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Structural Calculations

POWER ROAD BRIDGE OVER THE RWCD FLOODWAY

Prepared for
THE FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
by
CANNON & ASSOCIATES, INC.
CONSULTING ENGINEERS

March 4, 1985

Cannon & Associates, Inc.

Consulting Engineers
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Structural Calculations
POWER ROAD BRIDGE
OVER THE RWCD FLOODWAY

Bridge Consultant:	CANNON & ASSOCIATES, INC. CONSULTING ENGINEERS
Roadway Consultant:	THE WLB GROUP
Geotechnical Consultant:	SERGENT, HAUSKINS & BECKWITH

D E S I G N S U B M I T T A L

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CA Project No. 83-58
March 4, 1985
Sheet 1

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Consulting Engineers
Tucson, Arizona

Basis For Design

Ref. - 1) 'Scope of Work' by Maricopa Co. Highway Dept
Rec'd Feb 24, 1984
2) 'Std. Spec. For Highway Bridges' by AASHTO, 1977, Updated
3) Technical Advisories - Referenced When Used
4) MAG, 'Uniform Standard Specifications for Public Works Construction', 1979

Loading:

A) Design to Accomodate HS20-44 Loading
B) Calculation based on Service loads & the Working Stress Method
C) Thermal Range: Rise = 30°F
Fall = 40°F
Mean = 70°F
D) Seismic Zone: AASHTO Risk Zone 2
E) Stream Flow: V = 7.0 ft/s, d = 7.2', Q = 4900 cfs
F) Allowance for 25 psf future wearing surface
G) Scour at Piers

Materials:

A) Concrete:

Superstructure	f'_c = 4000 psi
Substructure	f'_c = 3000 psi
Drilled Shafts	f'_c = 3000 psi
All other Concrete	f'_c = 3000 psi

B) Steel

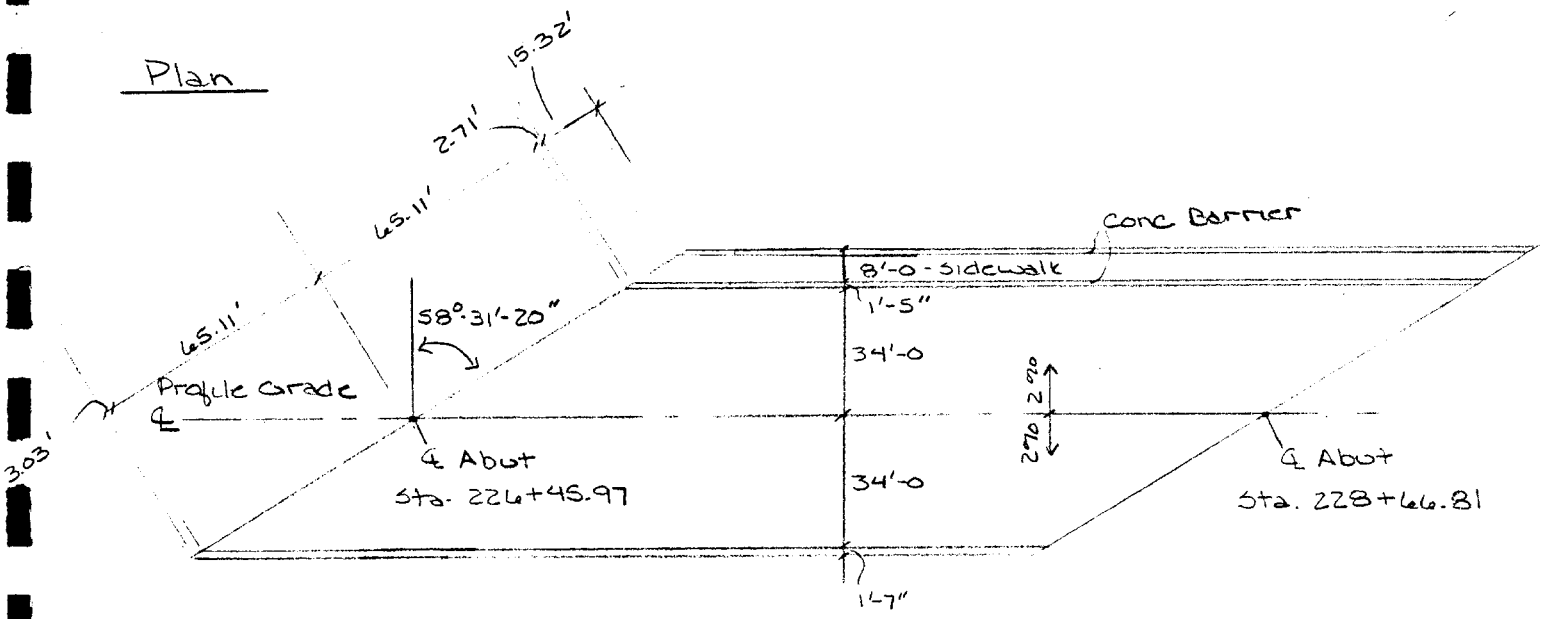
Bars #6 and smaller - Grade 40	f_s = 20,000 psi
Bars #7 and larger - Grade 60	f_s = 24,000 psi

Basis of Design

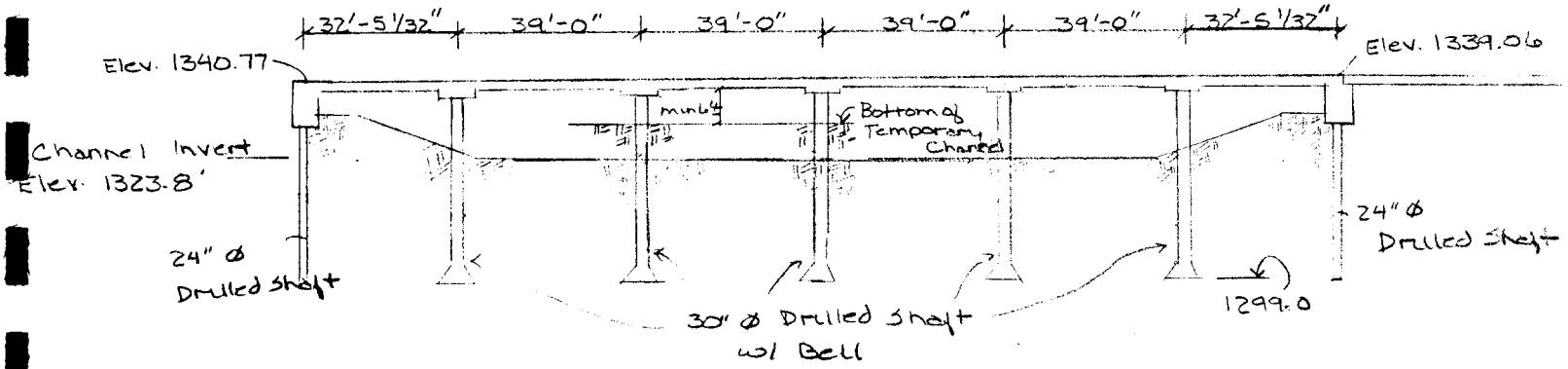
A) Pier Columns are pinned to girders - slab continuous
B) Integral Abutment - Abutment cap is pinned to caissons
C) Scour Elev @ 1313.8 (10' below channel invert)

Configuration

Plan



Elevation @ ℄ Profile Grade



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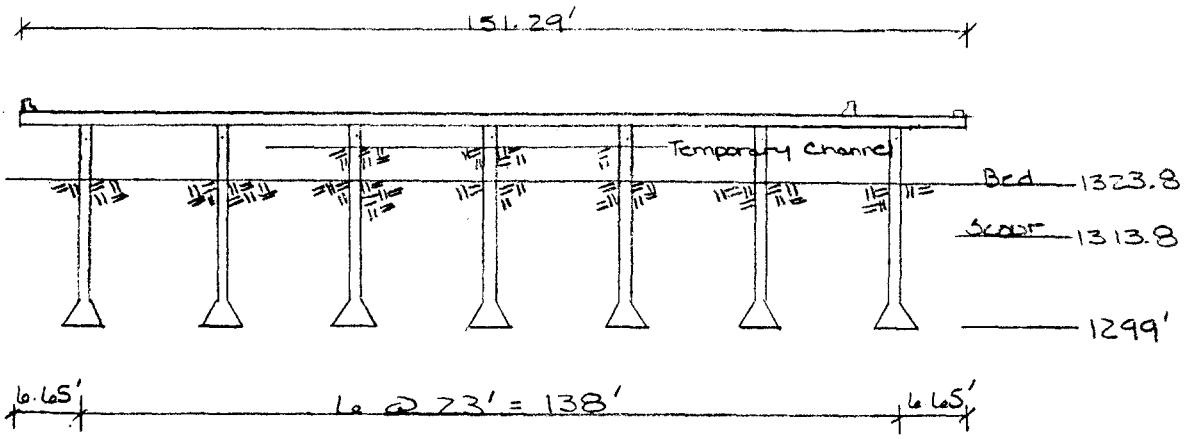
Pier Cap Elevation



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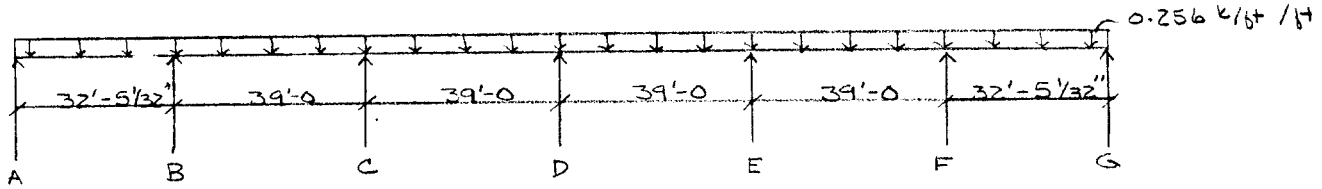
Deck Design

Deck Design

- Use 18½" thick deck slab
- Reinforcement Parallel to Traffic

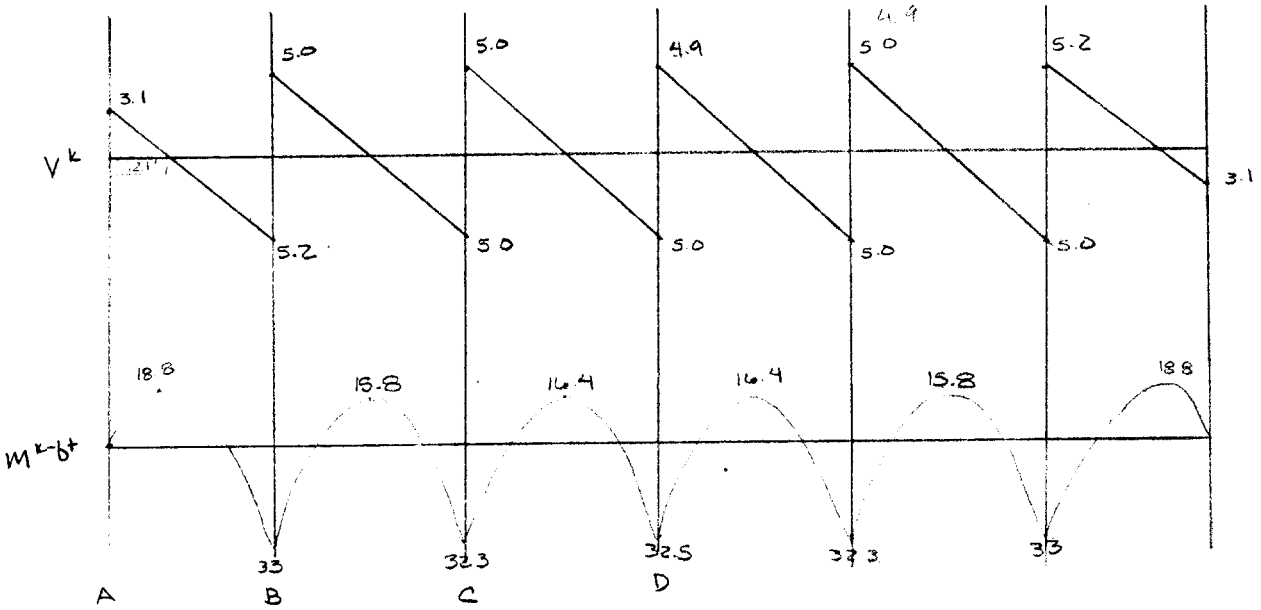
Dead Load: Wearing Surface: = 0.025 ksf
Slab: (.150)(18.5/12) = 0.231 ksf
0.256 ksf



Three Mom Eq.: $M_A l_1 + 2M_B(l_1+l_2) + M_C l_2 = -\frac{w}{4}(l_1^3 + l_2^3)$

$M_A = M_G = 0$; $M_B = M_F = -33.0^{k\text{-}ft}$; $M_C = M_E = -32.3^{k\text{-}ft}$; $M_D = -32.5^{k\text{-}ft}$

$R_A = R_G = 3.1^k$; $R_B = R_F = 10.2^k$; $R_C = R_E = 10.0^k$; $R_D = 9.9^k$



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Deck Design

Live Load:

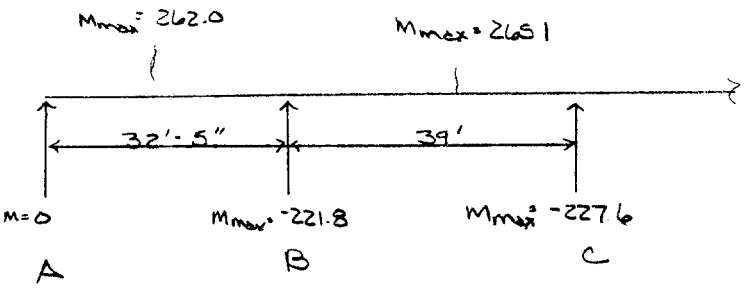
Ref. 'Moments Shears and Reactions For Continuous Highway Bridges'

Assume LL Reactions like those calculated for 4 Spans (conservative)

$N = 39/32.42 = 1.20$ - use Table 4.2 - See Appendix

Total Length = $2(32.42 + 39) = 143'$

Max. Moments:



Impact Coefficient = 0.300 (AASHTO 1.2.12)

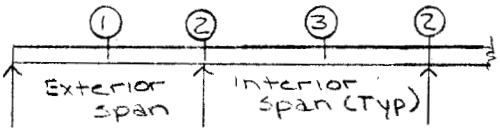
Distribution of Lane Loads (AASHTO 1.3.2(c))

$2E = 2(4 + 0.06(32.42)) = 11.9'$ (32.42' span)

$= 2(4 + 0.06(39)) = 12.7'$ (39' span)

Deck Design

DL+LL+I (Moments/ft. width)



① Exterior Span

DL = 18.8 k-ft

LL+I = 1.3(262/11.9) = 28.6 k-ft

DL+LL+I = 47.4 k-ft

② Support

DL = -33.0 k-ft

LL+I = 1.3(-221.8/11.9) = -24.2 k-ft

DL+LL+I = -57.2 k-ft

③ Interior Span

DL = 16.4 k-ft

LL+I = 1.3(265.1/12.7) = 27.1 k-ft

DL+LL+I = 43.5 k-ft

Use spacing of bars = 6" ; h = 18½"

① Exterior Span - Positive Mom Reinf. Use #8 & #10 alternating @ 6" o.c.

M = 47.4 k-ft Try #8 & #10 /12" $A_s/ft = (0.79+1.27) = 2.06 \text{ in}^2/ft$

$d = 18\tfrac{1}{2}'' - \tfrac{1}{2}'' - 1'' - 1.27''/2 = 16.4''$ Use d = 16" $n = \frac{29000}{57\sqrt{4000}} = 8.0$

$n\frac{A_s}{bd} = 8\frac{2.06}{(12)(16)} = 0.0858$ k = 0.337 j = 0.887

$\text{req'd } d = \sqrt{\frac{M}{\frac{1}{2}f_c k j b}} = \sqrt{\frac{47.4(12)}{\frac{1}{2}(1.6)(.337)(.887)(12)}} = 14.1'' < 16''$ ∴ O.K.

$F_s = \frac{M}{A_s j d} = \frac{47.4(12)}{(2.06)(.887)(16)} = 19.4 \text{ ksi} < 24.0 \text{ ksi}$

$F_c = \frac{kF_s}{n(1-k)} = \frac{0.337(19.4)}{8(1-.337)} = 1.23 \text{ ksi} < 1.60 \text{ ksi}$ ∴ O.K.

