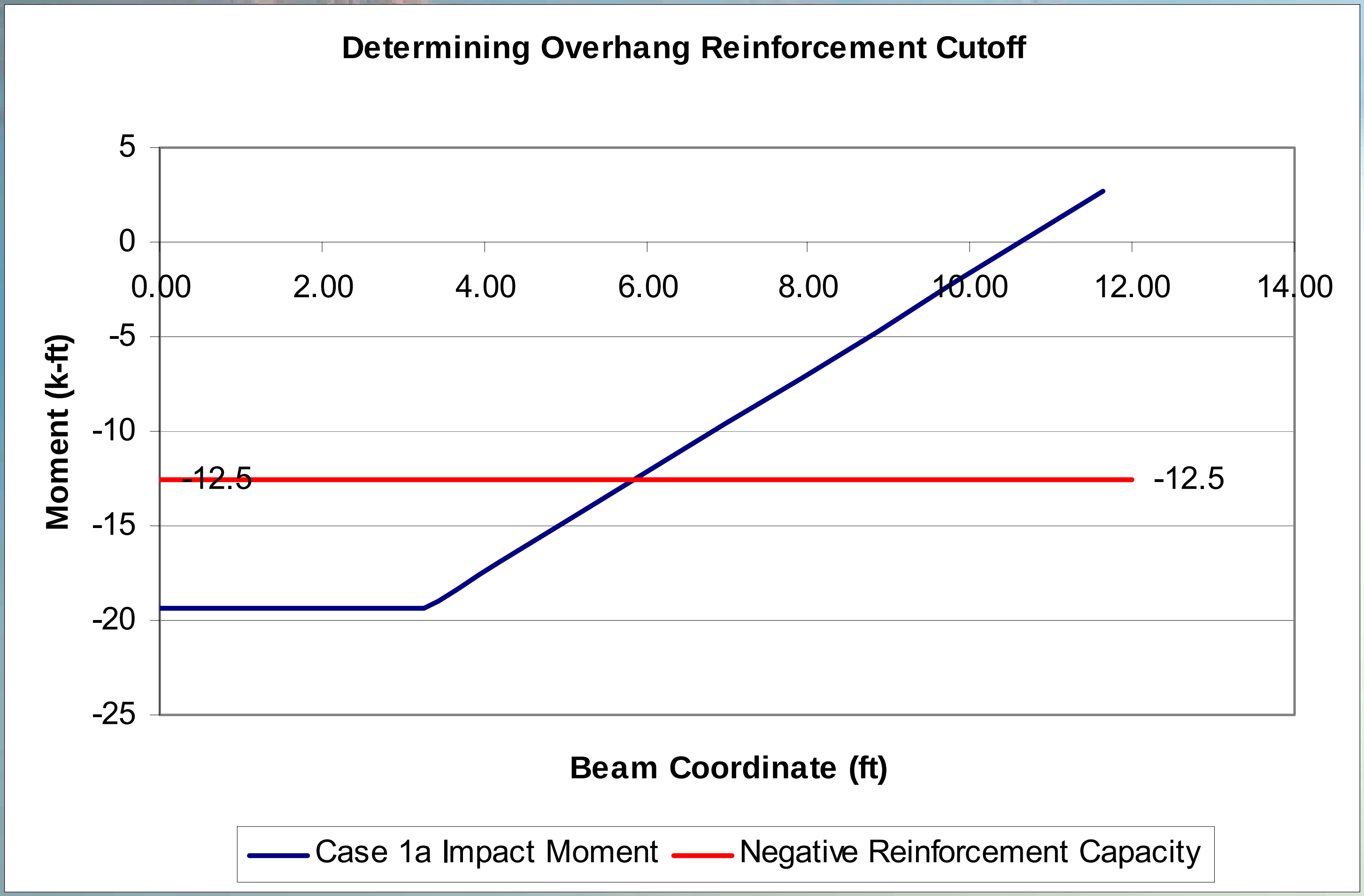
Concrete Deck Design

Design for Moment in Deck Overhang

- Determined that vehicle collision controlled design.
    - Extreme Limit State
    - Dependent on barrier properties (TL-4).
    - Vehicle collision creates uniform moment on overhang.
- Calculate steel – similar to negative deck moment.
    - #5 bars @ 4” C-C
- Compute overhang development length
    - More STAAD.pro…apply concentrated moment to deck.