Deck Design

② Support - Negative Mom Reinf Use #10 @ 6"

$M = -57.2^{k\text{-}ft}$ Try #10 @ 6" $A_s = 2(1.27) = 2.54 \text{ in}^2$

$d = 18\tfrac{1}{2} - 2 - 1.27/2 = 15.9''$ Use $d = 15''$ $n = 8$

$n \frac{A_s}{bd} = 8 \frac{2.54}{(12)(15.)} = 0.1129$ $k = 0.376$; $j = 0.875$

Req'd $d = \sqrt{\frac{57.2(12)}{\tfrac{1}{2}(1.6)(.376)(.875)(12)}} = 14.74'' < 15''$ ∴ OK

$f_s = \frac{M}{A_s j d} = \frac{57.2(12)}{(2.54)(.875)(15)} = 20.6 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.376(20.6)}{8(1-0.376)} = 1.55 \text{ ksi} < 1.60 \text{ ksi}$ ∴ OK

③ Interior Span - Positive Mom Reinf Use #10 @ 6"

$M = 43.5^{k\text{-}ft}$ Try #10 @ 6" $A_s = 2.54 \text{ in}^2$ $d = 16''$

$n \frac{A_s}{bd} = 8 \frac{2.54}{(12)(16)} = 0.1058$ $k = 0.3666$; $j = 0.878$

$f_s = \frac{M}{A_s j d} = \frac{43.5(12)}{(2.54)(.878)(16)} = 14.6 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.3666(14.6)}{8(1-0.3666)} = 1.05 \text{ ksi} < 1.60 \text{ ksi}$ ∴ OK

Required Transverse Reinforcement (AASHTO 1.3.2(E), 1.5.12)

Bottom of Slab: - Use #5 @ 9"

Distribution Reinf. $\% = \frac{100}{\sqrt{S}} = \frac{100}{\sqrt{32.42}} = 17.6\%$ (32.42' span)

$= \frac{100}{\sqrt{39}} = 16.0\%$ (39' span)

$.176 \left(\frac{19.4}{24}\right) 2.06 \text{ in}^2 = 0.29 \text{ in}^2/\text{ft}$ Use #5 @ 9"

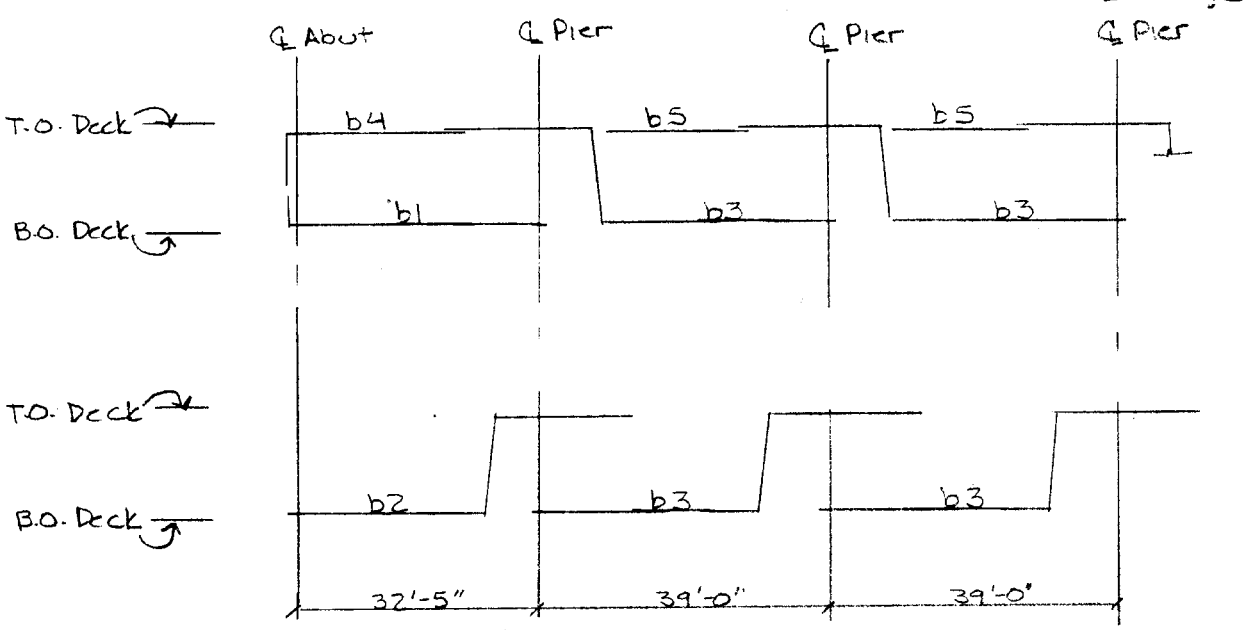
$.16 \left(\frac{14.6}{24}\right) 2.54 \text{ in}^2 = 0.25 \text{ in}^2/\text{ft}$

Top of Slab - Distr. Reinf. Use #4 @ 18"

Deck Design

Rebar bend (cutoff)

Rebar Placing Diagram



bar	size	spacing
b1	#8	12"
b2	#10	12"
b3	#10	6"
b4	#6	12"
b5	#6	12"

} b1 & b2 alternate @ 6" o.c.

AASHTO 1.5.7 (D) min $\rho = 0.002$

Neg. Reinf. Req $A_s/ft = 0.002(16")(12") = 0.38\ in^2/ft$

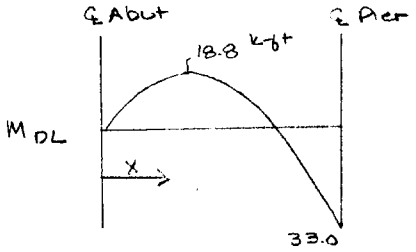
use #6 (b4 & b5) @ 12" o.c $A_s = 0.44\ in^2/ft$

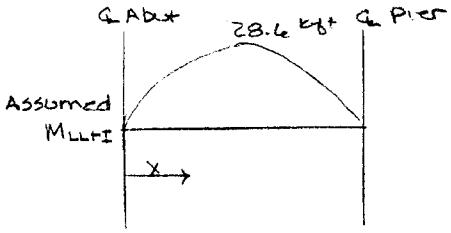
Deck Design

Bar b2

Pos. Reinf.:

Max M @ cutoff $= \left(\frac{0.79}{2.06}\right) 47.4 \left(\frac{24.0}{19.4}\right) = 22.5^{k\text{-}ft}$





$M_{DL} = 3.1x - .13x^2$ $\qquad$ $M_{LL} \approx 3.53x - 0.11x^2$

$M_{D+L+I} \approx 6.63x - .24x^2 = 22.5^{k\text{-}ft}$ @ $x = \dfrac{6.63 \pm \sqrt{(6.63)^2 - 4(.24)(22.5)}}{2(-.24)} = 3.96', 23.7'$

Bend b2 @ $23.7' + 32.42/20 = 25.3'$ min. from ℄ Abut

Neg. Reinf:

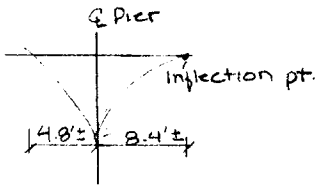
Assume LL Mom Distr = DL Mom Distr

Max M_{DL} @ cutoff $= \frac{1}{2}(-33.0)\left(\frac{24.0}{20.6}\right) = -19.2^{k\text{-}ft}$

$M_{DL} = 3.1x - .13x^2 = -19.2^{k\text{-}ft}$ @ $x = \dfrac{3.1 \pm \sqrt{(3.1)^2 + 4(.13)(19.2)}}{2(.13)} = 28.9'$

Bend b2 @ $32.4' - 28.9' + 32.42/20 = 5.1'$ min from ℄ Pier

AASHTO 1.5.13 C) - Extend b2 past inflection pt. + $\ell/20$



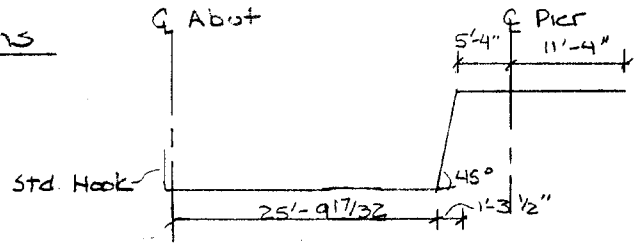
Inflection pt @ $M_b = 0 = 5.0x - .128x^2 - 33$ @ $x = 8.4'$

Extend b2: $8.4' + 39/20 = 10.4'$ min. from ℄ Pier - Controls

or ℓ_d past b3 cutoff $= 2.9' + 54''/12 = 7.4'$ min.

AASHTO 1.5.13 A)

Bar b2 Dimensions



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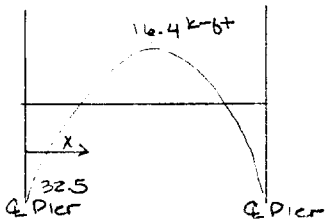
12/31/84

Deck Design

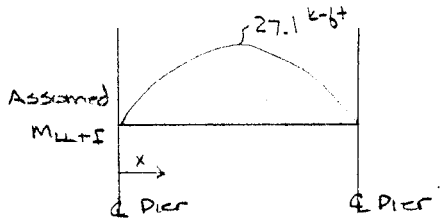
Bar b3

Pos. Reinf

Max M @ cutoff = $\frac{1}{2}(43.5^{k\text{-}ft})\left(\frac{24.0}{14.6}\right) = 35.7^{k\text{-}ft}$



$M_{DL} = 5.0x - 0.128x^2 - 32.5$



$M_{LL+I} \approx 2.8x - 0.07x^2$

$M_{D+L+I} = 7.8x - 0.2x^2 - 32.5 = 35.7$ @ $x = \frac{7.8 \pm \sqrt{(7.8)^2 - 4(.2)(68.2)}}{2(.2)} = 13.2, 25.8'$

Bend b3 @ $25.8' + 39/20 = 27.75'$ min. from ℄ Pier

Neg. Reinf.

Assume LL Mom Distr = DL mom Distr

Max M_{DL} @ cutoff = $-19.2^{k\text{-}ft}$ (as bar b2)

$M_{DL} = 5.0x - 0.128x^2 - 32.5 = -19.2$ @ $x = \frac{5.0 \pm \sqrt{5^2 - 4(.128)(13.3)}}{2(.128)} = 2.9'; 36.2'$

Bend b3 @ $2.9' + 39/20 = 4.85'$ min from ℄ Pier

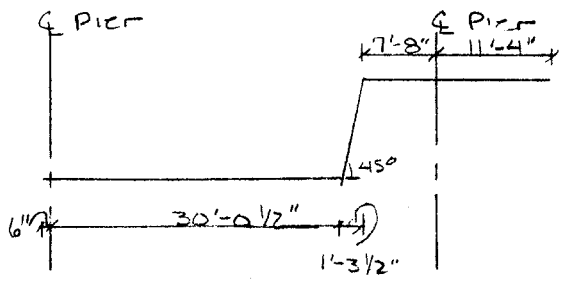
AASHTO 1.5.13 - Extend b3 past inflection pt

$l = 8.4' + 39/20 = 10.4$ min. from ℄ Pier ← controls

or l_d past b2 cutoff

$l = 3.5' + 54/12 = 8'$ min from ℄ Pier

Bar b3 Dimensions



Deck Design

DL Camber - Calc of EI

$E_c = 57\sqrt{4000} = 3605$ ksi

AASHTO (1.5.23(G))

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$I_g = 12''(18'')^3/12'' = 5832 \text{ in}^4$

$M_{cr} = f_r I_g / y_T = 7.5\sqrt{4000}(5832)/9 = 307{,}400$ #-in

Pos. M- Ext. Span: (calc. for $M_a = 47.4^{k\text{-}ft}$ (DL+LL+I))

$I_{cr} = \frac{bc^3}{3} + n A_s (d-c)^2$ $n = 8$, $A_s = 2.06$; $c = kd = .337(16) = 5.4''$

$= \frac{12(5.4)^3}{3} + 8(2.06)(16-5.4)^2 = 2482 \text{ in}^4$

$$I_e = \left(\frac{307{,}400}{47.4(12000)}\right)^3 5832 + \left[1 - \left(\frac{307{,}400}{47.4(12000)}\right)^3\right] 2482 = 3011 \text{ in}^4$$

Neg M- Support: ($M_a = 57.2^{k\text{-}ft}$)

$A_s = 2.54 \text{ in}^2$; $c = 0.376(15) = 5.64''$ $I_{cr} = \frac{12(5.64)^3}{3} + 8(2.54)(15-5.64)^2 = 2498 \text{ in}^4$

$$I_e = \left(\frac{307{,}400}{57.2(12000)}\right)^3 5832 + \left[1 - \left(\frac{307{,}400}{57.2(12000)}\right)^3\right] 2498 = 2797 \text{ in}^4$$

Pos M- Int. Span ($M_a = 43.5^{k\text{-}ft}$)

$A_s = 2.54 \text{ in}^2$; $c = 0.366(16) = 5.86''$ $I_{cr} = \frac{12(5.86'')^3}{3} + 8(2.54)(16-5.86)^2 = 2894 \text{ in}^4$

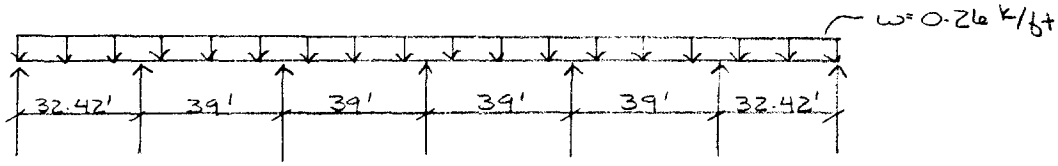
$$I_e = \left(\frac{307{,}400}{43.5(12000)}\right)^3 5832 + \left[1 - \left(\frac{307{,}400}{43.5(12000)}\right)^3\right] 2894 = 3494 \text{ in}^4$$

Exterior Span - use $I_e = 1/2(3011 + 2797) = 2904 \text{ in}^4$
Interior Span - use $I_e = 1/2(3494 + 2797) = 3146 \text{ in}^4$

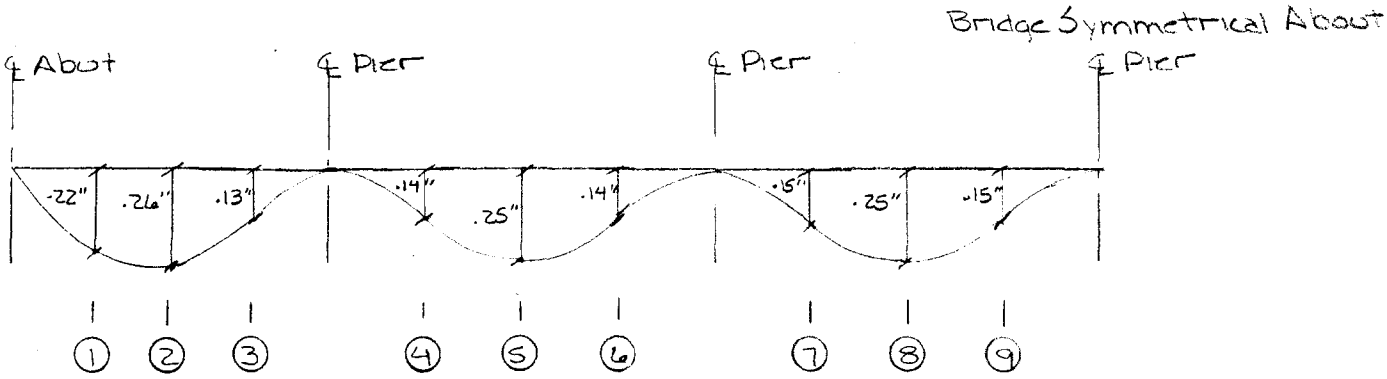
Deck Design

DL Camber (cont.)

Dead Loading



Resulting Initial DL Deflections: (@ Span quarter points)



Long Term Deflections

AASHTO 1.5.23 (G)

Mult. DL Defl. by [2 - 1.2 (As'/As)] for Long Term Deflection

Point	As'	As	Factor [2 - 1.2(As'/As)]	Δ_{DL}(1 + Factor)	Use Camber
1	0.44	2.06	1.74	0.604"	5/8"
2	0.44	2.06	1.74	0.713"	11/16"
3	1.27	2.06	1.26	0.294"	5/16"
4	1.71	2.54	1.19	0.307"	5/16"
5	0.44	2.54	1.79	0.698"	11/16"
6	1.27	2.54	1.40	0.336"	5/16"
7	1.71	2.54	1.19	0.328"	5/16"
8	0.44	2.54	1.79	0.698	11/16"
9	1.27	2.54	1.40	0.360"	5/16"

Deck Design

Live Load Deflection:

Conservatively Calculate Deflection from max LL moment on exterior span applied to the longer interior span

M_{max} LL + I = 28.6 k-ft M_{max} DL+LL+I = 47.4 k-ft

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$I_g = 5832 \text{ in}^4$; $I_{cr} = 2482 \text{ in}^4$ $M_{cr} = 307{,}400$ #-in

$$I_e = \left(\frac{307.4}{47.4(12)}\right)^3 5832 + \left[1 - \left(\frac{307.4}{47.4(12)}\right)^3\right] 2482 = 3011 \text{ in}^4$$

$$\Delta_{LL} = \frac{M\ell^2}{16.5EI} = \frac{28.6(39)^2(12)^3}{16.5(3605)(3011)} = 0.42''$$

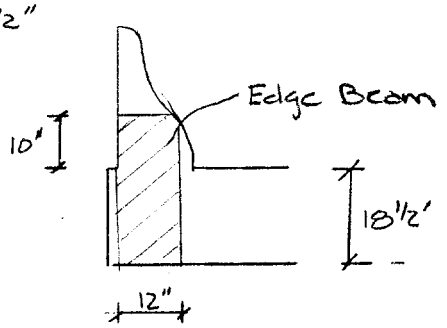
AASHTO (1.7.6) $\ell/1000 = 39(12)/1000 = 0.47'' > 0.42''$

∴ Deflection O.K.

Deck Design

Edge Beam Design

width of beam = 1'-0" depth = 18½" + 10" = 28½"



Dead Load: Slab 0.231 k/ft

Barrier 0.40 k/ft

0.631 k/ft

DL Moments (From Deck Calc)

Exterior Spans (@ 0.4 AB) $M = \frac{0.631}{0.256}(18.8) = 46.3$ k-ft

Supports $M = \frac{0.631}{0.256}(-33.0) = -81.3$ k-ft

Interior Spans $M = \frac{0.631}{0.256}(16.4) = 40.4$ k-ft

Live Load (AASHTO 1.3.2 (D))

Live Load Moment of 0.10 PS P = 16k

Max + Mom. @ 0.40 Exterior Spans:

Table A4.2 'Moments Shears Reactions ....'

Load @ 0.4 AB Coeff = 0.2070

M = 0.2070(16)(32.42) = 107.4 k-ft

0.1(107.4) = 10.7 k-ft

Max - Mom @ Support

Table A4.2

Load @ 0.4 Span 2 Coeff = -0.1038

M = -0.1038(16)(32.42) = -53.8 k-ft

0.1(-53.8 k-ft) = -5.4 k-ft

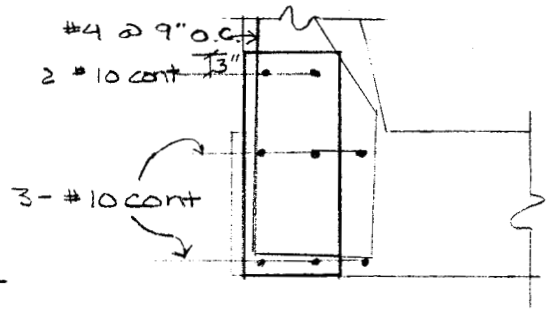
Total Moments:

Max + M = 46.3 + 10.7 = 57.0 k-ft

- M = -81.3 - 5.4 = -86.7 k-ft

Deck Design

Edge Beam (cont)



Design for M = -86.7 k-ft Use 2 #10

$d = 28\frac{1}{2} - 3 = 25\frac{1}{2}''$ Use $d = 25''$ $A_s = 2(1.27) = 2.54 \text{ in}^2$

$n\frac{A_s}{bd} = 8\frac{2.54}{(12)(25)} = 0.0677$ $k = 0.307$ $j = 0.898$

Req'd $d = \sqrt{\frac{86.7(12)}{\frac{1}{2}(1.6)(.307)(.898)(12)}} = 19.8'' < 25''$ ∴ O.K.