Concrete Deck Design

Design for Moment in Deck Overhang

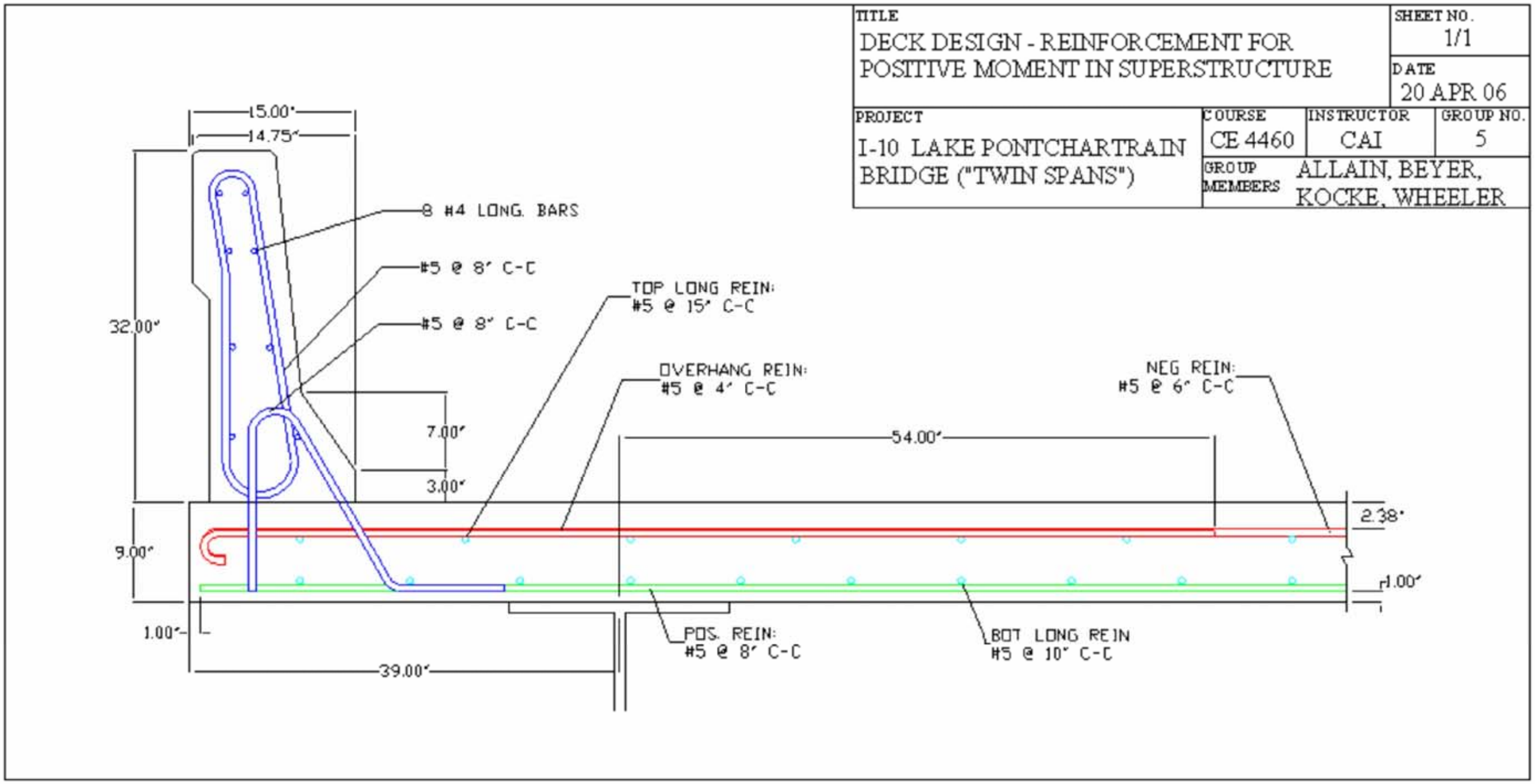


Concrete Deck Design

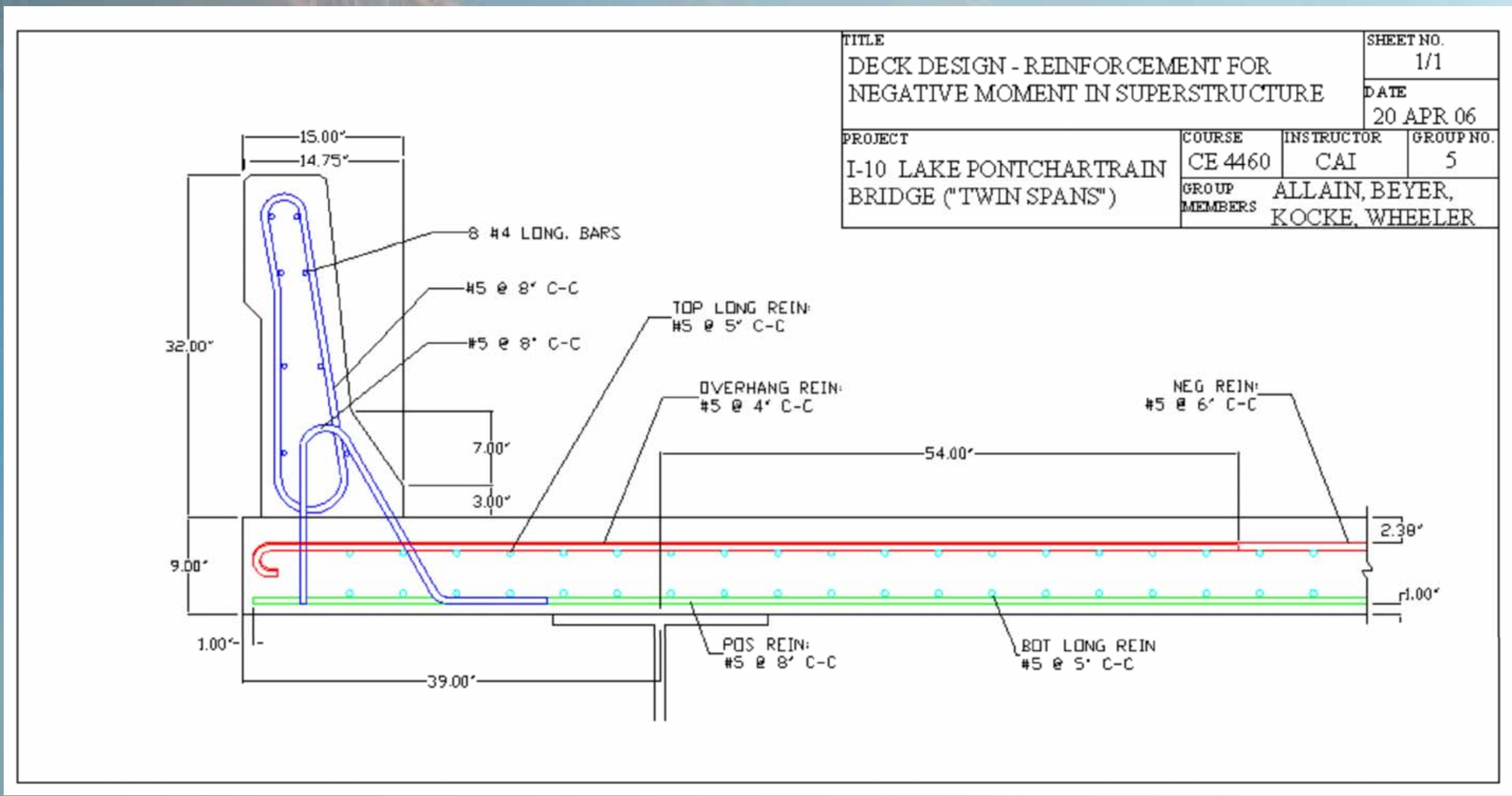
Design Longitudinal and Temperature Rein.

- Use AASHTO equations to determine % steel required for longitudinal and temperature reinforcement.
- Bottom reinforcement - #5 bars @ 10”
- Top reinforcement - #5 bars @ 15”
- Longitudinal reinforcement over piers:
    - Bottom and top - #5 bars @ 5”
    - More rein. is needed to prevent cracking when the superstructure is in negative moment when passing over the piers.

Concrete Deck Design



Concrete Deck Design



Deck Drainage

- Drainage provided by a 2.01% lateral slope of the bridge deck. The change in elevation of the deck is done by changing the height of the concrete haunches.
- Deck crowns between the inside and middle travel lane.
    - 25’ – 3” from the edge of the deck.
- Barrier drainage is provided 12” wide by 6” high slots at the base of the barrier, spaced at 6’ – 0” C-C.
    - Given by specifications.

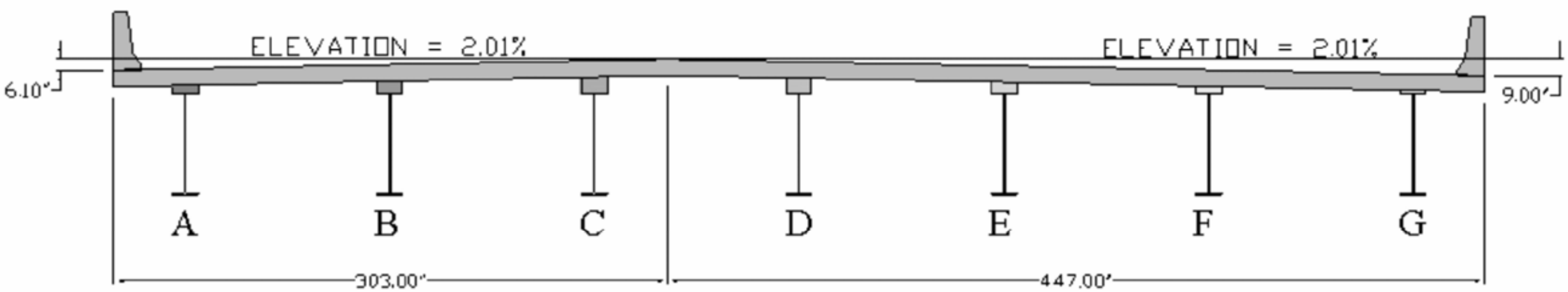
Deck Drainage

TITLE	SHEET NO.
BRIDGE DECK DRAINAGE DESIGN	1/2
	DATE 02 MAY 06

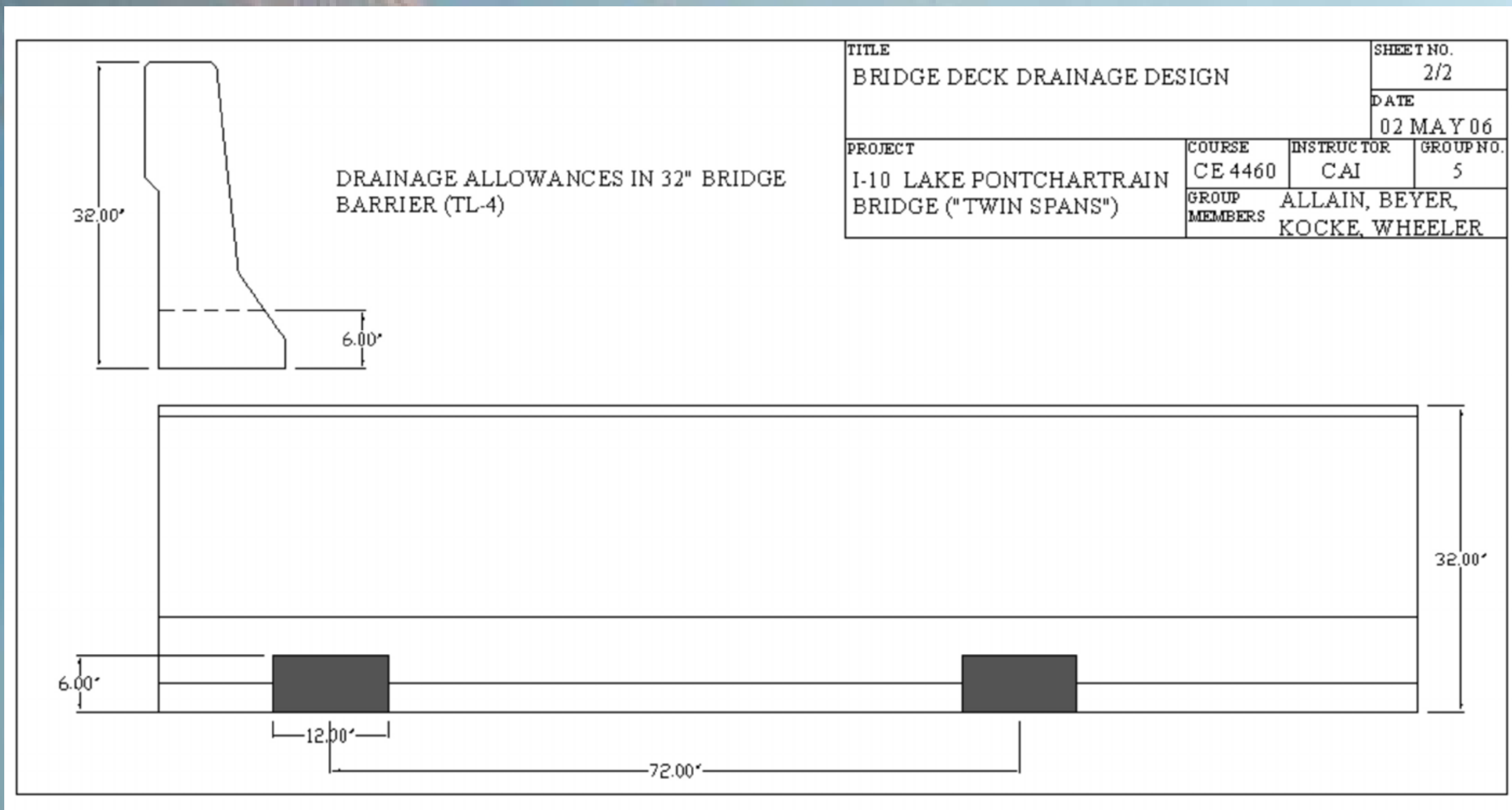
PROJECT	COURSE	INSTRUCTOR	GROUP NO.
I-10 LAKE PONTCHARTRAIN BRIDGE ("TWIN SPANS")	CE 4460	CAI	5
	GROUP MEMBERS	ALLAIN, BEYER, KOCKE, WHEELER	

HAUNCH DEPTHS

GIRDER	DEPTH
A	3.90"
B	6.15"
C	8.40"
D	7.75"
E	5.50"
F	3.25"
G	1.00"



Deck Drainage



Girder Design

- Basic Design Steps
  1. Obtain 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

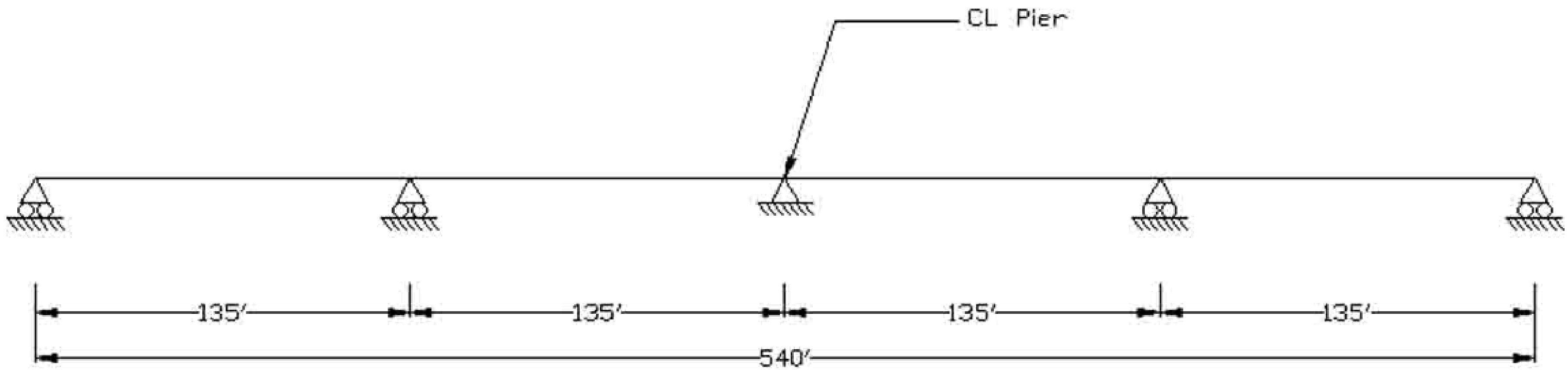
Girder Design

1. Obtain Design Criteria

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’