$f_s = \frac{86.7(12)}{2.54(.898)(25)} = 18.2 \text{ ksi} < 24.0 \text{ ksi}$

$F_c = \frac{0.307(18.2)}{8(1-0.307)} = 1.0 \text{ ksi} < 1.6 \text{ ksi}$ ∴ O.K.

Pos Moment = 57.0 k-ft < 86.7 k-ft

∴ O.K Use 2 #10

Shear:

$V_{max} = \left(\frac{.631}{.256}\right)(5.2^k) = 12.8^k$

$v = 12.8/(12)(25) = 0.043 \text{ ksi}$

$v_c = 0.95\sqrt{4000} = 60.1 \text{ psi} > 43 \text{ psi} > 30 \text{ psi}$

AASHTO 1.5.10 Req'd $A_v = \frac{50 b_w s}{f_y}$

#4 @ 9" Req'd $A_v = \frac{50(12)(9)}{40000} = 0.135 \text{ in}^2 < 0.20 \text{ in}^2$ ∴ O.K

Pier Design

Pier Cap Design

Bridge Skew = 58° - 31'-20" ℓ = 151.3'

From Deck Calc:

R_{DL} = 10.2 k/ft Due to Skew: w_{DL} = cos 58.52° (10.2) = 5.33 k-ft

Use cap d = 7 1/2"; w = 4'-0"

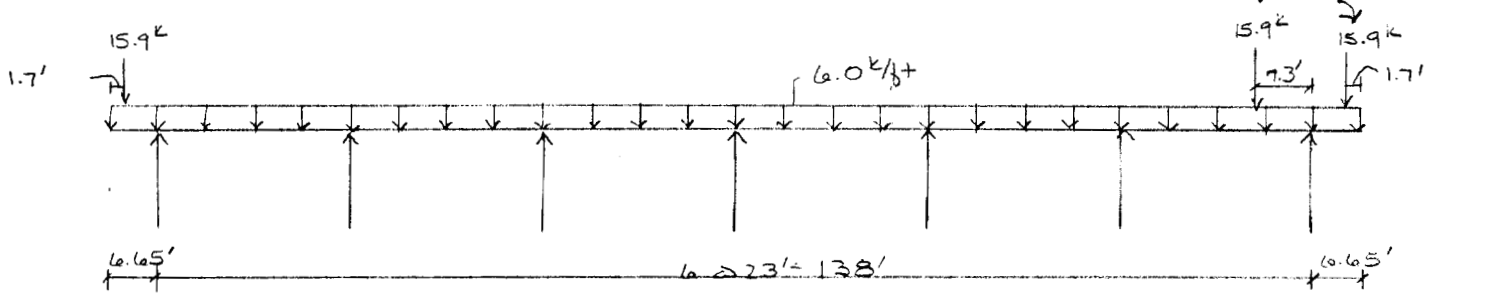
w_D = 5.33 + .150 (4) (7.5/12) = 5.7 k/ft Use w_D = 6.0 k/ft

Load from Barrier - 0.4 k/ft

Concentrated Load = $\frac{0.4}{0.256}$ (10.2) = 15.9 k

Note: Barrier rather than handrail load used to account for fu. lane potential

Dead Load



211.4	264.5	264.5								-289.6	318.6	-211.4
	53.1	26.5								-53.6	-107.2	
	-211.4	291.0								-343.2	211.4	
		3/7	4/7	1/2	1/2	1/2	1/2	1/2	1/2	4/7	3/7	
	-211.4	291.0	-264.5	264.5	-264.5	264.5	-264.5	264.5	-264.5	264.5	-343.2	211.4
		-11.4	-15.1	0	0	0	0	0	0	45.0	33.7	
			0	-7.6		0	0	0	22.5	0		
		0	0	3.8	3.8	0	0	-11.2	-11.2	0	0	
			1.9	0	0	1.9	-5.6	0	0	-5.6		
		-.8	-1.1	0	0	1.9	1.9	0	0	3.2	2.4	
	-211.4	278.8	-279.3	260.7	-260.7	268.3	-268.3	253.3	-253.3	307.1	-307.1	211.4

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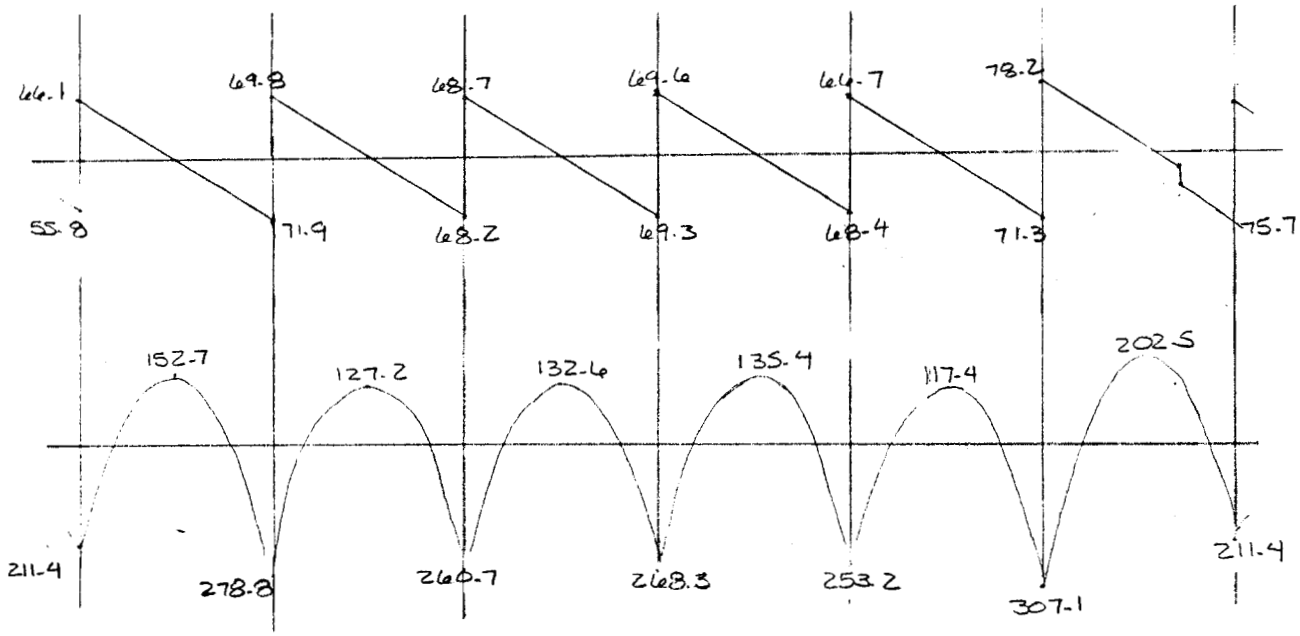
Pier Design

Dead Load Shear & Mom:



Live Load Moments:

Ref.: 'Moments, Shears, & Reactions for Cont Highway Bridges'
Table. 4.2 - See Appendix

Reaction/Lane = 64.5^k (Interpolating for L=2(39+32.42)=142.8'

$w_{LL} = 64.5^k \cos 58.52^\circ / 12' = 2.81\ ^k/ft$

max $\pm M_{LL} \approx 2.81(23)^2/8 = 186\ ^{k\text{-}ft}$

Pier Design

Max Pos Mom D+L+I Use 10 #9

$M = 152.7^{k\text{-}ft} + 1.3(186^{k\text{-}ft}) = 394.5^{k\text{-}ft}$ ($M_{DL} = 202.5$ can't occur w/ $M_{LL} = 186$)

Try 10 #9 $d = 26'' - 1\tfrac{1}{2}'' - .625'' - 1.128/2 = 23.3''$ Use $d = 23''$ $A_s = 10.00\ in^2$

$n\frac{A_s}{bd} = 8.0\frac{10.00}{(48)(23)} = 0.0725$ $k = 0.315$ $j = 0.895$

Req'd $d = \sqrt{\frac{394.5(12)}{\frac{1}{2}(1.6)(.315)(.895)(48)}} = 20.9'' < 23''$ ✓

$f_s = \frac{M}{A_s j d} = \frac{394.5(12)}{(10.00)(.895)(23)} = 23.0\ ksi < 24.0\ ksi$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.315(23.0)}{8(1-.315)} = 1.32\ ksi < 1.60\ ksi$ ∴ O.K

Max Neg Mom D+L+I Use 10 #11

$M = 278.8^{k\text{-}ft} + 1.3(186^{k\text{-}ft}) = 520.6^{k\text{-}ft}$

Try 10 #11 $d = 26'' - 2'' - 1.27'' - .5'' - .625'' - 1.41/2 = 20.9''$ $A_s = 10(1.56) = 15.6\ in^2$

Use $d = 20''$

$n\frac{A_s}{bd} = 8.0\frac{15.6}{(48)(20)} = 0.130$ $k = 0.396$ $j = 0.868$

Req'd $d = \sqrt{\frac{520.6(12)}{\frac{1}{2}(1.6)(.396)(.868)(48)}} = 21.8'' > 20''$

∴ Treat as Doubly Reinforced

$f_s = 24\ ksi$ $f_c = 1.6\ ksi$ $d = 20.$ $d' = 1\tfrac{1}{2}'' + .625'' + 1.128/2 = 2.69''$ Use $d' = 3''$

$k = \frac{n}{n+r}$ $r = f_s/f_c = 24/1.6 = 15$ $k = \frac{8}{8+15} = 0.348$ $j = 0.884$

$M = M_1 + M_2$

Tensile Steel $A_s = A_{s1} + A_{s2}$; Comp. Steel $= A_s'$

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$M_1 = \frac{f_c}{2} k j b d^2 = \frac{1.6}{2}(0.348)(0.884)(48)(20")^2 = 4725^{k\text{-}in} = 394^{k\text{-}ft}$

$A_{s1} = \frac{M_1}{f_s j d} = \frac{394(12)}{24(.884)(20)} = 11.1 \text{ in}^2$

$M_2 = M - M_1 = 520.6 - 394 = 127^{k\text{-}ft}$

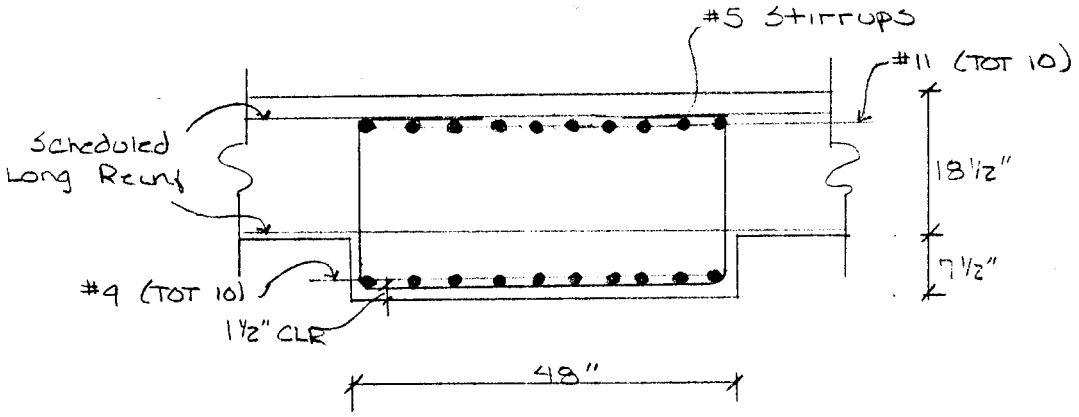
$M_2 = A_{s2} f_s (d - d') = A_s' f_s' (d - d')$

$f_s' = 2 f_s \frac{k - d'/d}{1-k} = 2(24) \frac{.348 - 3/20}{1 - .348} = 14.6 \text{ ksi}$

$A_{s2} = \frac{127(12)}{24(20-3)} = 3.74 \text{ in}^2$ $\quad A_s' = \frac{127(12)}{14.6(20-3)} = 6.14 \text{ in}^2$

Tensile Reinf $\quad A_s = 11.1 + 3.74 = 14.8 \text{ in}^2$ $\quad$ Use 10 #11 $\quad A_s = 15.6 \text{ in}^2$

Comp Reinf $\quad A_s' = 6.14 \text{ in}^2$ $\quad$ 10 #9 $\quad A_s = 10.00 \text{ in}^2$



Pier Cap Section

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Pier Design

Shear Reinforcement

@ dist. d (20") from Interior Support:

V_{max} DL = $71.9^k - 6.0(20/12) = 61.9^k$

LL: V_{max} LL = $(2.81^k/ft)(23')/2 - 2.81(20/12) = 27.6^k$

$V_{D+L+I} = 61.9^k + 1.3(27.6^k) = 97.8^k$

$v = \frac{V}{bd} = \frac{97.8^k}{(48")(20")} = 0.102$ ksi $\qquad v_c = 0.95\sqrt{4000} = 60.1$ psi

#5 stirrup $A_v = 2(0.31) = 0.62$ in^2

Req'd $s = \frac{0.62(20{,}000)}{(102-60.1)(48)} = 6.2"$ $\qquad$ Use $s = 6"$

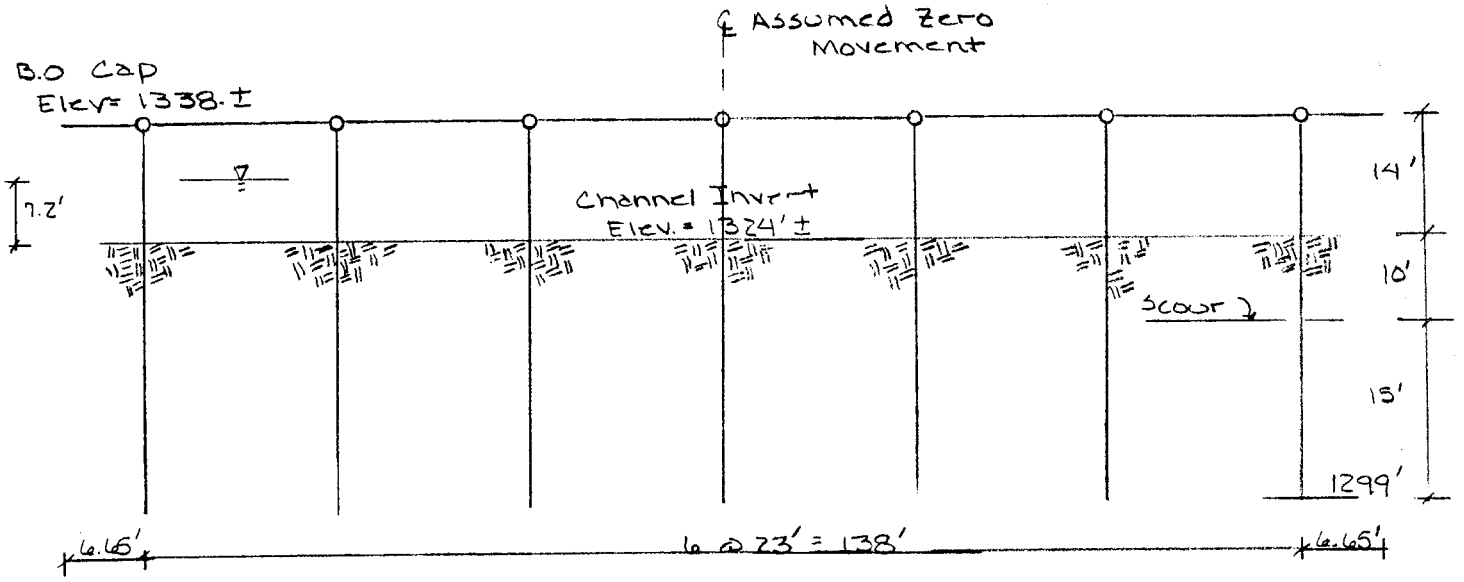
Use #5 stirrups @ 6" o.c.

Pier Design

Pier Column

- Embedment depth of piles checked using Brom's Method for the cohesionless and cohesive soil cases. - See Appendix
- Columns Designed using NAVDOCKS DM-7 - See Appendix

Transverse Model



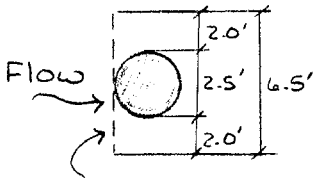
Stream Flow Forces

(AASHTO 1.2.17) $P = KV^2$

$V = 7.0$ ft/s $K = 1.375$

$P = 1.375(7.0)^2 = 67.4$ psf

Depth of Flow = 7.2'

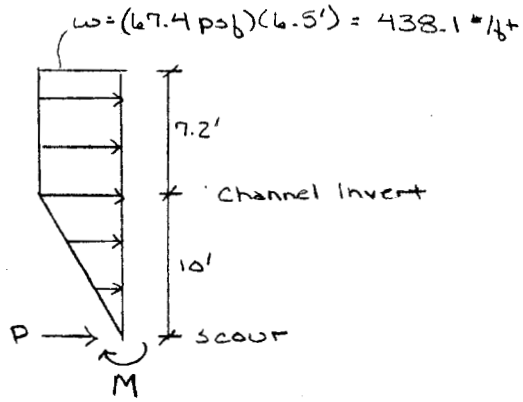


Per Corps of Engrs. Recomm.

Debris Laden Surface Assumed

Pier Design

Stream Flow Forces (cont.)



$P = 438.1(7.2) + 438.1(10)/2 = 5.34^k$

$M = 438.1(7.2)(10 + 7.2/2) + 438.1(10/2)(2(10)/3) = 57.5^{k\text{-}ft}$

Temperature Forces - Based on Uncracked Section

Transverse Direction:

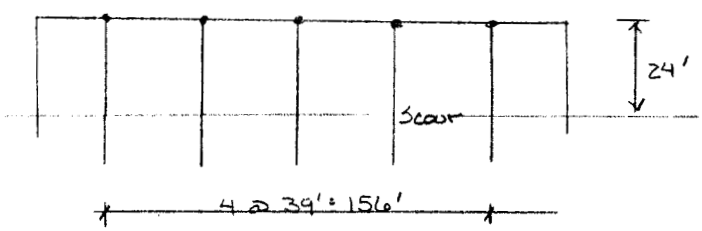
$\Delta_{\substack{outside\\pier}} = (0.0000006)(69')(12)(40°F) = 0.20''$

$P = \frac{3EI\Delta}{L^3}$ $E = 3122$ ksi $I = \pi(15)^4/4 = 39.761\ in^4$ $L = 24'$

$P = \frac{3(3122)(39.761)(.20)}{(24 \times 12)^3} = 3.12^k$

$M_{@\ scour} = 3.12^k(24') = 75^{k\text{-}ft}$

Longitudinal Direction:



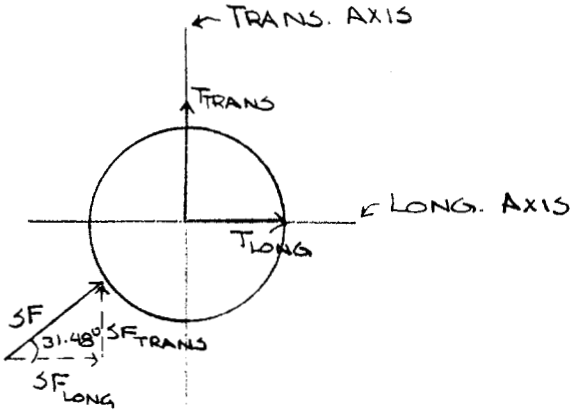
$\Delta_{\substack{outside\\pier}} = (0.0000006)(78')(12)(40°F)$
$= 0.22''$

$P = \frac{3(3122)(39.761)(0.22)}{(24 \times 12)^3} = 3.43^k$

$M_{@\ scour} = (3.43^k)(24') = 82^{k\text{-}ft}$

Pier Design

Load Case IV - Stream Flow and Temperature



$T_{TRANS} = 3.12^k$ @ 24'

$T_{LONG} = 3.43^k$ @ 24'

$SF = 5.34^k$ $\quad SF_{LONG} = 5.34 \cos 31.48° = 4.55^k$ @ 10.8'

$SF_{TRANS} = 5.34 \sin 31.48° = 2.79^k$ @ 10.8'

$P_T = 3.12^k + 2.79^k = 5.91^k$ @ $\dfrac{3.12(24) + 2.79(10.8)}{5.91} = 17.8'$

$P_L = 3.43^k + 4.55^k = 7.98^k$ @ $\dfrac{3.43(24) + 4.55(10.8)}{7.98} = 16.5'$

$P = (5.91^2 + 7.98^2)^{1/2} = 9.93^k$ $\quad$ <u>use $P = 10^k$ @ 17'</u>

$M_{@\ scour} = 10^k(17') = 170^{k\text{-}ft}$

Pier Design

Earthquake Forces (AASHTO 1.2.20) - Based on Uncracked Section

EQ = CFW

F = 1.0

W = Dead Load =	Wearing surface	= (.025 ksf)(76')(221')	= 420^k
	Barriers	= (.4 k/ft)(221')(2)	= 177^k
	Slab	= (.150 k/ft^3)(18.5/12)(221)(79')	= 4037^k
	Haunches	= (.150 k/ft^3)(7.5/12)(4')(151.3')(5)	= 284^k
	Columns	= (.150 k/ft^3)(14')(4.91 ft^2)(35)	= 361^k
	Pedestrian Curb & Handrail	= (0.2 k/ft)(221')	= 44^k
	Abutment Diaph	= (.150 k/ft^3)(2.5')(3')(165.3')(2)	= 372^k
			5695^k

- Assume piers resist total seismic force

$P = P_1 + P_2$ $\quad$ P_1 - force to cause 1" movement of piers

P_2 - force to cause 1" movement of abutment

$$P_1 = \frac{3EI\Delta}{L^3} \times 35 \text{ piles} \; ; \; \Delta = 1''$$

$$P_1 = \frac{3(3122)(39,761)(1)}{(14 \times 12)^3} \times 35 = 2749^k$$

P_2 - Calc using NAVDOCKS DM-7 - See Appendix

$f = .03$ ksi $\quad EI = 3122(\pi)(12)^4/4) = 5.08 \times 10^7$ k-in^2

$$T = \left(\frac{EI}{f}\right)^{1/5} = (5.08 \times 10^7/.03)^{1/5} = 70.1''$$

$L = (1340 - 10) - 1299 = 31'$ $\quad L/T = 31(12)/70.1 = 5.3$

$$P_2 = \frac{\delta E I}{F_\delta T^3} \times 13 \text{ shafts/abut} \times 2 \text{ abut} \qquad F_\delta = 2.3 \quad \delta = 1''$$

$$P_2 = \frac{1(5.08 \times 10^7)}{2.3(70.1)^3} \times 26 = 1667^k$$

$P = 2749^k + 1667^k = 4416^k$

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$T = 0.32\sqrt{W/P} = 0.32\sqrt{5695/4416} = 0.363$

From AASHTO Fig. 1.2.20 D (Zone II A=0.22g) C= 0.085

$EQ = 0.085(1.0)(5695) = 484^k$

- Assume earthquake force is evenly distributed between all 35 columns

$EQ/col = 484/35 = 13.8^k$ use 14^k

$M_{@\ invert} = 14^k(14') = 196^{k-ft}$

Pier Design

Design of Columns Using DM-7 & Assuming Uncracked Section

Case IV - Stream Flow and Temperature $1.0(D+L+S+T) \leq 1.25$

$f = 60\ \#/in^3$

EI (for gross sect.) $= (3122\ ksi)(\pi(15'')^4/4) = 1.24 \times 10^8\ k\text{-}in^2$

$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{1.24 \times 10^8}{.06}\right)^{1/5} = 73''$

$L = 15'$ $\quad L/T = 15(12)/73 = 2.5$

$P = 10^k \quad M = 170^{k\text{-}ft}$

Max. Moment (1982 DM-7, Figure 11)

@ $z/T = 0$ $\quad M_M = 1.0(170) = 170^{k\text{-}ft}$ - controls

$M_P = 0.0$

@ $z/T = 1.0$ $\quad M_M = 0.75(170) = 128^{k\text{-}ft}$

$M_P = 0.60(10^k)(73/12) = \underline{36^{k\text{-}ft}}$

$164^{k\text{-}ft}$

$M_{CASE\ IV} = 170/1.25 = 136^{k\text{-}ft}$

ext. col. $P_D = 122^k$

$P_L = (2.8^k/ft)(23'/2 + 6.65') = 51^k$

$P_{CASE\ IV} = \frac{122^k + 51^k}{1.25} = 138^k$

Pier Design

Case VII - Earthquake - 1.0 (D + SF + EQ) ≤ 1.33

Assume Earthquake and Stream Flow don't occur simultaneously - Earthquake controls

$f = 30\#/in^2$ $\quad EI_g = 1.24 \times 10^8$ k-in² $\quad T = \left(\frac{1.24 \times 10^8}{.03}\right)^{1/5} = 84''$ $\quad L = 25'$

$L/T = 25(12)/84 = 3.6$

Max Moment

@ $z/T = 0$ $\quad M_m = 1.0(196^{k\text{-}ft}) = 196^{k\text{-}ft}$

$M_p = 0.0$

@ $z/T = 1.2$ $\quad M_m = 0.76(196^{k\text{-}ft}) = 149^{k\text{-}ft}$

$M_p = 0.73(14^k)(84/12) = 72^{k\text{-}ft}$

$\Big\}$ $221^{k\text{-}ft}$ - Controls

$M_{case\ VII} = 221/1.33 = 166^{k\text{-}ft}$

$P_{case\ VII} = 122/1.33 = 92^k$

Check If Section Cracks

AASHTO 1.5.26 $\quad f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{3000} = 411$ psi

Gross Section $S = \pi(15'')^3/4 = 2651$ in⁴

Case IV: $M = 136^{k\text{-}ft}$ $\quad f_b = 136(12)/2651 = 0.616$ ksi > 0.411 ksi

∴ Section Cracks, Recalc. M using Stiffness Representative of cracked section.

Case VII: $M = 166^{k\text{-}ft}$ $\quad f_b = 166(12)/2651 = 0.751$ ksi > 0.411 ksi

∴ Section Cracks, Recalc M using Representative Stiffness

Pier Design

Stiffness Representative of Cracked Section

Ref: AASHTO 1.5.34

Conservatively use $EI = \left(\frac{E_c I_g}{2.5}\right) / (1+\beta_d)$ $(\beta_d = 0)$

$$EI = (3122 \text{ ksi})(\pi (15'')^4/4) / 2.5 = 49,653,000 \text{ k-in}^2$$

Ref: Heins & Lawrie, 'Design of Modern Conc Hwy. Bridges,' Wiley 1984

$EI = E_c I_g (0.2 + 1.2 \rho_t \frac{E_s}{E_c})$ where $\rho_t = \frac{A_s}{A_c}$

For 8 #11 $\rho_t = \frac{8(1.56)}{\pi (15)^2} = 0.0176$

$$EI = 3122 \pi (15)^4/4 \left(0.2 + 1.2(0.0176) \frac{29000}{3122}\right) = 49,179,000 \text{ k-in}^2$$

$49,653,000 \text{ k-in}^2 > 49,179,000 \text{ k-in}^2$

$\therefore$ Use $EI = 49,653,000 \text{ k-in}^2$

Stream Flow & Temp - Revised M for Cracked Section

Stream Flow @ Scour $P = 5.34^k$ $(SF_{TRANS} = 2.79^k;\ SF_{LONG} = 4.55^k)$

$M = 57.5^{k\text{-}ft}$

Temperature

Transv. Direction $\Delta = 0.20''$ $P = \frac{3(49,653,000)(0.20)}{(24 \times 12)^3} = 1.25^k$

$M_{@\,scour} = 1.25^k (24') = 30.0^{k\text{-}ft}$

Long. Direction $\Delta = 0.22''$ $P = \frac{3(49,653,000)(0.22)}{(24 \times 12)^3} = 1.37^k$

$M_{@\,scour} = 1.37^k (24') = 32.9^{k\text{-}ft}$

Pier Design

SF & Temp cont)

$P_{TRANS} = 2.79^k + 1.25^k = 4.04^k$ @ $\frac{2.79(10.8') + 1.25(24')}{4.04} = 14.9'$

$P_{LONG} = 4.55^k + 1.37^k = 5.92^k$ @ $\frac{4.55(10.8) + 1.37(24)}{5.92} = 13.8'$

$P = (4.04^2 + 5.92^2)^{1/2} = 7.17^k$ Use $P = 7.2^k$ @ 14.3'

$M_{@\ scour} = 7.2^k(14.3') = 103^{k\text{-}ft}$

DM-7 $T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{49,653,000}{.06}\right)^{1/5} = 60.8''$ $L/T = 15(12)/60.8 = 3.0$

Max Mom:

@ $z/T = 0$ $M_m = 1.0(103) = 103^{k\text{-}ft}$

$M_p = 0.0$

@ $z/T = 1.2$ $M_m = 0.77(103) = 79^{k\text{-}ft}$

$M_p = 0.70(7.2)(60.8/12) = 26^{k\text{-}ft}$

$\Big\}$ $105^{k\text{-}ft}$ ← controls

$M_{CASE\ IV} = 105^{k\text{-}ft}/1.25 = 84^{k\text{-}ft}$

Earthquake - Revised M for Cracked Section

$EQ = CFW$ $F = 1.0$; $W = 5695^k$

$P = P_1 + P_2$ $P_1 = \frac{3(49,653,000)(1)}{(14 \times 12)^3} \times \overset{\#\ of\ shafts}{35} = 1100^k$

Calc P_2 using DM-7 $EI_{abut} = 20,338,000\ k\text{-}in^2$ (See Abut. Shaft Calc)

$T = (20338000/.03)^{1/5} = 58.4''$ $L/T = 31(12)/58.4 = 6.4$

$P_2 = \frac{1(20338000)}{2.3(58.4)^3} \times 26 = 1154^k$

$P = 1100 + 1154 = 2254^k$ $T = 0.32\sqrt{5695/2254} = 0.51 \Rightarrow C = 0.09$ (AASHTO Fig 1.2.20D)

$EQ = 0.09(1.0)(5695) = 512^k$ $EQ/shaft = 512/35 = 15^k$ $M_{@\ Invert} = 15^k(14') = 210^{k\text{-}ft}$

Pier Design

EQ (cont)

DM-7 $T = \left(\frac{49\,653\,000}{.03}\right)^{1/5} = 69.8''$ $L/T = 25(12)/69.8 = 4.3$

Max Moment:

@ $z/T = 0$ $M_m = 1.0(210) = 210^{k\text{-}ft}$

$M_p = 0.0$

@ $z/T = 1.25$ $M_m = 0.75(210) = 158^{k\text{-}ft}$

$M_p = 0.77(15^k)(69.8/12) = 67^{k\text{-}ft}$ } $225^{k\text{-}ft}$ – controls

$M_{case\ VII} = 225/1.33 = 169^{k\text{-}ft}$

$P_{case\ VII} = 92^k$

∴ Case VII Controls Design

$l_u = 14'$; $K = 2.1$; $r = 7.5''$ $Kl_u/r = 2.1(14)(12)/7.5 = 47 < 100$

$P_c = \frac{\pi^2(49\,653\,000)}{((2.1)(14)(12))^2} = 3937^k$

$\delta = \frac{1.0}{1 - \frac{2.5(92)}{3937}} = 1.06$

$M_c = 1.06(169) = 179^{k\text{-}ft}$

Design Loads $M_u = 179^{k\text{-}ft}/0.35 = 511^{k\text{-}ft}$

$P_u = 92^k/0.35 = 263^k$

Use 8 #11 Verts – O.K See Circon Output

AASHTO 1.5.11 (B)

Use 1/2" Ø Spiral ties @ 6" pitch

```
^^^^^^^^^^^^^^^^
CIRCON
vvvvvvvvvvvvvvvv

                "P"

FY=60,000.00
Fc=3,000.00
D=30.00
D1=4.21
AS1=1.56
N=8.00
P=263.00
MX=511.00
MY=1.00E-8

REBAR COORDINATES
Rb=10.79 ∠=0.00
Rb=10.79 ∠=45.00
Rb=10.79 ∠=90.00
Rb=10.79 ∠=135.00
Rb=10.79 ∠=180.00
Rb=10.79 ∠=225.00
Rb=10.79 ∠=270.00
Rb=10.79 ∠=315.00

<.8>PMAX=1,511.68

ebR=12.78
MbR=7,175.94
PbR=561.63
cbR=15.26
B∠=1.12E-9

P=262.69
MX=554.38 > 511
MY=1.08E-8
cR=11.79
ecR=2.11
```

Pier Design

Embedment Depth per Brom's Method

See Appendix

Per Soil Report -

1) Cohesive case: $K_hD = 400\#/in^2$ ($K_hD = 200\#/in^2$ for Earthquake)

2) Cohesionless case: $n_h = 60\#/in^3$ ($n_h = 30\#/in^3$ for Earthquake)

Concrete Pile: $D = 30''$; $I = 39,761\ in^4$; $E = 3122$ ksi

Soil: $\phi = 30°$; $C_u = 900$ psf ; $K_p = 3.0$; $\gamma = 120\#/ft^3$

$L = 15'$ Stream Flow & Temp (Case IV)

$L = 25'$ Earthquake (Case VII)

Cohesive Case:

IV: $\beta = \sqrt[4]{\frac{K_hD}{4EI}} = \sqrt[4]{\frac{400\#/in^2}{4(3122000\ psi)(39761\ in^4)}} = 0.0053/in$

$\beta L = 0.0053(15')(12) = 0.95 < 2.25$ ∴ Rigid Pile Action

Stream Flow & Temp: ($L = 15'$)

$P = 10^k$ @ $17'$ $e/D = 17'/2.5 = 6.8$ $\frac{P}{C_uD^2} = \frac{10,000}{900(2.5)^2} = 1.78$

→ Req'd $L/D = 4.5 \pm$ $L = 4.5(2.5) = 11.2 < 15$ $FS = \frac{15}{11.2} \times 1.25 = 1.67$

VII: $\beta = \sqrt[4]{\frac{200\#/in^2}{4(3122000)(39761)}} = 0.0045/in$

$\beta L = 0.0045(25)(12) = 1.35 < 2.25$ ∴ Rigid Pile Action

Earthquake -

$P = 13^k$ $e = 14'$

$e/D = 14/2.5 = 5.6$ $\frac{P}{C_uD^2} = \frac{13000}{900(2.5)^2} = 2.31$ Req'd $L/D = 4.5 \pm$

$FS = \frac{25}{11.2}(1.33) = 3.0$

Pier Design

Broms Method (cont)

Cohesionless Case

SF & T : $\eta = \sqrt[5]{\frac{n_h}{EI}} = \sqrt[5]{\frac{160 \text{ \#/in}^3}{(3122000)(39761)}} = 0.014$

$\eta L = 0.014(15)(12) = 2.52 > 2.0$ ∴ Flexible Pile Action

EQ: $\eta = \sqrt[5]{\frac{30}{(3122000)(39761)}} = 0.012$

$\eta L = 0.012(25)(12) = 3.6 > 2.0$ ∴ Flexible Pile Action

∴ Embedment Length no longer a factor, Flexible Pile action - for which shafts are designed - takes over

∴ Embedment Depth of 15' below scour O.K. for Lateral Load

Pier Design

Vert. Capacity

Axial Load = $150^k + 69^k = 219^k$

Embedment of 15' below scour req'd per Brom's Analysis

Ref: Figure 4, Sergent, Hauskins & Beckwith Soil Report - See Appendix

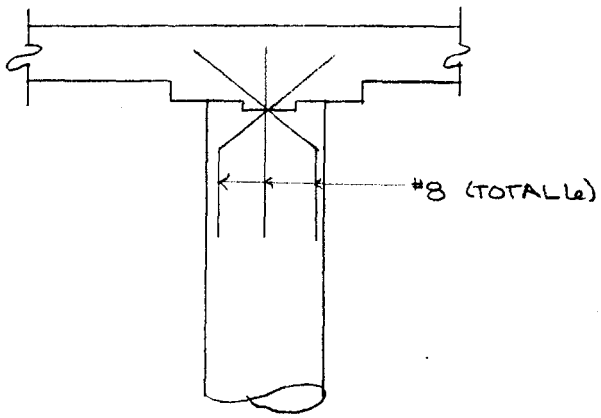
Bell Diam: 4.5'

Allowable Bearing capacity @ 1299' = $275^k > 219^k$

Use 30" Ø columns w/ 8 #11 Verts & 1/2" Ø spiral ties @ 6" pitch w/ 4.5' Ø Bell & Pier Tip Elev. = 1299'

Pier Design

Pier Column Pin Design



Lateral Load:

Temperature $P = (1.25^2 + 1.37^2)^{1/2} = 1.85^k$

Earthquake $P = 15^k$ ← Controls.