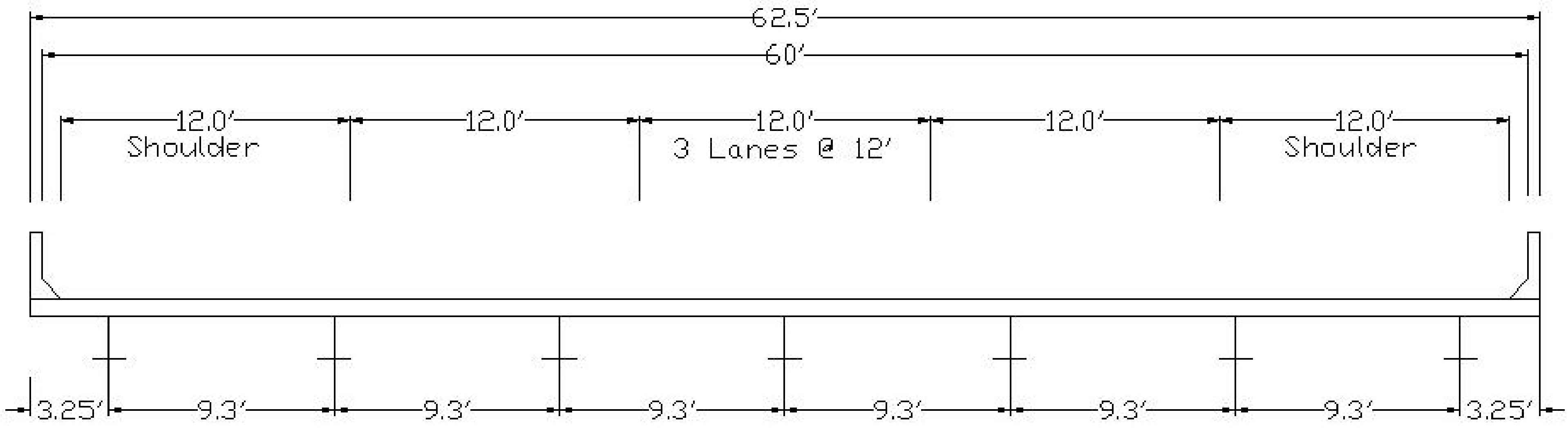
Girder Design

2. Span Configuration



Girder Design

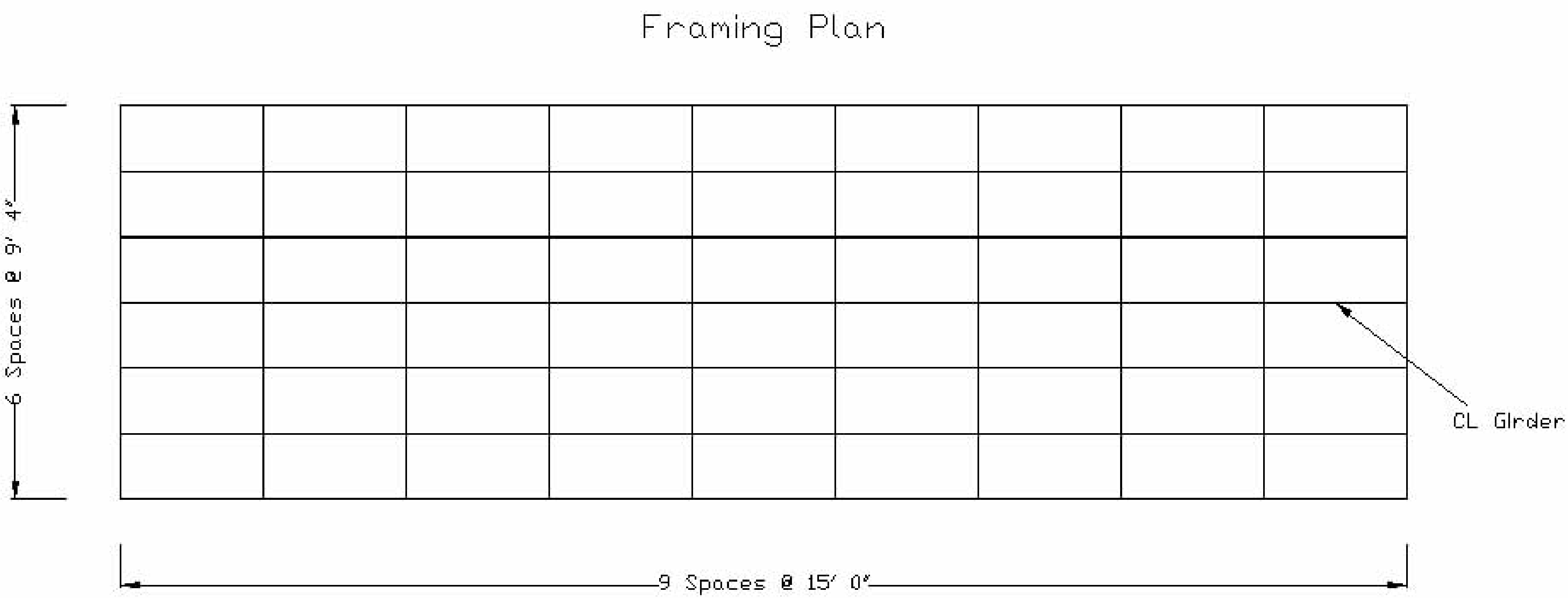
2. Superstructure Cross Section



- 3 Lanes and 2 Shoulders @ 12’ Spacing
- 7 Steel Girders @ 9.33’ Spacing

Girder Design

2. Framing Plan



Girder Design

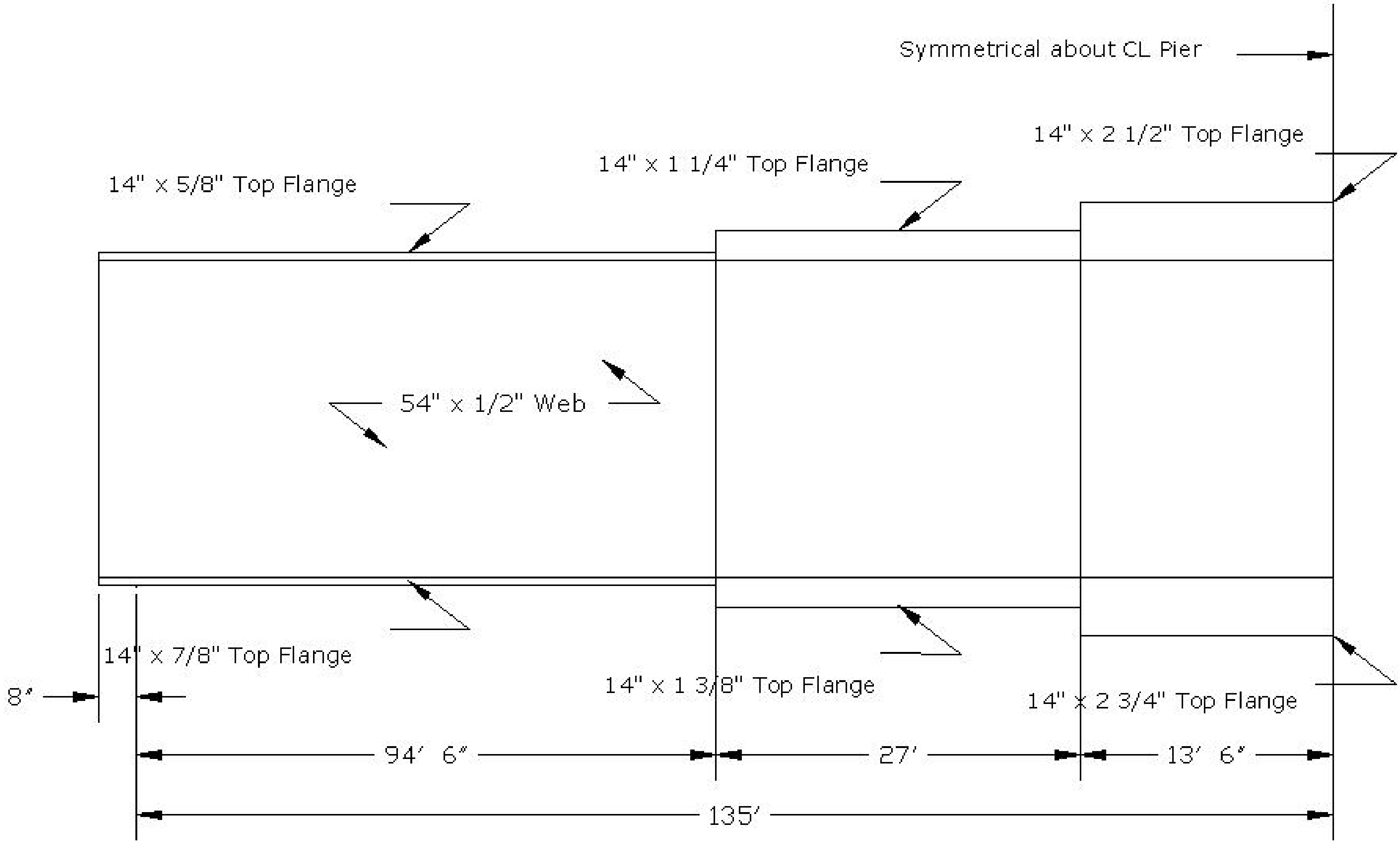
2. Design factors from AASHTO LRFD Bridge Design Specifications

Load Combinations and Load Factors							
Limit State	Load Factors						
	DC	DW	LL	IM	WS	WL	EQ
Strength I	1.25	1.50	1.75	1.75	-	-	-
Service II	1.00	1.00	1.30	1.30	-	-	-
Fatigue	-	-	0.75	0.75	-	-	-

Resistance Factors	
Type of Resistance	Resistance Factor, ϕ
For flexure	$\phi_f = 1.00$
For shear	$\phi_v = 1.00$
For axial compression	$\phi_c = 0.90$

Girder Design

2. Select Trial Girder



Girder Design

3. Compute Section Properties

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	-

Girder Design

3. Compute Section Properties

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	-

Girder Design

4. Compute Dead Load Effects

Concrete Deck- Dead Load per Unit Length

DL_{deck} = 1.05 k/ft

Stay-in-Place Forms- Dead Load per Unit Length

$DL_{deckforms}$ = 0.122 k/ft

Miscellaneous Dead Load per Unit Length

DL_{misc} = 0.015 k/ft

Concrete Parapets - Dead Load per Unit Length

DL_{par} = 0.167 k/ft

Future Wearing Surface- Dead Load per Unit Length

DL_{fws} = 0.292 k/ft

Girder Design

5. Compute Live Load Effects

- Performing an analysis and using Lever Rule yields

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

Girder Design

5. Calculate Design Shear and Moments

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

From Staad.pro

Girder Design

5. Calculate Design Shear and Moments

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

Girder Design

5. Calculate Design Shear and Moments

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

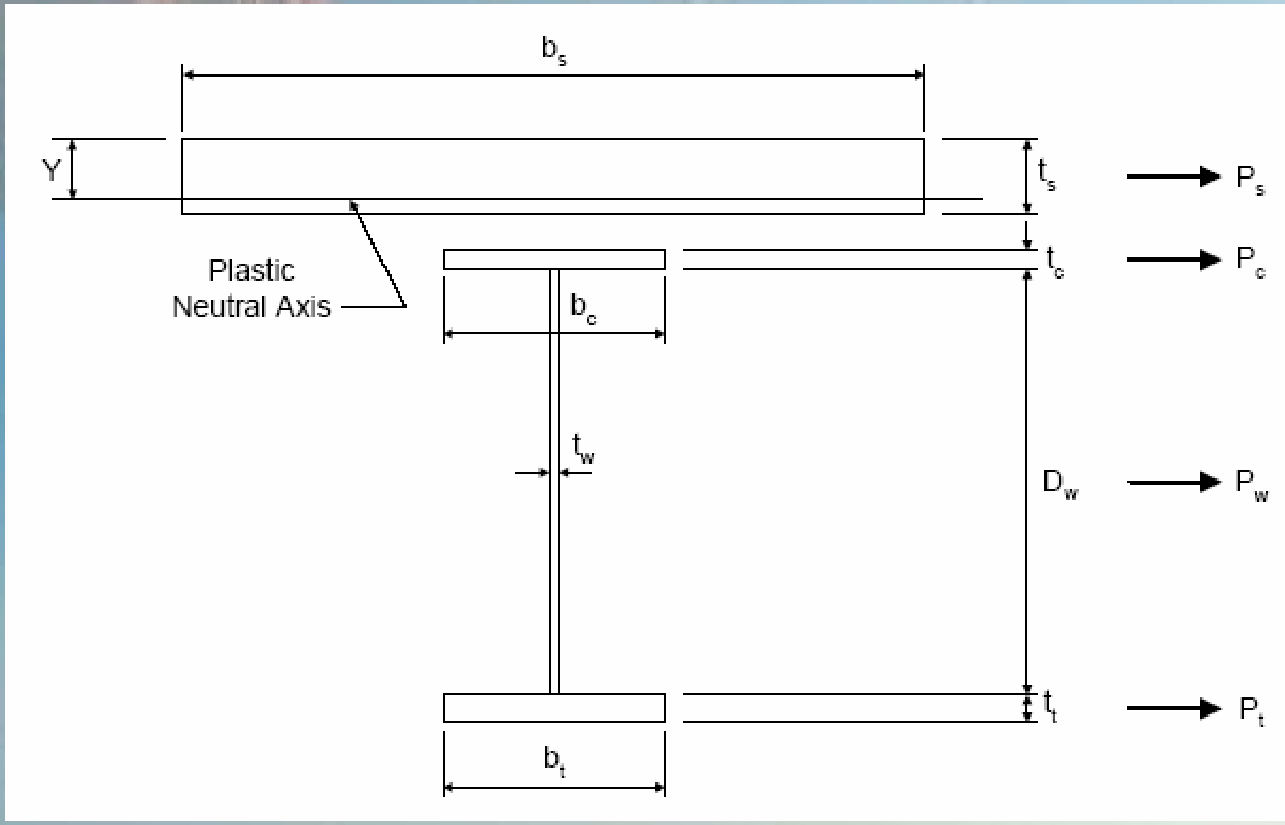
Girder Design

5. Calculate Design Shear and Moments

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

Girder Design Checks - Positive Region

- Section Proportion Limits
- Plastic Moment Capacity



-Location of plastic neutral axis: Y = 6.30”

Therefore, Mp = 7677 k-ft

Girder Design Checks - Positive Region

- Compact or Non-compact???
  - Flange and Web were both found to be compact
  - Therefore, section is considered compact for positive moment region

Girder Design Checks - Positive Region

- Design for flexure – Strength L.S.

$$\sum \eta_i \gamma_i M_i \leq M_r$$

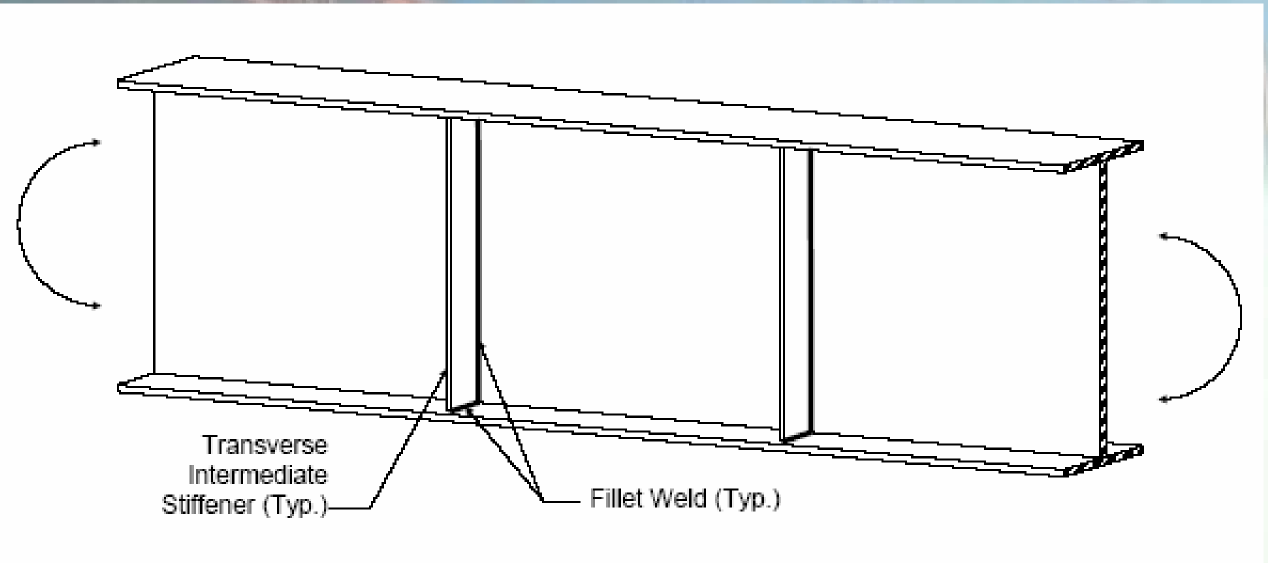
$$M_r = \phi_f M_n = 7677k - ft$$

$$\sum \eta_i \gamma_i M_i = 6572k - ft$$

Therefore, girder is adequate for positive moment region

Girder Design Checks - Positive Region

- Design for flexure – Fatigue L.S.



$$f_{botgdr} \leq \Delta F_n$$

$$3.43ksi \leq 6ksi$$ **(OK)**

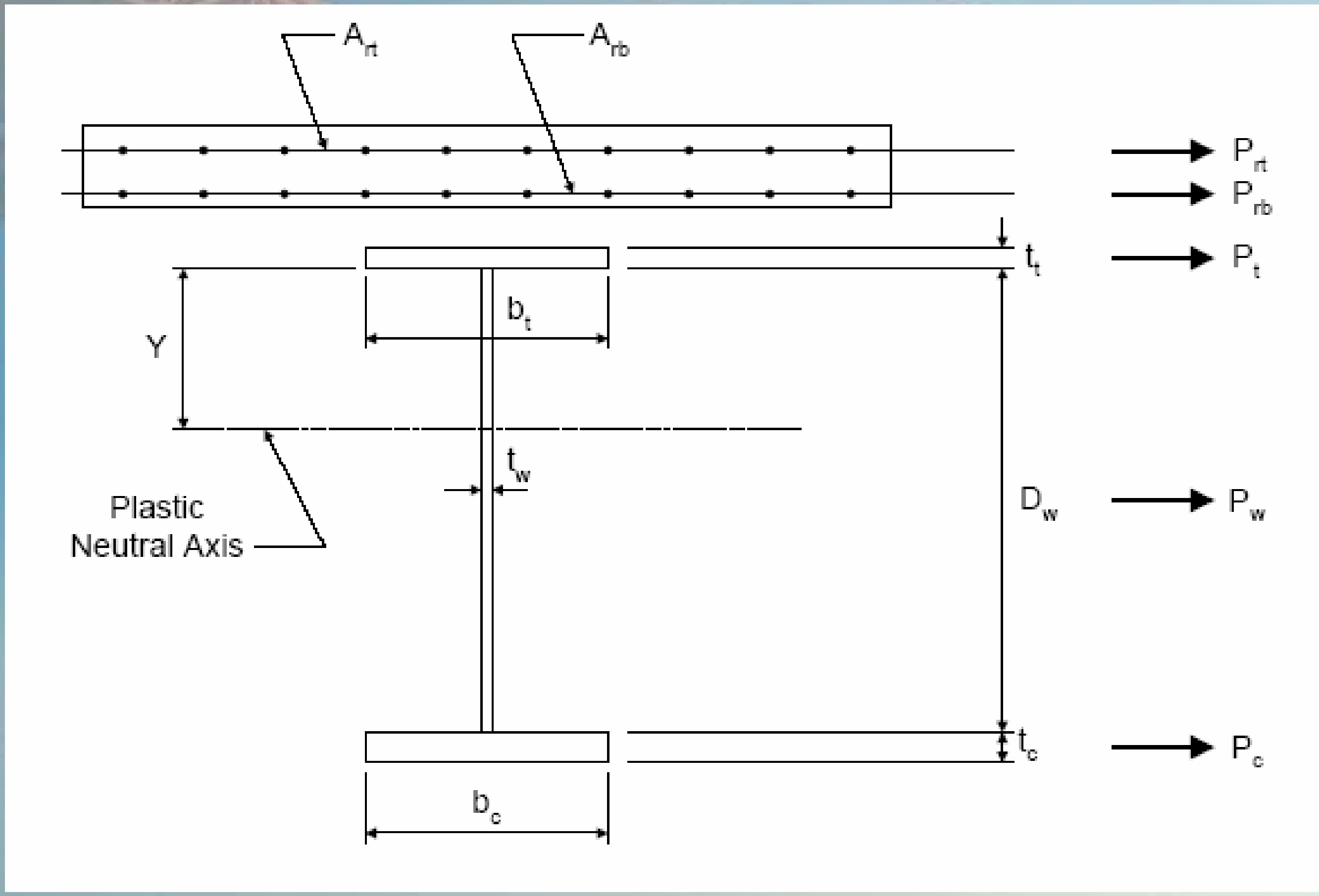
Girder Design - Positive Region

– Other Checks performed:

- Service L.S.
- Constructability
- Lateral Torsional Buckling

Girder Design Checks - Negative Region

- Section Proportion Limits
- Plastic Moment Capacity



Girder Design Checks - Negative Region

- Compact or Non-compact???
  - Web was found to be non-compact
  - Therefore, section is considered to be non-compact for negative moment region

Girder Design Checks - Negative Region

- Design for flexure – Strength L.S.
  - Nominal resistance is based on LTB

$$\sum \eta_i \gamma_i F_i \leq F_r$$

$$22.02ksi \leq 50ksi$$ **(OK)**

Girder Design Checks - Negative Region

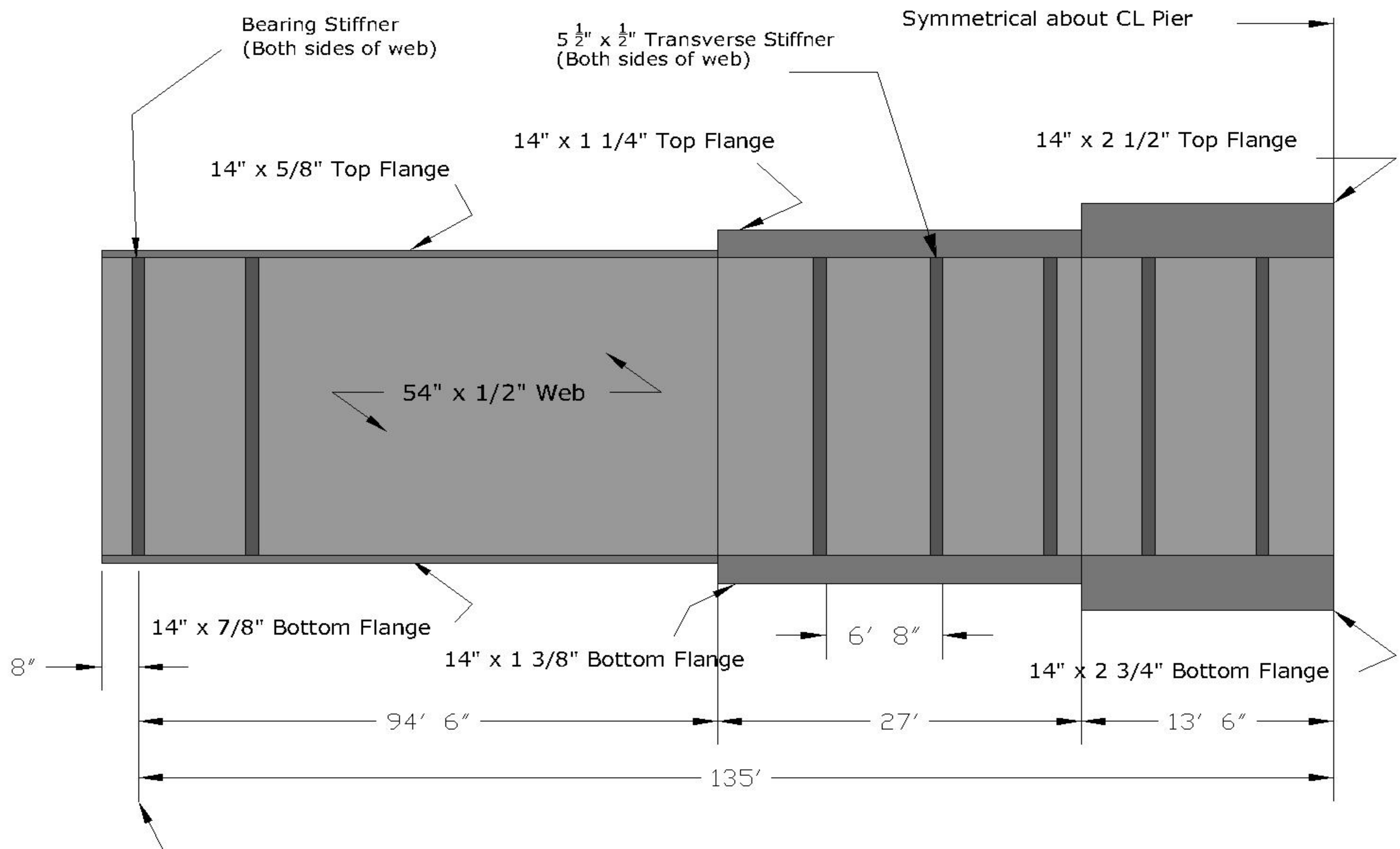
- Other checks performed
  - Shear
  - Fatigue
  - Service
  - Constructability