Design for $V = 15^k / 1.33 = 11.3^k$

AASHTO (1.5.29 (D)) Req'd $A_{vf} = V / f_s \mu$ $\mu = 1.4$; $f_s = 24$ ksi

$A_{vf} = 11.3^k / (24\text{ ksi})(1.4) = 0.34 \text{ in}^2$

6 - #8 $A_s = 6(0.79) = 4.74 \text{ in}^2 > 0.34 \text{ in}^2$

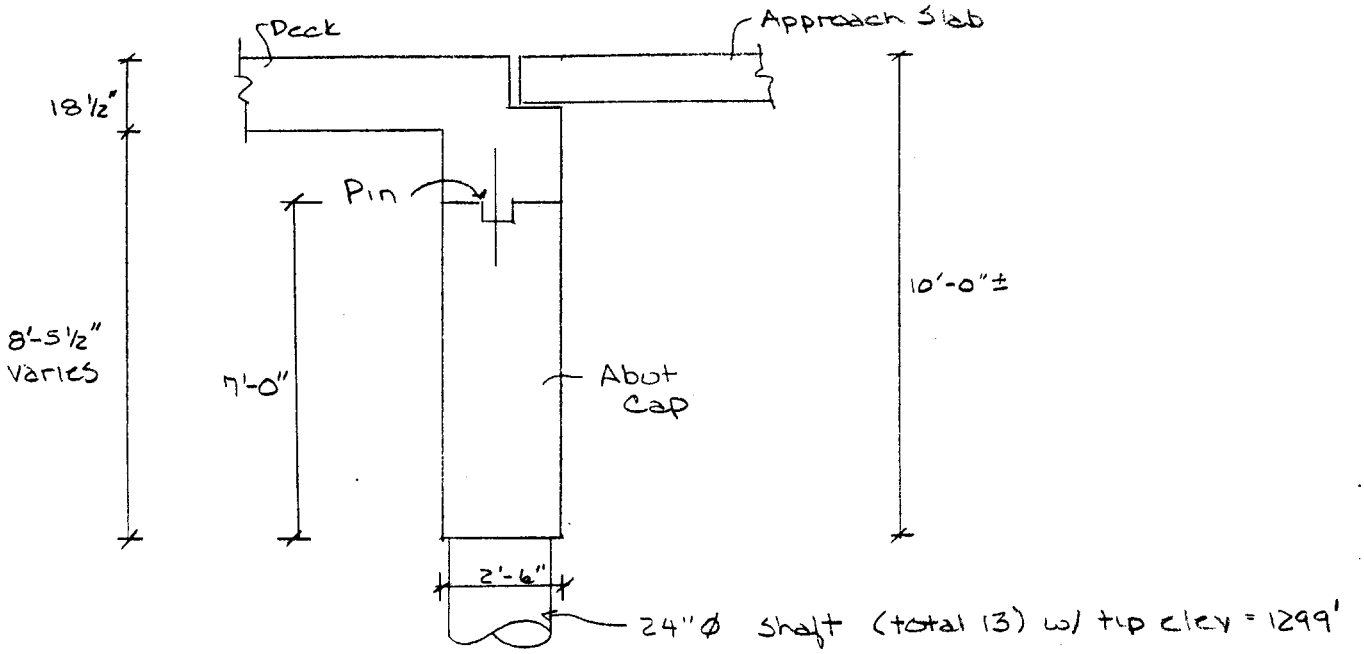
∴ O.K. Use 6 #8 in Shear Key

Abutment Design

Note: The abutment cap depth was increased from 7' to 9' for Abutment 1. See supplemental calcs at the end of this section

Abutment Design

Abutment Cap



Dead Load

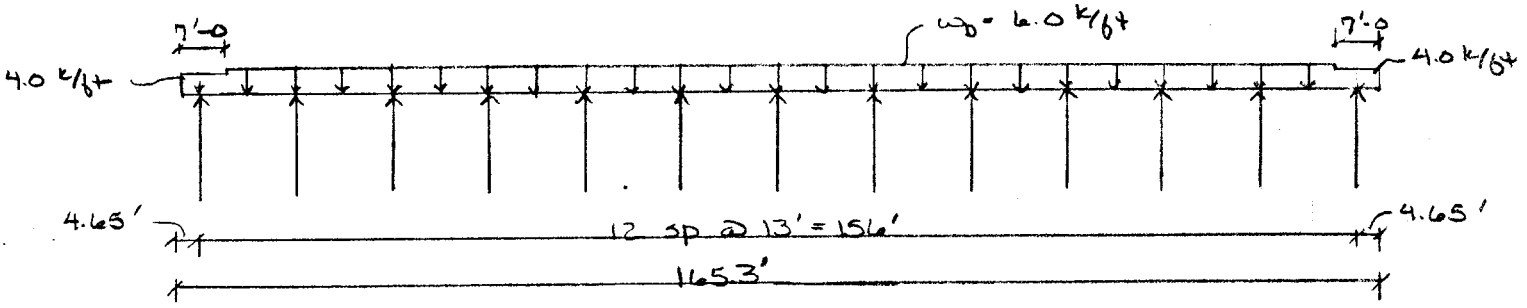
From Deck Calc: $R = 3.1\ ^k/_{ft}$ Barrier Load $= \left(\frac{3.1\ ^k/_1}{.256\ ^k/_{sf}}\right)\left(\frac{2(0.4)+0.2}{151.29}\right) = 0.08\ ^k/_{ft}$

$$w_D = \cos 58.52^\circ (3.1\ ^k/_{ft}) + 0.08\ ^k/_{ft} = 1.70\ ^k/_{ft}$$

Weight of Abutment: $(0.150\ ^k/_{ft^3})(2.5')(10') = 3.75\ ^k/_{ft}$ use $4.0\ ^k/_{ft}$

Total uniform dead load: $w_D = 1.70 + 3.75 = 5.45\ ^k/_{ft}$

Use $w_D = 6.0\ ^k/_{ft}$



Abutment Design

Live Load

$76'/12' = 6.33 \Rightarrow$ use 6 traffic Lanes

Ref.: Moments, Shears & Reactions for Continuous High Br.
Table 4.2 – See Appendix

$R_{LL} \approx 48^k/\text{Lane}$

$w_L \approx 6(48^k/\text{Lane})/151.29' = 1.90\ k/ft$ use $w_L = 2.0\ k/ft$

Interior caisson:

$P_{DL} \approx 13'(6\ k/ft) = 78^k$
$P_{LL} \approx 13'(2\ k/ft) = 26^k$
104^k

Exterior caisson

$P_{DL} \approx (13'/2 + 4.65')(4\ k/ft) + (4')(2\ k/ft) = 53^k$
$P_{LL} = (4')(2\ k/ft) = 8^k$
61^k

D+L:

$\text{Max} \pm M \approx 0.1\, w\ell^2 = 0.1(8\ k/ft)(13')^2 = 135\ k\text{-}ft$

$d = 7'\text{-}0'' - 6'' - 2(0.75'') - 0.875''/2 = 76.0''$ use $d = 75''$

Try 3 #7 T&B $A_s = 3(0.60) = 1.80\ in^2$ $n = 29{,}000/57\sqrt{3000} = 9.3$
use $n = 9$

$n\frac{A_s}{bd} = 9\frac{1.80}{(30)(75)} = 0.0072$ $k = 0.113$ $j = 0.962$

$f_s = \frac{M}{A_s j d} = \frac{135(12)}{(1.80)(.962)(75)} = 12.5$ ksi

$F_a = \frac{k f_s}{n(1-k)} = \frac{.113(12.5)}{9(1-.113)} = 0.18$ ksi $\therefore$ o.k. use 3 #7 T&B

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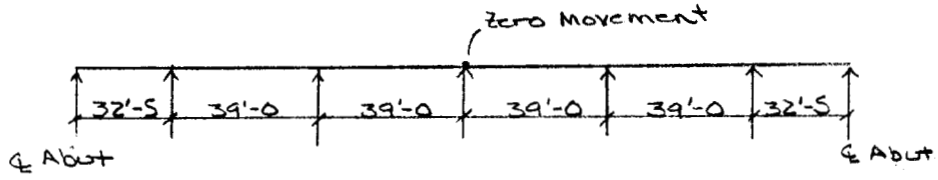
designed by RP
checked by RT 10-8-84 ME 1-21-85
date 10/2/84 sh. 40 of
12/18/84

Abutment Design
Lateral Load Reinf.

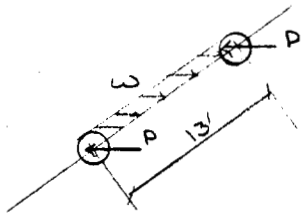
- Assume abutment cap moves as a rigid body with slab

Cap Movement due to temp:



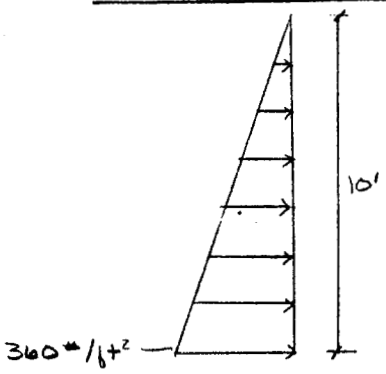
$\delta = (.000006)(40°)(110.4')(12) = 0.32''$

P_δ /caisson = 22^k – See Caisson Design Calcs



$w = 22^k/13' = 1.7\ ^k/LF$

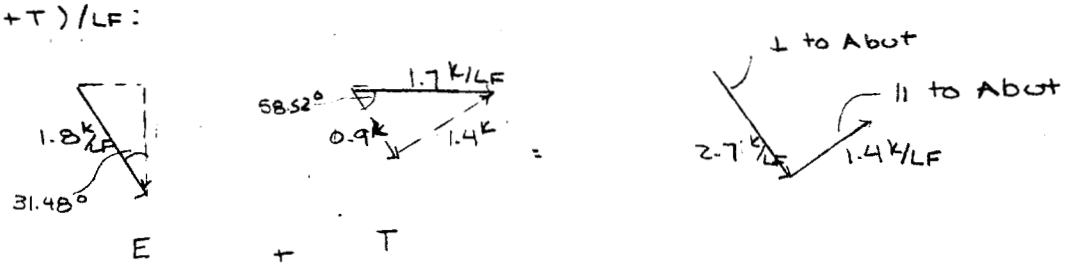
Earth Pressure



Active Earth Pressure = 36 pcf

Long Force = 1/2 (360)(10) = 1.8^k/L.F.

(E + T)/LF:



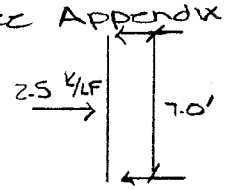
Abutment Design

E+T Long. Force = 2.7 K/LF

Design Load: 2.7 / 1.25 = 2.2 K/LF (AASHTO 1.2.22 - Case IV)

use 2.5 K/LF Ref: 'Bridge Planning & Design Manual, Vol IV', pg 6-22
See Appendix

use #5 @ 12" $M \approx 2.5(7')/4$
$= 4.4$ K-ft

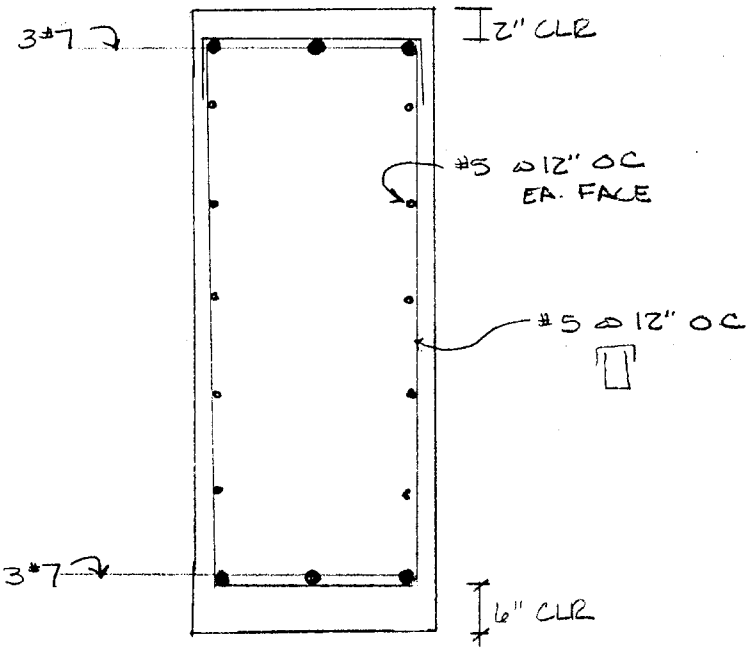


$d = 30'' - 3'' - .625/2 = 26.7''$ use $d = 26''$ $A_s = 0.31$ in^2

$n \frac{A_s}{bd} = 9 \frac{0.31}{(12)(26)} = 0.0089$ $k = 0.125$ $j = 0.958$

$f_s = \frac{4.4(12)}{(0.31)(.961)(26)} = 6.81$ ksi ∴ O.K use #5 @ 12"

$V_{max} = 2.5/(12)(26) = 0.008$ ksi $< \frac{1}{2}(0.95\sqrt{3000}) = 26$ psi



Abutment Cap
Section

Abutment Design

AASHTO 1.5.7 (D)

$\rho_{min} = 0.002$

$A_g = (12')(26'') = 312 \text{ in}^2$ $\quad A_s = 2(0.31) = 0.62 \text{ in}^2$

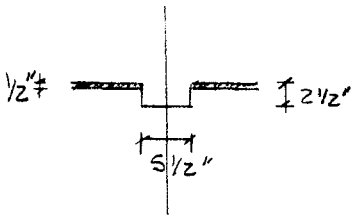
$\rho = \frac{0.62}{312} = 0.002 = 0.002$ $\quad$ O.K.

AASHTO 1.4.7 (B) Temp. Reinf. Req'd = 1/8 in²/ft ht.

Use #5 bars @ 12" $\quad A_s = 0.31 \text{ in}^2/\text{ft} > 0.125 \text{ in}^2/\text{ft}$

Shear Key (pin)

(AASHTO 1.5.29 (D))



Req'd $A_{vf} = V / f_s \mu$

$\mu = 1.4 \quad f_s = 20 \text{ ksi} \times 1.25$

$V = .672 \text{ k/LF} + 0.9 \text{ k/LF} = 1.62 \text{ k/LF}$

Req'd $A_{vf} = 1.62 \text{ k/LF} / (20 \text{ ksi})(1.25)(1.4) = 0.046 \text{ in}^2/\text{LF}$

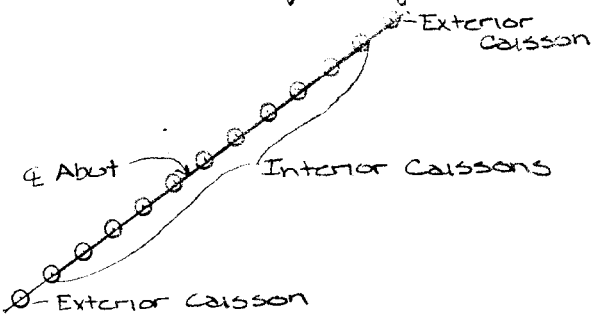
Use #4 @ 12" $\quad A_{vf} = 0.20 \text{ in}^2/\text{LF}$

Abutment Design

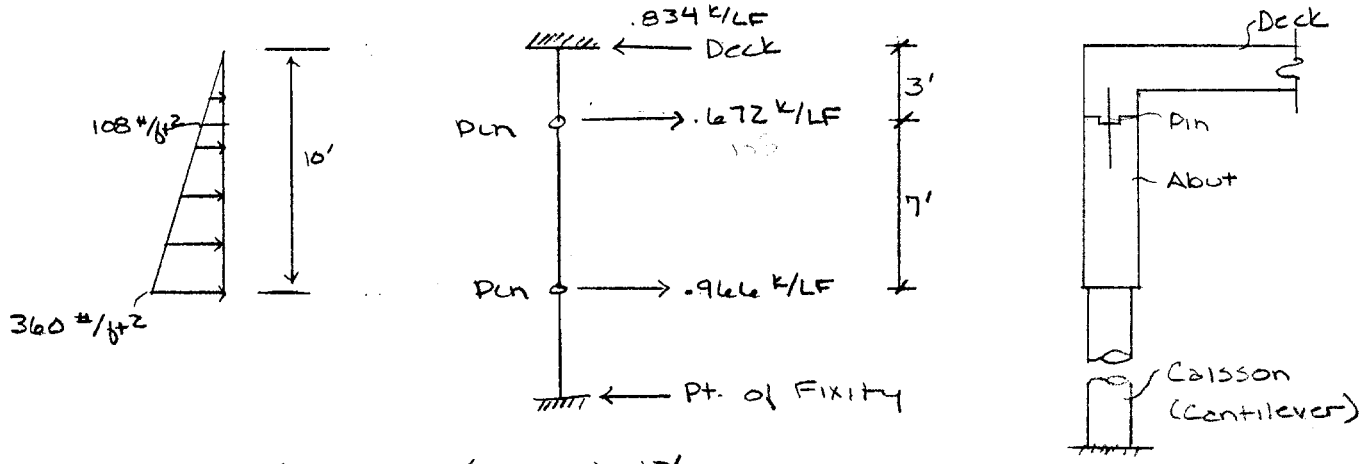
Caisson Design

Assumptions:

- Abutment cap is pinned to caissons - no moment transfer
- No earthquake forces on caissons - earthquake forces taken entirely by piers.
- Assume future bank protection - No stream flow force



Lateral Load due to Earth Pressure



Interior Caissons Support 13'

Lateral Load / int. shaft = (.966 k/LF)(13') = 12.6 k

Exterior Caissons Support 13/2 + 4.65' = 11.15'

Lateral Load / ext. shaft = (.966 k/LF)(11.15') = 10.8 k

Abutment Design

Ref: NAVDOCKS DM-7:

$f = 0.06\ ^k/in^3$ 24" Ø Caisson $I = \pi(12)^4/4 = 16{,}286\ in^4$

$E = 3122$ ksi $EI = 5.08 \times 10^7$ k-in²

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{3122(16286)}{.06}\right)^{1/5} = 161.1''$$

L: 1) 1340.74 - 10 - 1299 = 32'

2) 1339.02 - 10 - 1299 = 30' use L = 31'

$$L/T = 31(12)/161.1 = 6.1$$

Temperature Forces:

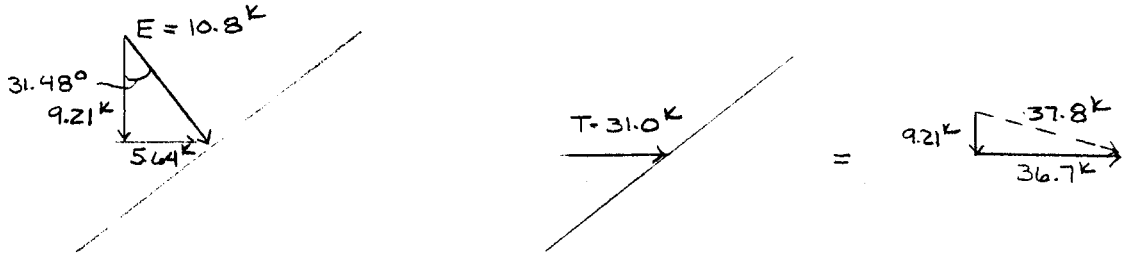
$$\delta = 0.32'' \quad F_\delta = 2.3 \quad P_\delta = \frac{\delta EI}{F_\delta T^3} = \frac{0.32(5.08\times10^7)}{2.3\,(161.1)^3} = 31.0^k/\text{shaft}$$

Axial Loads: Exterior : $P_{D+L} = 161^k$

Interior : $P_{D+L} = 104^k$

∴ Load Case IV on exterior shafts controls design

E+T:



E+T - Lateral Load = 37.8^k

M_{max} @ $z/T = 1.3$ $M = 0.77(37.8^k)(161.1/12) = 148$ k-ft

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Abutment Design

Check If Section Cracks

$F_r = 7.5\sqrt{3000} = 411$ psi

Case IV: $M = 148^{k\text{-}ft}/1.25 = 118^{k\text{-}ft}$ $\quad S_g = \pi(12'')^3/4 = 1357\ in^3$

$f_b = M/S = 118(12)/1357 = 1.04$ ksi > 0.411 ksi

∴ Section cracks - Recalculate M using Stiffness Representative of Cracked Section.

Stiffness Representative of Cracked Section

Ref: AASHTO 1.5.34

Use $EI = \left(\frac{E_c I_g}{2.5}\right)/(1+\beta_d)$ $\quad \beta_d = 0$

$$EI = \frac{3122\pi(12)^4/4}{2.5} = 20{,}338{,}000\ k\text{-}in^2$$

Ref: Heins & Lawrie, 'Design of Modern Conc Hwy Bridges', Wiley 1984

$EI = E_c I_g (0.2 + 1.2\rho_t \frac{E_s}{E_c})$ For 8 #7 $\rho_t = \frac{8(0.60)}{\pi(12)^2} = 0.0106$

$$EI = 3122\pi(12)^4/4\left(0.2 + 1.2(0.0106)\frac{29000}{3122}\right) = 16{,}176{,}000\ k\text{-}in^2$$

$20{,}338{,}000\ k\text{-}in^2 > 16{,}176{,}000\ k\text{-}in^2$

∴ Use $EI = 20{,}338{,}000\ k\text{-}in^2$

DM-7: $T = \left(\frac{20{,}338{,}000}{.06}\right)^{1/5} = 50.8''$ $\quad L/T = 31(12)/50.8 = 7.3$

Temp Forces $\delta = 0.32''$; $F_\delta = 2.3$ $\quad P_\delta = \frac{0.32(20\,338\,000)}{2.3(50.8)^3} = 21.6$ k/shaft

E+T - Lateral Load $= ((9.21)^2 + (5.64 + 21.6)^2)^{1/2} = 28.8^k$

M_{max} @ $z/T = 1.3$ $\quad M = 0.77(28.8^k)(50.8/12) = 94^{k\text{-}ft}$

Abutment Design

CASE IV : $M = 94/1.25 = 75^{k\text{-}ft}$

$P = 61/1.25 = 49^{k}$

- Slenderness Need Not be Considered - Caisson fully embedded

Design Loads: $M_u = 75/0.35 = 214^{k\text{-}ft}$

$P_u = 49/0.35 = 140^{k}$

Use 8 #7 - See Circon Output

```
^^^^^^^^^^^^^^^^
CIRCON
vvvvvvvvvvvvvvvv

                    *P*

FY=60,000.00
Fc=3,000.00
D=24.00
D1=4.00
AS1=0.60
N=8.00
P=140.00
MX=214.00
MY=1.00E-8

REBAR COORDINATES
Rb=8.00 ∠=0.00
Rb=8.00 ∠=45.00
Rb=8.00 ∠=90.00
Rb=8.00 ∠=135.00
Rb=8.00 ∠=180.00
Rb=8.00 ∠=225.00
Rb=8.00 ∠=270.00
Rb=8.00 ∠=315.00

<.8>PMAX=857.61

ebR=8.75
MbR=2,942.06
PbR=336.23
cbR=11.84
B∠=2.68E-9

P=139.60
MX=205.36 ≈ 214 ∴ O.K
MY=9.60E-9
cR=8.44
ecR=1.47
```

Abutment Design

Brom's Method to Check Embedment Length - See Appendix

Check 2 cases:

1) Cohesive Case: $K_h D = 400$ #/in²

2) Cohesionless Case $\eta_h = 60$ #/in³

Abut. Shafts: $D = 24''$; $I_g = 16{,}286$ in⁴ ; $E_c = 3122$ ksi

Soil: $\phi = 30°$; $c_u = 900$ psf; $K_p = 3.0$; $\gamma = 120$ #/ft³

$L = 31'$

Cohesive Case:

$$\beta = \sqrt[4]{\frac{K_h D}{4EI}} = \sqrt[4]{\frac{400}{4(3122)(16286)}} = 0.00666$$

$\beta L = 0.00666(31)(12) = 2.45 > 2.25$ ∴ Flexible Pile Action

Cohesionless Case

$$\eta = \sqrt[5]{\frac{\eta_h}{EI}} = \sqrt[5]{\frac{.06}{(3122)(16286)}} = 0.0164$$

$\eta L = 0.0164(31)(12) = 6.1 > 2.0$ ∴ Flexible Pile Action

∴ Abutment Shaft Embedment Length Adequate

- No longer a factor - flexible pile action takes over.

Abutment Design

Bearing Capacity of Caisson

@ 1299' Soil Bearing Capacity = 185^k per SHB report

Axial Load = 104^k < 185^k - See Appendix

∴ O.K. Tip Elev. = 1299'

Investigate Need For Additional Caisson due to Skew:

Ref: Bridge Planning & Design Manual

No. of Piles Required under 1/4 Support @ Obtuse corner:

$$\frac{\text{"k" x reaction on piles}}{\text{allow. pile value}}$$

Use "k" = 0.50

Reaction on piles: DL: (16.0 ^{k}/ft)(151.3') + 4.0 ^{k}/ft (14') = 964^k
LL: (48 ^{k}/lane)(6 lanes) = 288^k
1252^k

Allow. Pile Value: 185^k (Soil Bearing Capacity Controls)

Req'd No. of Piles = $\frac{0.50(1252)}{185}$ = 3.38

∴ Shafts @ 13'-0" O.C. O.K.

Abutment Design - 9' Cap Supplement

Vertical Loads - Cap Design

Weight of Abutment: $(0.150\ k/ft^3)(2.5')(12') = 4.5\ k/ft$

Total DL = $1.70\ k/ft + 4.50\ k/ft = 6.2\ k/ft$ Use $7.0\ k/ft$

Total D+L = $7.0\ k/ft + 2.0\ k/ft = 9.0\ k/ft$

$\pm M_{max} = 0.1(9\ k/ft)(13')^2 = 152\ k\text{-}ft$

Use 3 #7 T & B

$A_s = 1.80\ in^2$; $n = 9$; $d = (9')(12''/1') - 6'' - 0.625 - 0.875/2 = 100.9''$ Use $d = 100''$

$n\frac{A_s}{bd} = 9\frac{1.80}{(30)(100)} = 0.0054$ $k = 0.099$; $j = 0.967$

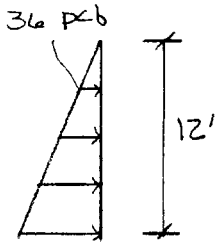
$f_s = \frac{152(12)}{(1.80)(.967)(100)} = 10.5\ ksi < 24\ ksi$

$f_c = \frac{0.099(10.5)}{9(1-.099)} = 0.13\ ksi < 1.2\ ksi$ ∴ O.K

Lateral Loads - Cap Design

$T = 1.7\ k/LF \Rightarrow T_\perp = 0.9\ k/LF$, $T_{\parallel} = 1.4\ k/LF$

Earth Pressure

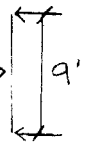


$E_\perp = .036(12')^2/2 = 2.6\ k/ft$

$(E+T)_\perp/LF = 0.9\ k/LF + 2.6\ k/LF = 3.5\ k/LF$

Design Load = $3.5/1.25 = 2.8\ k/LF$ Use $3.0\ k/LF$

$M = 3.0(9')/4 = 6.75\ k\text{-}ft/ft$



Abutment Design - 9' Cap Supplement

Use #5 @ 12" - Lateral Reinforcement

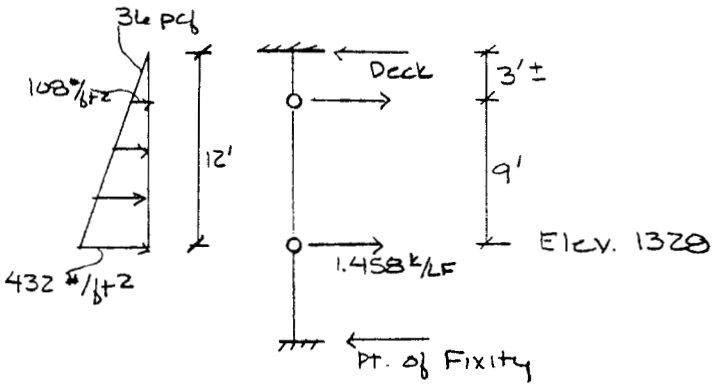
$n \frac{A_s}{bd} = 0.0089$, $k = 0.125$; $j = 0.958$

$f_s = \frac{6.75(12)}{(0.31)(0.958)(26)} = 10.5$ ksi < 20 ksi

$f_c = \frac{0.125(10.5)}{9(1-0.125)} = 0.17$ ksi < 1.2 ksi ∴ O.K.

Check Shear: $v_{max} = 30/12(26) = 0.0096$ ksi < 26 psi ∴ O.K.

Caisson Design



Caisson L = 1328 - 1299 = 29'

Analyze Using DM7 and cracked section

Interior Caisson Lateral Load = (1.458 k/LF)(13') = 18.95 k

Exterior Caisson Lateral Load = (1.458 k/ft)(4.17') + (.036)(14')²/2(7')
= 30.8 k

∴ Exterior Caisson Controls Design

$E_{TRANS} = 26.3^k$
$E_{LONG} = 16.1^k$

Abutment Design - 9' Cap Supplement

Temperature Force $L/T = 29(12)/50.8 = 6.9$; $F_\Delta = 2.3$; $\Delta = 0.32''$

$$P_\Delta = \frac{0.32(20\,338\,000)}{2.3(50.8)^3} = 21.6^{k}/\text{shaft}$$

$$E+T = ((26.3)^2 + (16.1 + 21.6)^2)^{1/2} = 46^{k}$$

$$M_{max} @ Z/T = 1.3 = 0.77(46^{k})(50.8/12) = 150^{k\text{-}ft}$$

$$\text{Axial Load} = (4.5^{k}/ft)(7') + (9.0^{k}/ft)(4.17') = 69^{k}$$

Case IV Design Loads: $M_u = 150/0.35(1.25) = 343^{k\text{-}ft}$

$P_u = 69/0.35(1.25) = 158^{k}$

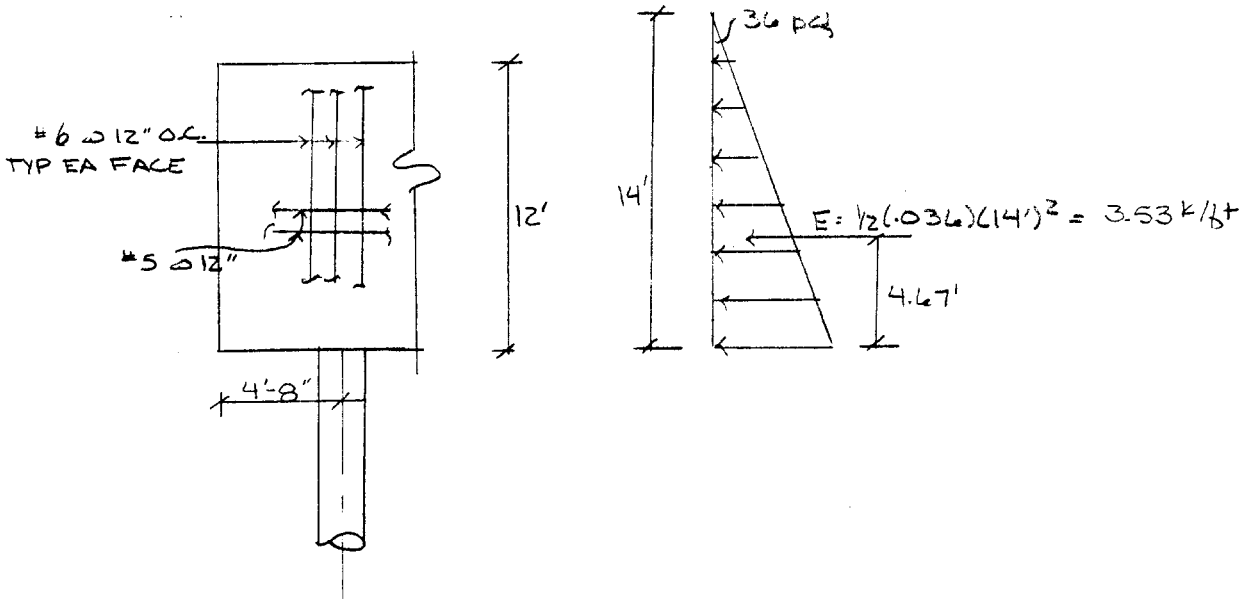
Use 8 #11 - See Circon output

$322.23/343.0 = 1.065$ ∴ O.K.

```
CIRCON

FY=60,000.00
Fc=3,000.00
D=24.00
D1=4.14
AS1=1.56
N=8.00
P=158.00
MX=343.00
MY=1.00E-8

REBAR COORDINATES
Rb=7.87 ∡=0.00
Rb=7.87 ∡=45.00
Rb=7.87 ∡=90.00
Rb=7.87 ∡=135.00
Rb=7.87 ∡=180.00
Rb=7.87 ∡=225.00
Rb=7.87 ∡=270.00
Rb=7.87 ∡=315.00

(.8)PMAX=1,047.52

ebR=13.32
MbR=3,917.24
PbR=294.12
cbR=11.76
B∡=1.67E-9

P=157.33
MX=322.23
MY=9.39E-9
cR=10.29
ecR=2.05
```

Abutment Design - 9' Cap Supplement

Check End Cantilever Wall



Vertical Cantilever:

$M = (3.53\ k/ft)(4.67') = 16.5\ k\text{-}ft/ft \qquad A_s/ft = 0.44\ in^2 \qquad n = 9$

$d = 30'' - 2'' - 0.75''/2 = 27.6'$ use $d = 27''$ $\quad n\frac{A_s}{bd} = 9\frac{0.44}{(12)(27)} = 0.0122 \Rightarrow k = 0.145,\ j = 0.952$

$f_s = \frac{16.5(12)}{(0.44)(.952)(27)} = 17.5\ ksi < 20.0\ ksi$

$f_c = \frac{0.145(17.5)}{9(1-0.145)} = 0.33\ ksi < 1.20\ ksi$

Horiz. Cantilever

$M = 3.53(4.67)^2/2 = 38.5\ k\text{-}ft \qquad A_s = 12(0.31) = 3.72\ in^2;\ d = 26''$

$n\frac{A_s}{bd} = 9\frac{3.72}{(144)(26)} = 0.00894 \quad k = 0.125;\ j = 0.958$

$f_s = \frac{38.5(12)}{(3.72)(.958)(26)} = 4.99\ ksi < 20\ ksi \quad \therefore$ O.K.

$f_c = \frac{0.125(4.99)}{(9)(1-0.125)} = 0.08\ ksi < 1.2\ ksi \quad \therefore$ O.K.

Abutment Design

use 24" Ø Shafts w/ 8 #11 Verts ;

1/2" Ø Spiral ties @ 6" pitch

; tip elev = 1299' (total 13 per abut.)