Check Wind Effects on Girder Flanges

- Wind speed = 130 mph

$$F_u + F_w \leq F_r$$

$$-22.9ksi + 0.043ksi \leq 50ksi \qquad \textbf{(OK)}$$



TITLE			SHEET NO.
Girder Design	Not to Scale		1/1
			DATE 20 APR 06
PROJECT	COURSE	INSTRUCTOR	GROUP NO.
I-10 LAKE PONTCHARTRAIN BRIDGE ("TWIN SPANS")	CE 4460	CAI	5
	GROUP MEMBERS	ALLAIN, BEYER, KOCKE, WHEELER	

Final Steel Girder Design

Rust-Proofing Solution

- All steel components will be hot-dipped galvanized.
- Protection will last for 70 years
- Specified according to ASTM A 123/A 123M

Bolted Field Splice Design

Steps

- Identify Splice Locations
- Compute Girder Moments at Splice
- Compute Flange Splice Design Loads
- Design Flange Splice
- Compute Web Splice Design Loads
- Design Web Splice
- Final Splice Design

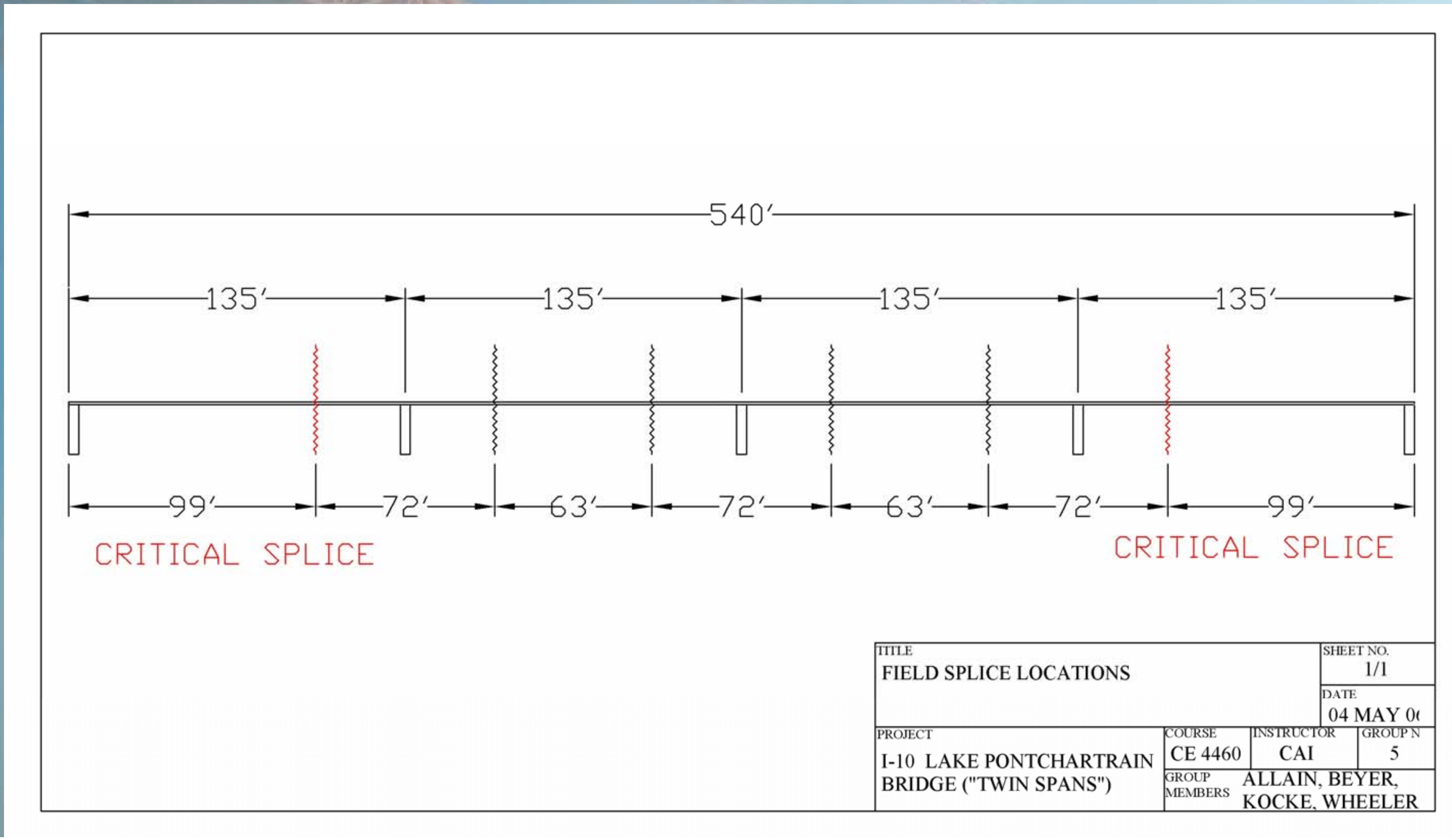
Bolted Field Splice Design

Identify Splice Locations

- Conditions for splice locations:
  - Splices must be positioned such that the girder size meets shipping restrictions.
    - Girders can be barged in – minor concern.
  - Splices should be placed near the point of dead load contraflexure.
  - Splices should be located where total moment is relatively small.

Bolted Field Splice Design

Identify Splice Locations



Bolted Field Splice Design

Compute Girder Moments at Splice

- STAAD.Pro
- Design for the worst case.
    - Three symmetrical pairs of splices.
    - Outer splices are the critical splices.
- Controlling Positive Moment L.S. is Strength I
- Controlling Negative Moment L.S. is Fatigue

Bolted Field Splice Design

Compute Flange Splice Design Loads

- Dependent on shape properties.
- Controlling stresses did not meet minimum design loads.
  - Bottom flange: F_{cf} = 34.8 ksi < 37.5 ksi
  - Top flange: F_{cf} = 25.2 ksi < 37.5 ksi
  - Use 37.5 ksi for both flanges.

Bolted Field Splice Design

Design Flange Splice

- Trial splice.
- Check yielding and fracture of splice plates.
  - For tension, compression, and block shear.
- Check bolts for:
  - Shear
  - Minimum spacing
  - Maximum spacing for sealing
  - End spacing
  - Edge distance

Bolted Field Splice Design

Compute Web Splice Design Loads

- Must use girder shear forces
  - Controlling shear is Strength I: V_u = 242.9 k
- Moment effects:
  - Moment in the web splice plates
  - Axial force, H_w, due to eccentricity of the shear force.
  - $M_T = M_{uw} + M_{uv} = M_w + V_{uw}e$ = 336.2 k-ft

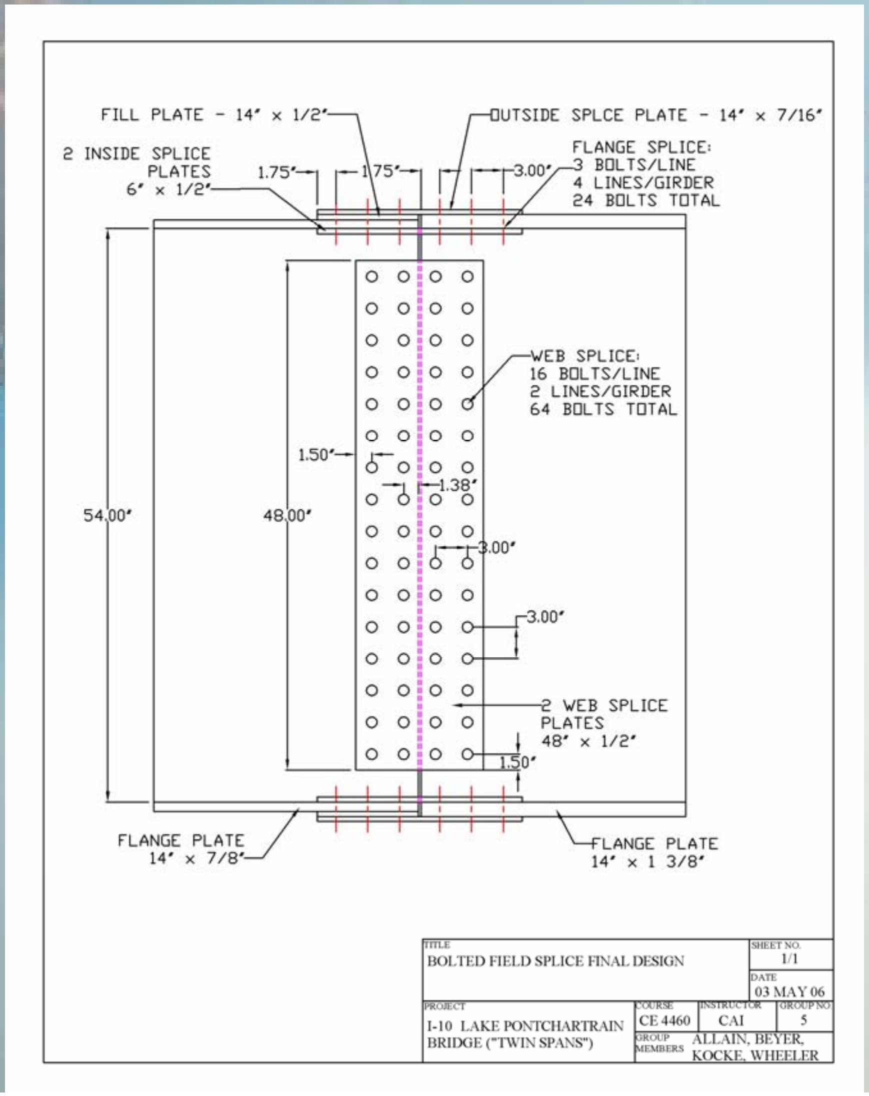
Bolted Field Splice Design

Design Web Splice

- Check bolts for:
  - Minimum spacing
  - Maximum spacing
  - Edge distance
  - Shear
  - Vertical moment on extreme bolt
  - Horizontal moment on extreme bolt
- Shear yielding and block shear rupture of splice plates
- Flexural yielding of splice plates

Bolted Field Splice Design

Final Splice Design



Substructure Design

- Determine design method for pier cap (cast-in-place reinforced concrete)
- Establish Width of Pier Cap = 62.5’
- Compile Force Effects on Substructure
- Analyze Structure and Compile Load Combinations
- Design Cap
- Design Foundations (Structural Considerations)

Live loads from the superstructure

- Maximum truck = 71.78 k
- Minimum truck = 0.0 k
- Maximum lane = 98.71 k
- Minimum lane = 0.0 k

(Obtained from the girder live load analysis to obtain the maximum unfactored live load reactions for the interior and exterior girder lines.)