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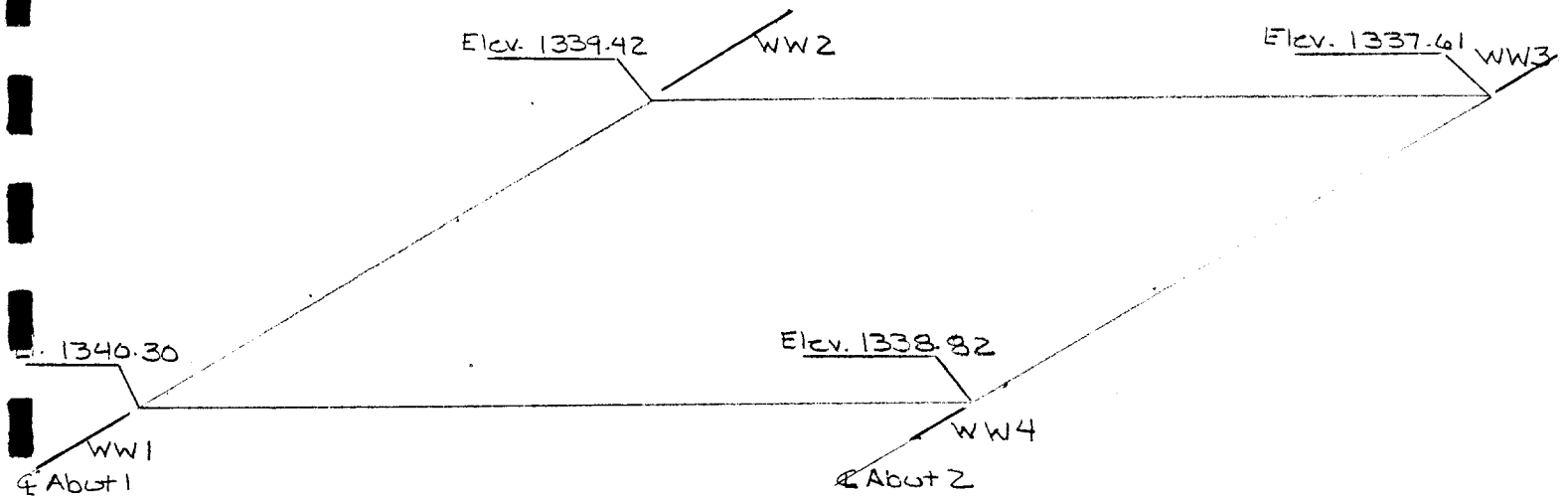
Wingwall Design

Wingwall Design



Key Plan

Ref: The WLB Group Civil Dwgs - Contours

Required Wingwall Lengths

Proposed Fldwy Elev. @ Abut = 1332.55'

Existing Grades:
WW1: Elev = 1337'
WW2: Elev = 1330' ← Fldwy Elev = 1332.55 > 1330'
WW3: Elev = 1335'
WW4: Elev = 1337'

↑ Use for Existing Elev.

3:1 Slope Begins 2' from Edge of Deck

Abutment Wall Extends 7' each side of bridge

Req'd wingwall length = $[(El_{\text{ABUT}} - El_{\text{EXIST}}) \times 3 + 2] / \cos 58.522° - 7'$

WW1: $l = [3(1340.3 - 1337) + 2] / \cos 58.522° - 7' = 16'$ Use 25'
WW2: $l = [3(1339.42 - 1332.55') + 2] / \cos 58.522° - 7' = 36'$ Use 35'
WW3: $l = [3(1337.61 - 1335) + 2] / \cos 58.522° - 7' = 12'$ Use 15'
WW4: $l = [3(1338.82 - 1337) + 2] / \cos 58.522° - 7' = 7'$ Use 15'

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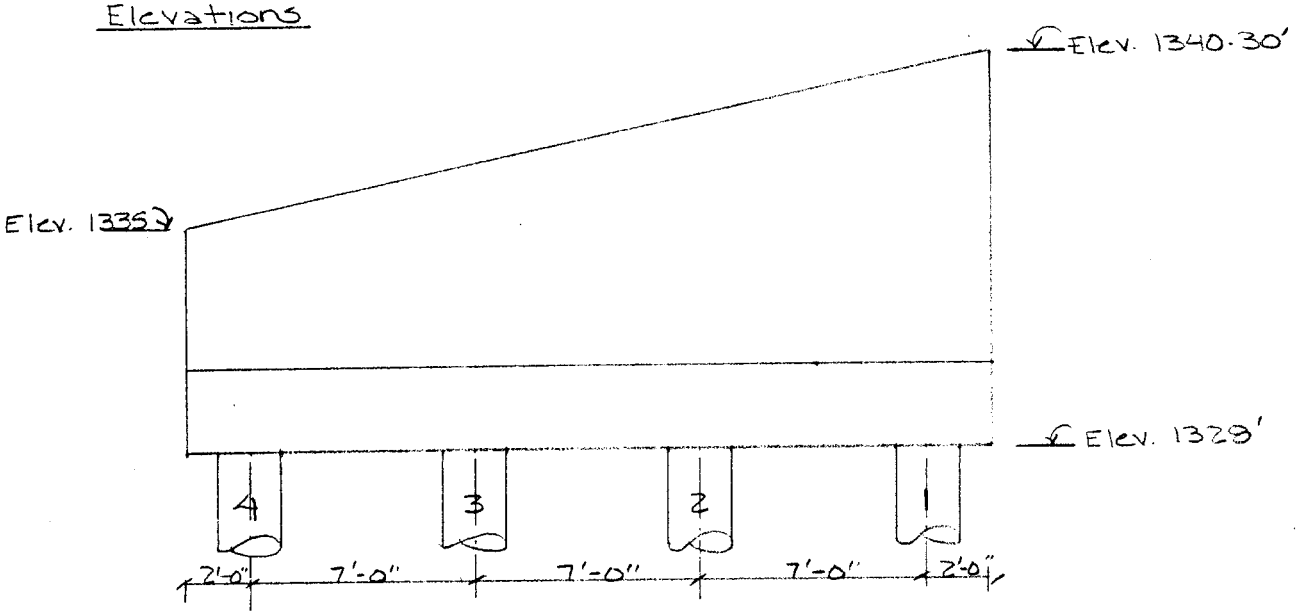
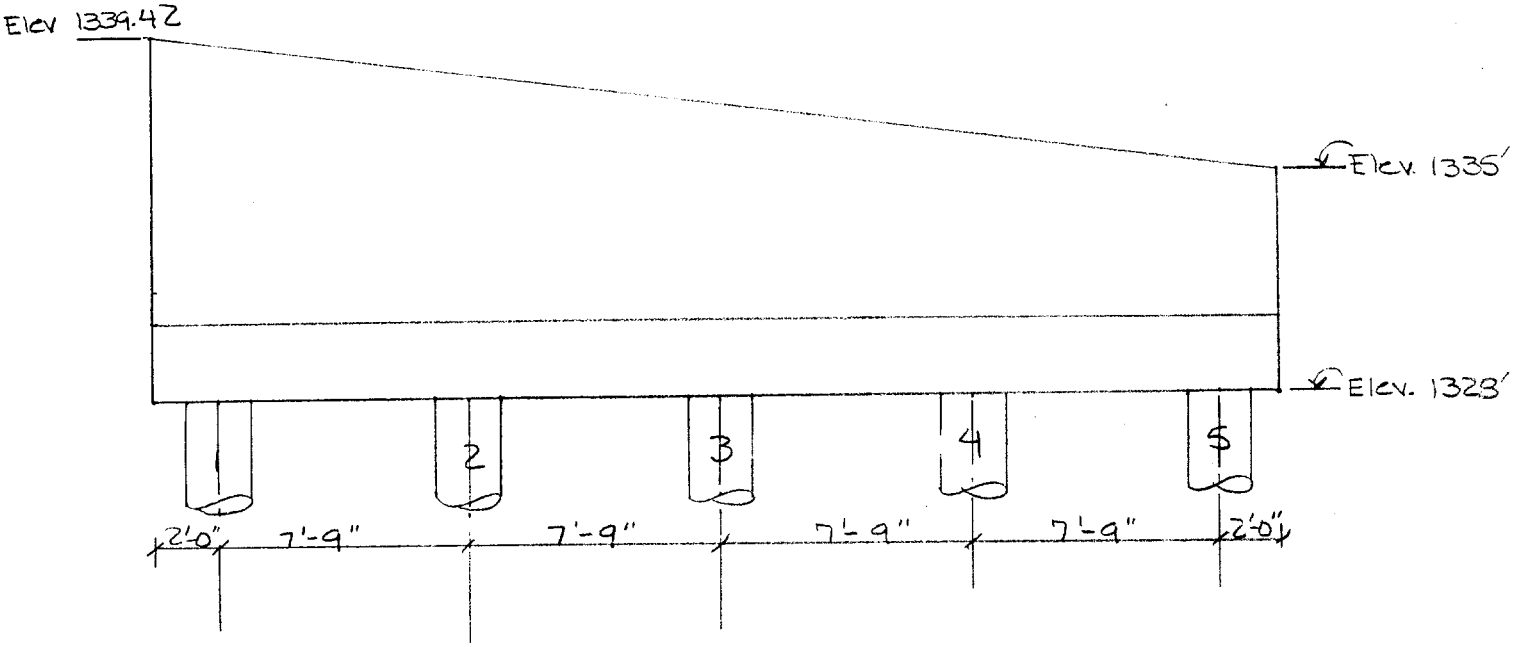
Wingwall Design

Elevations



Wingwall 1



Wingwall 2

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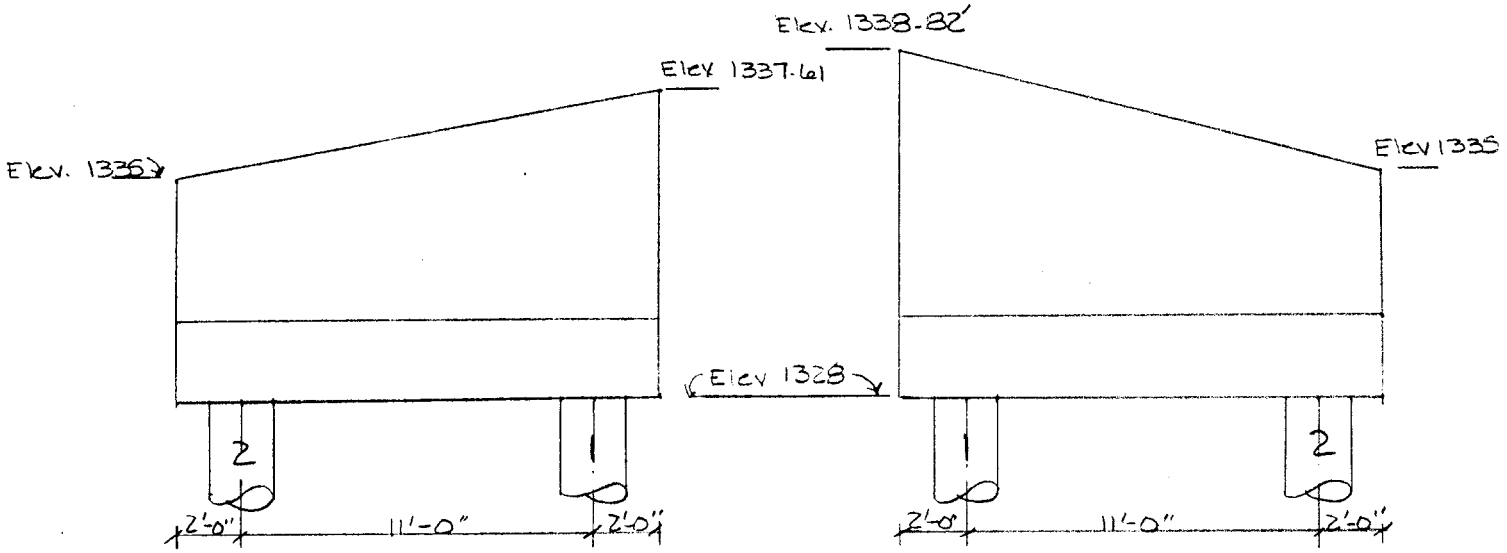
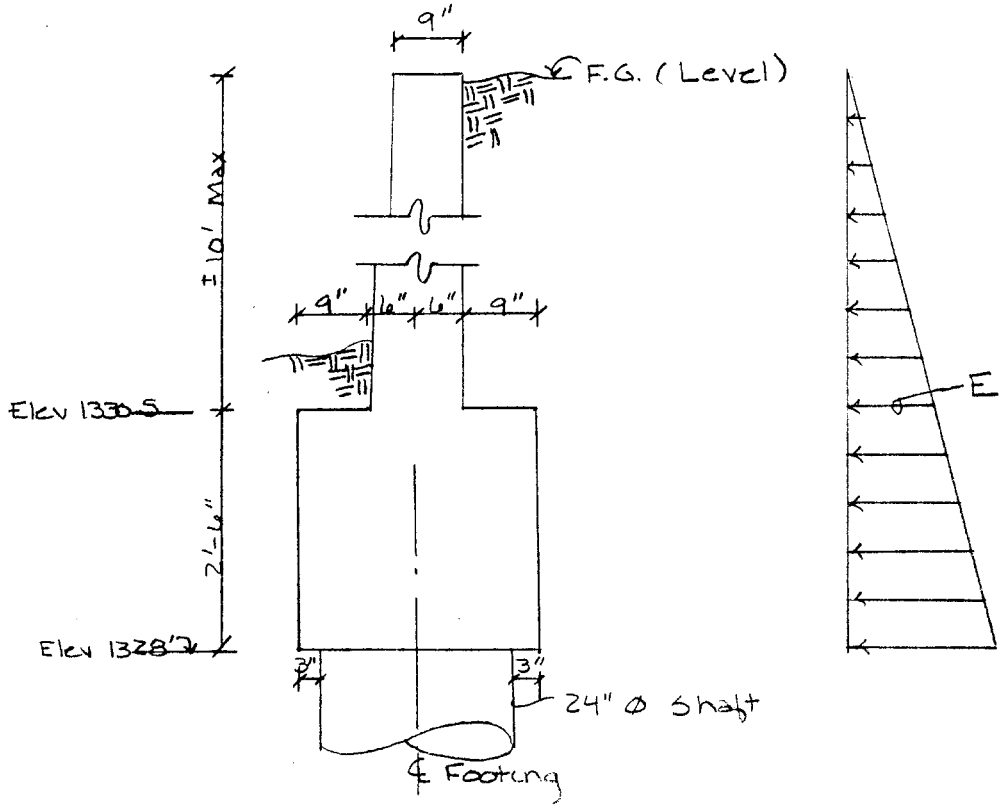
Wingwall Design



Wingwall 3

Wingwall 4

Configuration:



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Wingwall Design

Design Parameters

1.) Service Load Design
2) $F'_c = 3000$ psi, $n = 9$
3) $F_y = 40$ ksi ($\leq$ #6) $F_s = 20$ ksi
   $F_y = 60$ ksi ($\geq$ #7) $F_s = 24$ ksi
4) Soil Unit Wgt = 0.11 kcf
5) Neglect earth pressure & cover @ toe
6) Equiv. Fluid Pressure = 0.036 kcf
7) Assume No Surcharge

Note:

- Wall is designed for maximum height of 10'
- Shaft is designed for critical case
  - Wingwall 1 - Shaft 1
    (Tributary length of wall on shaft = 5.5'
    Average wall height = 9.22')

Shaft Design

ΣM @ ℄ Footing:

		Mom Arm	
Wall DL (□) = 0.75'(9.22')(0.15) =	1.04 k/'	× 0.125'	= 0.13 k-'/' ↻
Wall DL (△) = ½(0.25')(9.22')(0.15) =	0.17 k/'	× 0.333'	= 0.06 k-'/' ↺
Soil DL (□) = (0.75')(9.22')(0.11) =	0.76 k/'	× 0.875'	= 0.67 k-'/' ↻
Ftg. DL = (2.5')(2.5')(0.15) =	0.94 k/'	× 0	= 0
	$\Sigma V = 2.91$ k/'		
Earth Press. = ½(.036)(11.72')² =	2.47 k/'	× 11.72'/3	= 9.66 k-'/' ↺
	$\Sigma H = 2.47$ k/'		$\Sigma M = 8.92$ k-'/' ↺

Wingwall Design

Shaft Bearing Capacity

P: 2.91 K/LF

Per Geotechnical Report Min. Embedment = 15'

∴ Use 24" Ø Shafts w/ Tip Elev. = 1313

Bearing Capacity = 70K (per Geo Report)

Max. Spacing = 70/2.91 = 24' ∴ O.K.

Shaft Design

Group I Loading: 1.0[1.0D + 1.0E]

Design Loads:

$P_u = 2.91(5.5)/0.35 = 46^K$

$V_u = 2.47(5.5)/0.35 = 39^K$

$M_u = 8.92(5.5)/0.35 = 140^{k\text{-}ft}$

DM-7:

$f_r = 7.5 \times 10^{-3}\sqrt{3000} = 0.411$ ksi

$I_g = \pi(12'')^4/4 = 16286 \text{ in}^4$

$M_{cr} = 0.411(16286)/(12)^2 = 46^{k\text{-}ft} < 140^{k\text{-}ft}$ ∴ Section Cracks

Assume $EI = \frac{57\sqrt{3000}(16286)}{2.5} = 2.03 \times 10^7 \text{ k-in}^2$

$F = 60 \text{ \#/in}^3$ per Geo Report

$T = \left(\frac{2.03 \times 10^7}{.06}\right)^{1/5} = 50.8''$ $L/T = 15(12)/50.8 = 3.5$

Wingwall Design

Check δ @ top of wall

$$\delta \text{ @ } z/T = 0 = (1.6\,MT^2 + 2.5\,VT^3)/EI$$
$$= (1.6(140)(12)(50.8)^2 + 2.5(39)(50.8)^3)/2.03\times10^7$$
$$= 0.97''$$

$\delta = 0$ @ $z/T \approx 2$

δ @ top of wall assuming linear profile:

$$\delta = \frac{2(50.8/12) + (9.22' + 25')}{2(50.8/12)}(0.97'') = 2.3'' < 3.0'' \text{ Batter} \therefore \text{O.K.}$$

Find M_{max}

@ $z/T = 0$ $\quad M_m = 1.0(140) = 140^{k\text{-}ft}$
$M_p = 0$

@ $z/T = 1.25$ $\quad M_m = 0.76(140) = 106^{k\text{-}ft}$
$M_p = 0.73(39)(50.8/12) = 121^{k\text{-}ft}$
$227^{k\text{-}ft}$ ← Controls

Design Loads @ $z/T = 1.25$

$$P_u = 46^k + 1.25(50.8/12)\pi(1')^2(.150)/0.35 = 53^k$$
$$M_u = 227^{k\text{-}ft}$$

Shaft Reinf.

$\boxed{\text{Use } 12\ \#7}$ ← For Development Considerations

See Circon Output
$D1 = 3'' + 0.5'' + 0.875''/2 = 3.94'$
$AS1 = 0.60$
$N = 12$

Wingwall Design

Req'd l_d

@ M_{max}: $\frac{0.04(0.60)(60000)}{\sqrt{3000}} = 26.29''$ (@ $Z/T = 1.25$)

$Z = 1.25(50.8) = 63.5'' > 26.29''$

@ Top of Shaft:

$\left(\frac{140}{227}\right)26.29'' = 16.2''$ $\quad 30'' - 3'' - 6'' = 21'' > 16.2''$

∴ O.K.