Maximum Load Effects on Cap

- Braking force was considered to be negligible for pier cap design
- Load factors used for Strength I: 1.25 for DC, 1.50 for DW, and 1.75 for LL+IM

	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

Forces on Substructure

- Wind
- Water
- Scour
- Temperature
- Shrinkage
- Ship Collision
- Braking Force

Pier Cap

- Design Criteria

Concrete Strength	4 ksi
β_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.

Pier Cap Design Steps

- Flexural resistance
- Maximum positive moment
- Maximum negative moment
- Check for min. temperature and shrinkage steel
- Skin reinforcement
- Maximum shear
- Also checking for:
    - Limits for reinforcement
    - Flexural reinforcement
    - Service load applied steel stress

Pier Cap Flexural Resistance (S5.7.3.2)

- $M_r = \varphi M_n$

  where:

M_r = factored flexural resistance

Φ = flexural resistance factor = 0.9

Compression reinforcement is neglected in the calculation of the flexural resistance.

Check positive moment resistance (bottom steel)

$$M_n = A_s * f_y * (d_s - a/2) \qquad \text{(S5.7.3.2.2-1)}$$

Where:

D_s = cap depth – CGS_b

CGS_b = cover + stirrup dia. + ½*bar dia.

$A_s = (n_{bars\ tension})(A_{s\ bar})$

$a = (A_s * f_y)/(0.85 * f_c' * b)$

M_n = 1,511 k-ft

Therefore the factored flexural resistance, M_r, can be calculated as follows: $M_r = 0.9 * M_n$ = 1099.5 k-ft

$M_r > M_u$ then it is O.K.

Limits for Reinforcement

$c/d_e < 0.42$ (S5.7.3.3)

where:

$c = a / \beta_1$

$d_e = d_s$

Plugging in:

$c/d_e = 0.070 < 0.42$ then it is O.K.

Check Minimum Reinforcement Requirements (S5.7.3.3.2)

$$1.2\ M_{cr} = 1.2\ f_r\ S$$

where:

$$f_r = 0.24\ (f_c')^{0.5}$$

$$S = bh^2/6$$

$1.2M_{cr}$ = 884.74 k-ft. OR $1.33\ M_u$ = 1462.3 k-ft.

Minimum required section resistance = 884.74 k-ft.

Provided section resistance = M_r + 1360 k-ft. > 884.74 k-ft then it is O.K.

Check the flexural reinforcement distribution (S5.7.3.4)

$f_{s,allow} = Z/[(d_c A)^{1/3}] < 0.6\ f_y$ (S5.7.3.4-1)

where

Z = crack width parameter (k/in.)

dc = clear cover + stirrup dia. + ½ bar dia.

$A = 2^*\ d_c{}^*\text{cap width}) / n_{bars}$

$f_{s,allow} = 33.5$ ksi $< 0.6(60) = 36$ ksi

so it is OK

Check service load applied steel stress, fs, actual

$I_{transformed} = A_{ts}(d_s - y)^2 + (b^*y^3)/3$

$F_{s,actual} = (M_s c/I)n$

$= 33.03 \text{ ksi} < f_s, \text{allow} = 36 \text{ ksi}$

so it is OK

Maximum Negative Moment Resistance (Top Steel)

- Check negative moment resistance
- Find limits for reinforcement
- Check minimum reinforcement
- Check flexural reinforcement distribution
- Check the service load applied steel stress, f_s, actual

Check minimum temperature and shrinkage steel

$$A_{s,\ min1} = 0.11\ A_g/f_y$$
$$= 4.2\ \text{in.}^2$$

This area is to be divided between two faces (2.1 in.^{2} per face).

Skin Reinforcement

$$A_{sk} > 0.012\,(d_e - 30\,) < (A_s + A_{ps})/4$$

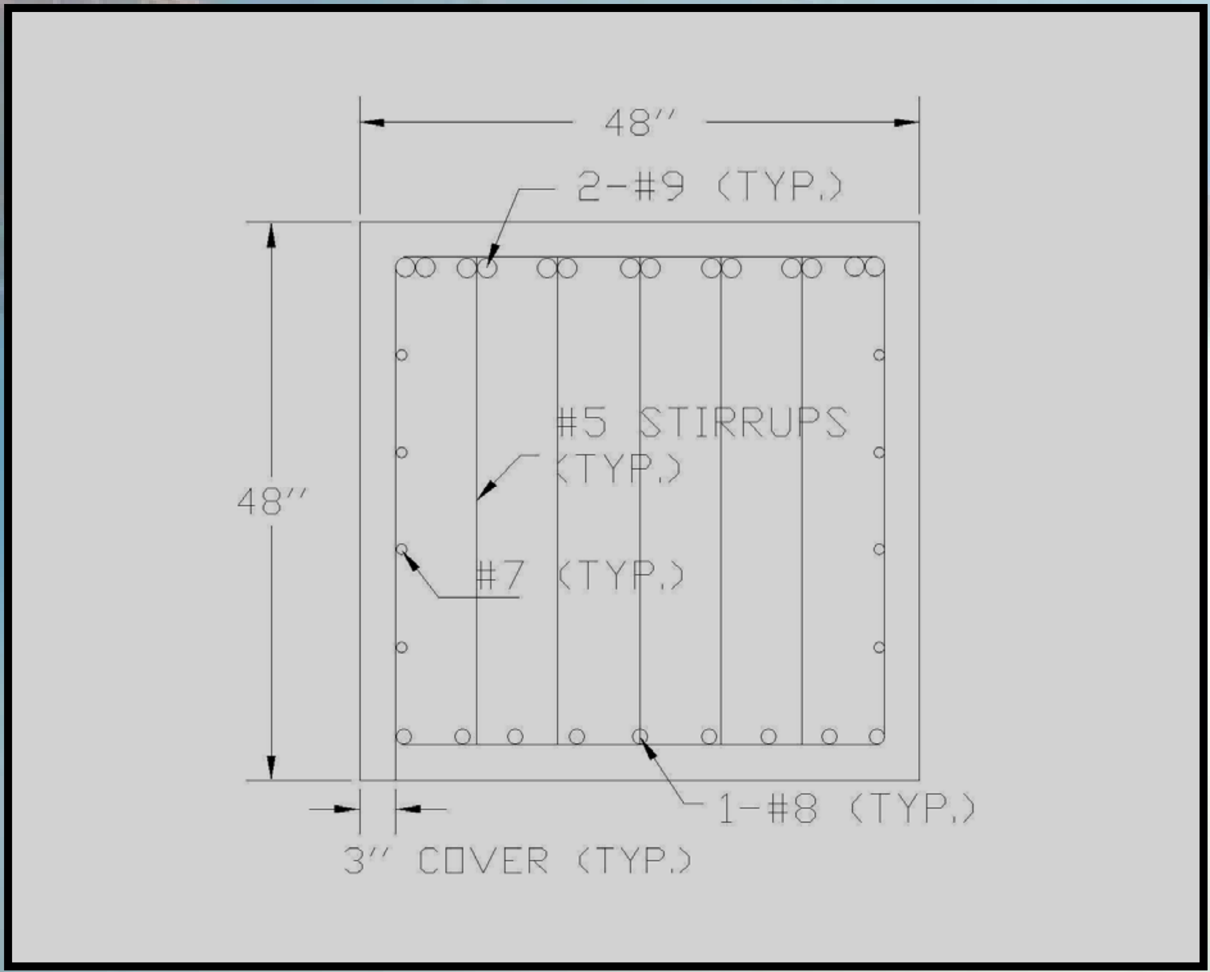
A_{sk} = 0.1545 in.^{2}/ft < 14.0/4 = 3.5 in.^{2}/ft

Required A_{sk} per face = 0.154(4)

= 0.618 in.^{2} < 2.4 in.^{2}

as provided, so OK

Cap Cross-Section



Maximum Shear

Shear, $V_u = 705.4$ k

$V_r = \varphi V_n$

V_n is the lesser of:

$V_n = V_c + V_s + V_p = 906.8$ k

OR

$V_n = 0.25\, f_c'\, b_v\, d_v + V_p = 1977.6$ k

Therefore use $V_n = 906.8$ k

$V_r = (0.9)(906.8)$

$= 816.12 \text{ k} > V_u = 705.4$ k

OK

Check the minimum transverse reinforcement (S5.8.2.5)

$A_v = 0.0316\ (f_c')\ 0.5\ b_v\ S\ /\ f_y$ (S5.8.2.5-1)

where

b_v = width of the web = 48 in.

Av = 0.354 in.^{2} < 1.86 in.^{2} provided OK

Check the maximum spacing of the transverse reinforcement

If $v_u < 0.125\, f_c'$, then $S_{max} = 0.8d_v < 24.0$ in.

If $v_u > 0.125\, f_c'$, then $S_{max} = 0.4d_v < 12.0$ in.

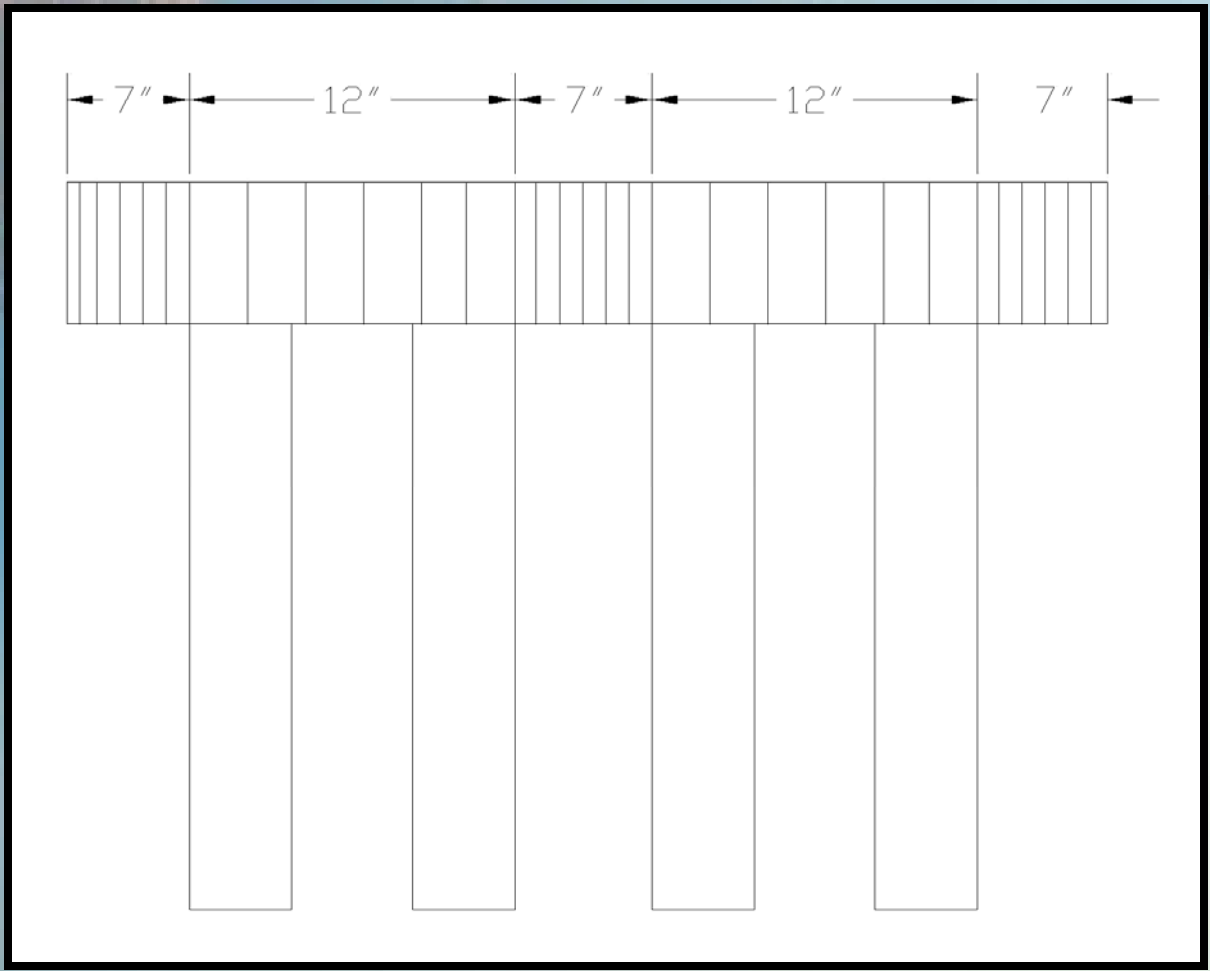
$v_u = V_u/\varphi b_v d_v = 0.396$ ksi $< 0.125\,(4) = 0.5$ ksi

$S_{max} = 0.8d_v = 32.96$ in. however since S_{max} cannot exceed 24'', use 24'' as a maximum.

$S_{actual} = 7$ in. < 24 in. OK

$S_{actual} = 12$ in. < 24 in. OK

Stirrup Distribution in the Bent Cap



Piles

- Dimensions : 5'-6'' diameter, 100' length
- Maximum slenderness ratio:

L/d < 20 should be maintained

for 5'-6'' piles and 100' pile lengths:

100'/5.5' = 18.18 < 20 OK

Pile lengths:

Elevation of bridge section = 38.17'

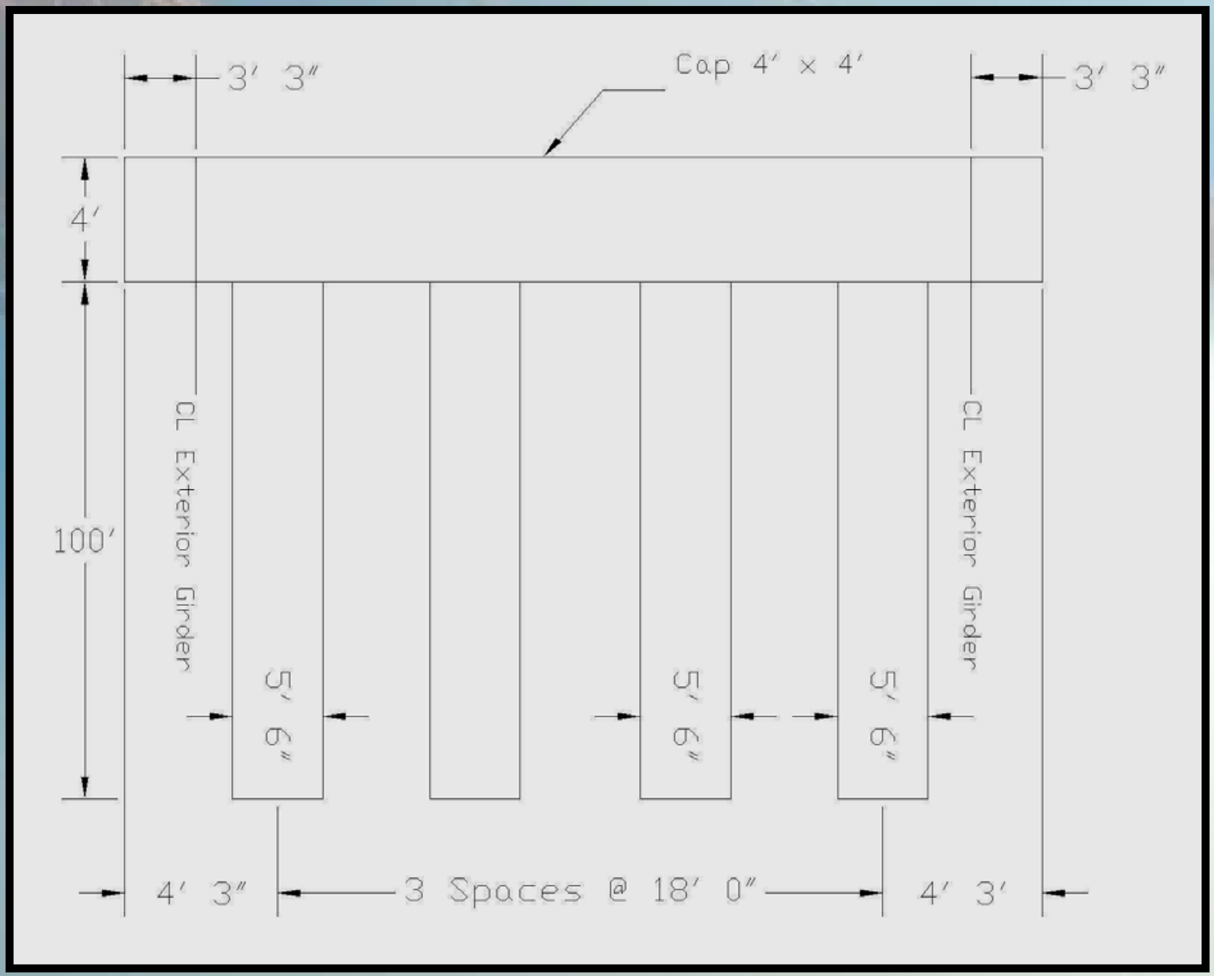
Scouring preliminary depth = 5'

Ground level = -15' (below sea level)

Therefore the piles should be at least 38.17 + 15 + 5 = 58.17' long

However taking into account the bad soils (clays) of the area, a 100' pile is preferable.

Substructure



Questions?