Shafts–
Use 12 #7 Verts
1/2" Ø Spiral Ties @ 6" pitch
Bottom Elev. = 1313.0'

```
CIRCON

FY=60,000.00
Fc=3,000.00
D=24.00
D1=3.94
AS1=0.60
N=12.00
P=53.00
MX=227.00
MY=1.00E-8

REBAR COORDINATES
Rb=8.06 ∠=0.00
Rb=8.06 ∠=30.00
Rb=8.06 ∠=60.00
Rb=8.06 ∠=90.00
Rb=8.06 ∠=120.00
Rb=8.06 ∠=150.00
Rb=8.06 ∠=180.00
Rb=8.06 ∠=210.00
Rb=8.06 ∠=240.00
Rb=8.06 ∠=270.00
Rb=8.06 ∠=300.00
Rb=8.06 ∠=330.00

<.8>PMAX=877.65

ebR=10.08
MbR=3,694.11
PbR=366.64
cbR=11.87
B∠=2.52E-9

P=52.38
MX=242.61 > 227
MY=1.07E-8 ∴ O.K.
cR=7.63
ecR=4.63
```

Wingwall Design

Wall Reinf

Max wall hgt

M @ base of wall = $\frac{1}{2}(.036)(10')^2(10'/3) = 6.0$ k-ft/LF

Try #5 @ 8" o.c.

$d = 12" - 2" - 0.625"/2 = 9.69"$ Use $d = 9"$; $b = 12"$, $A_s = 0.31\,(12/8) = 0.465\ in^2$

$n\frac{A_s}{bd} = 9\frac{0.465}{(12)(9)} = 0.03875 \Rightarrow k = 0.242$; $j = 0.919$

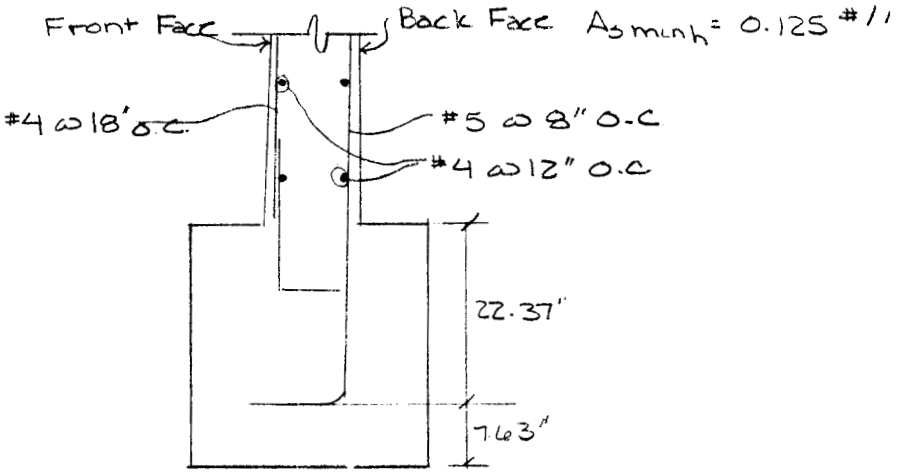
$f_s = \frac{M}{A_s j d} = \frac{6.0(12)}{(.465)(.919)(9)} = 18.72$ ksi < 20.0 ksi

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.242(18.72)}{9(1-0.242)} = 0.66$ ksi < 1.20 ksi

∴ O.K. Use #5 Verts @ 8" o.c. - Back Face

Development of #5 Bars

$l_d = 0.04(0.31)(40000)/\sqrt{3000} = 9.1" < 22.37"$ ∴ O.K



Wall Reinf.

Use #5 Vert @ 8" o.c. w/ STD 90° Hook @ Back Face
#4 Vert @ 18" o.c. @ Front Face
#4 Horiz @ 12" o.c Each Face

Wingwall Design

Footing Design

Longitudinal Footing Reinf.

$P = 2.91^k/LF$

$M_{max} \approx 2.91(7')^2/10 = 14.3^{k\text{-}ft}$

$d = 30'' - 6'' - 0.5'' - 1.128''/2 = 22.9''$ use $d = 22''$; $b = 30''$

Try 3 #9 $A_s = 3(1.00) = 3.00 \text{ in}^2$

$n\frac{A_s}{bd} = 9\frac{3.00}{(30)(22)} = 0.0409 \Rightarrow k = 0.248;\ j = 0.917$

$f_s = \frac{14.3(12)}{(3.00)(.917)(22)} = 2.84 \text{ ksi} < 24 \text{ ksi}$

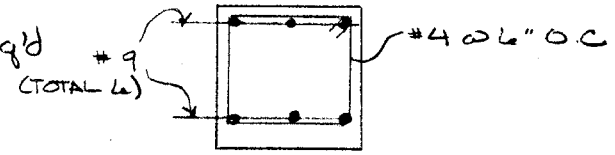
$f_c = \frac{0.248(2.84)}{9(1-0.248)} = 0.10 \text{ ksi} < 1.20 \text{ ksi}$ ∴ O.K. Use 3 #9 T & B

Shear Reinf.

$V_{max} = 2.91(7)/2 = 10.2^k$ $v = 10.2/(30)(22) = 0.015 \text{ ksi}$

$v_c = 0.95\sqrt{3000} = 52 \text{ psi} > 2(15 \text{ psi})$

∴ No Special Shear Reinf Req'd



Check Torsion

AASHTO 1.5.29 - Use 'Building Code Requirements for Reinforced Concrete - ACI 318-77' for combined torsion and shear.

Torsional Moment - $(6.0^{k\text{-}ft}/ft)(7'/2) = 21^{k\text{-}ft}$

Wingwall Design

Check if Torsion Can be Neglected = (ACI B.7.3 & 11.6.1)

$\phi(0.5\sqrt{F'_c}\,\Sigma x^2 y) = 0.85(0.5)\sqrt{3000}(30)^2(30) = 628512^{k\text{-}in} = 52.4^{k\text{-}ft}$

$0.55(52.4) = 28.8^{k\text{-}ft} > 21.0^{k\text{-}ft}$

∴ O.K. Torsion Reinf. Not Req'd.

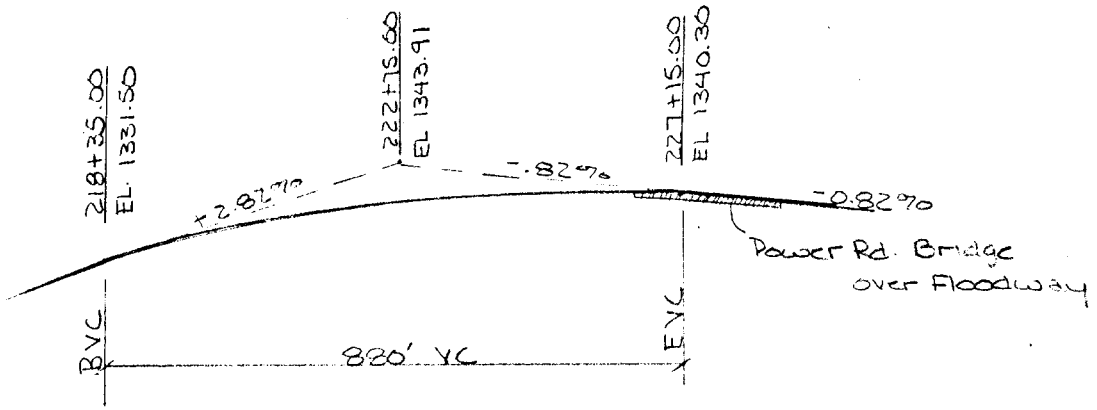
Footing Reinf. Use 3 #9 Cont. Top & Bottom #4 ▨ Stirrups @ 6" O.C.

BRIDGE GEOMETRY

Geometry

Deck Elevations

Vertical Curve Ref: The WLB Group Roadway Dwgs



Profile Grade @ ℄ Constr

@ ℄ Constr.:

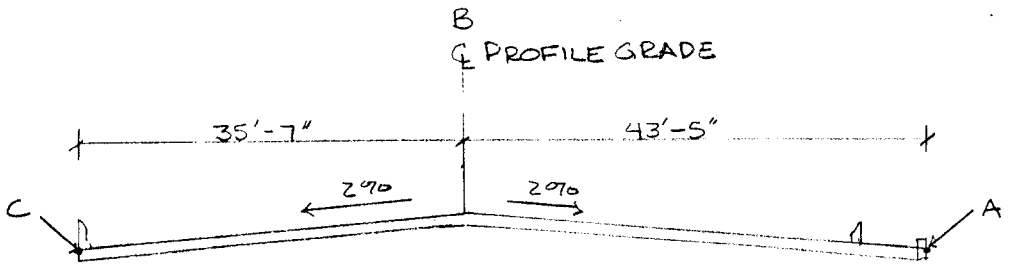
Location	STATION	ELEVATION*
℄ Abut 1	226 + 45.97	1340.77
℄ Pier 1	226 + 78.39	1340.57
℄ Pier 2	227 + 17.39	1340.28
℄ Pier 3	227 + 56.39	1339.96
℄ Pier 4	227 + 95.39	1339.64
℄ Pier 5	228 + 34.39	1339.32
℄ Abut 2	228 + 66.81	1339.06

* Top of Deck

Bridge Skew = 58°-31'-20"

Geometry

Offsets



SECTION

Line C -

Station Offset = $-35.58' \tan 58{:}52^\circ = -58.11'$

Vertical Offset = $.02(35.58') = -0.71'$

Line A -

Station Offset = $43.42' \tan 58.52^\circ = 70.91'$

Vertical Offset = $.02(43.42') = -0.87'$

Geometry

Theoretical Deck Elevations

-Stations

SPAN 1					
LINE	℄ ABUT 1 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 1 STA
C	225+87.85	225+95.96	226+04.06	226+12.17	226+20.27
B	226+45.97	226+54.08	226+62.18	226+70.29	226+78.39
A	227+16.88	227+24.99	227+33.09	227+41.20	227+49.30

SPAN 2					
LINE	℄ PIER 1 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 2 STA
C	226+20.27	226+30.02	226+39.77	226+49.52	226+59.27
B	226+78.39	226+88.14	226+97.84	227+07.64	227+17.39
A	227+49.30	227+59.05	227+68.80	227+78.55	227+88.30

SPAN 3					
LINE	℄ PIER 2 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 3 STA
C	226+59.27	226+69.02	226+78.77	226+88.52	226+98.27
B	227+17.39	227+27.14	227+36.89	227+46.64	227+56.39
A	227+88.30	227+98.05	228+07.80	228+17.55	228+27.30

SPAN 4					
LINE	℄ Pier 3 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 4 STA
C	226+98.27	227+08.02	227+17.77	227+27.52	227+37.27
B	227+56.39	227+66.14	227+75.89	227+85.64	227+95.39
A	228+27.30	228+37.05	228+46.80	228+56.55	228+66.30

SPAN 5					
LINE	℄ Pier 4 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 5 STA
C	227+37.27	227+47.02	227+56.77	227+66.52	227+76.27
B	227+95.39	228+05.14	228+14.89	228+24.64	228+34.39
A	228+66.30	228+76.05	228+85.80	228+95.55	229+05.30

SPAN 6					
LINE	℄ PIER 5 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ ABUT 2 STA
C	227+76.27	227+84.38	227+92.48	228+00.59	228+08.69
B	228+34.39	228+42.49	228+50.60	228+58.70	228+66.81
A	229+05.30	229+13.41	229+21.51	229+29.62	229+37.72

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Geometry

Theoretical Deck Elev. (cont)

- Elevations

SPAN 1					
LINE	℄ ABUT 1 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 1 ELEV
C	1340.30	1340.27	1340.24	1340.21	1340.18
B	1340.77	1340.72	1340.68	1340.63	1340.57
A	1339.42	1339.35	1339.28	1339.22	1339.15

SPAN 2					
LINE	℄ PIER 1 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 2 ELEV
C	1340.18	1340.14	1340.09	1340.04	1339.98
B	1340.57	1340.51	1340.43	1340.36	1340.28
A	1339.15	1339.07	1338.99	1338.91	1338.83

SPAN 3					
LINE	℄ PIER 2 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 3 ELEV
C	1339.98	1339.92	1339.86	1339.79	1339.72
B	1340.28	1340.20	1340.12	1340.04	1339.96
A	1338.83	1338.75	1338.67	1338.59	1338.51

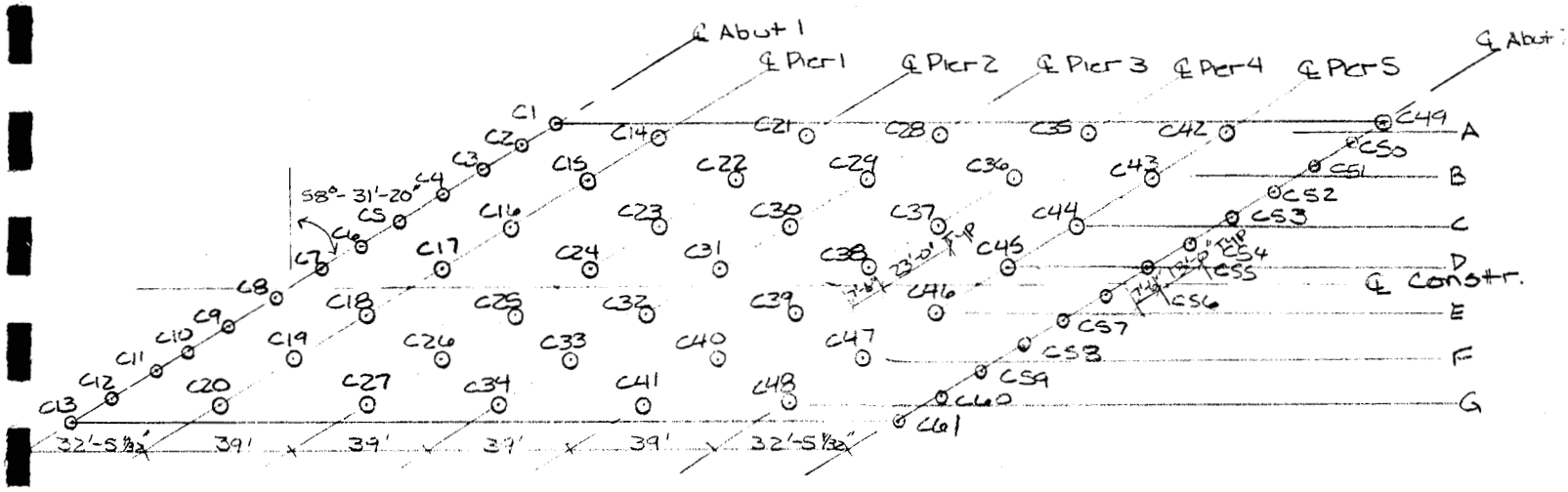
SPAN 4					
LINE	℄ PIER 3 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 4 ELEV
C	1339.72	1339.64	1339.57	1339.49	1339.41
B	1339.96	1339.88	1339.80	1339.72	1339.64
A	1338.51	1338.43	1338.35	1338.27	1338.19

SPAN 5					
LINE	℄ PIER 4 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 5 ELEV
C	1339.41	1339.33	1339.25	1339.17	1339.09
B	1339.64	1339.56	1339.48	1339.40	1339.32
A	1338.19	1338.11	1338.03	1337.95	1337.87

SPAN 6					
LINE	℄ PIER 5 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ ABUT 2 ELEV
C	1339.09	1339.02	1338.95	1338.89	1338.82
B	1339.32	1339.25	1339.19	1339.12	1339.06
A	1337.87	1337.80	1337.74	1337.67	1337.61

Geometry

Caisson Cut-Off Elevations



Abut 1 - Caisson Cut-offs - C1-C13

B.O. Abutment Cap Elev = 1328.'

Caisson cutoff = 1328' + 3" = 1328.25'

Abut 2 - Caisson Cut-offs - C49-C61

B.O. Abutment Cap Elev = 1328'

Caisson Cutoff = 1328' + 3" = 1328.25'

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Geometry

Theoretical Pier Caisson Cut-offs

GRID LINE	STATION OFFSET	VERT. OFFSET
A	+65.24'	-3.05'
B	+45.63'	-2.81'
C	+26.01'	-2.57'
D	+ 6.40'	-2.33'
E	-13.22'	-2.41'
F	-32.83'	-2.65'
G	-52.45'	-2.89'

Caisson	STATION	ELEV
C14	227+43.63	1337.02'
C15	227+24.02	1337.42'
C16	227+04.40	1337.82'
C17	226+84.79	1338.20
C18	226+65.17	1338.25
C19	226+45.56	1338.12
C20	226+25.94	1337.97
C21	227+82.63	1336.70'
C22	227+63.02	1337.10'
C23	227+43.40	1337.50'
C24	227+23.79	1337.90
C25	227+04.17	1337.97
C26	226+84.56	1337.88
C27	226+64.94	1337.77
C28	228+21.63	1336.38'
C29	228+02.02	1336.78'
C30	227+82.40	1337.18
C31	227+62.79	1337.58
C32	227+43.17	1337.66

CAISSON	STATION	ELEV
C33	227+23.56	1337.58
C34	227+03.94	1337.50
C35	228+60.63	1336.06
C36	228+41.02	1336.46
C37	228+21.40	1336.86
C38	228+01.79	1337.26
C39	227+82.17	1337.34
C40	227+62.56	1337.26
C41	227+42.94	1337.18
C42	228+99.63	1335.74
C43	228+80.02	1336.14
C44	228+60.40	1336.54
C45	228+40.79	1336.94
C46	228+21.17	1337.02
C47	228+01.56	1336.94
C48	227+81.94	1336.86

QUANTITIES

Quantities

Superstructure

4000 psi Concrete

Slab - (220.84' − 4.79')(79')(18½"/12"/1)/(27 cu ft/cu yd) = 975 cu yd.

About 1 - (151.29')(1340.77' − (2(1340.77) − 1340.30 − 1339.42)/4 − 1337.0')(2.5')/(27 cu ft/cu yd) = 46 cu yd.

About 2 - (151.29')(1339.06' − (2(1339.06) − 1338.82 − 1337.60)/4 − 1335.0')(2.5')/(27 cu ft/cu yd) = 50 cu yd.

Piers 1 ⇒ 5 - 5[(151.29')(6')(7.5"/12"/1)/(27 cu ft/cu yd)] = 105 cu yd

Includes haunch

Edge Beam - ((220.84')[(17")(3") + (12.1")(7") + (4.4")(3.5")]/144 in²/ft²)/27 = 9 cu yd

1185 cu yd

Reinforcing Steel - Grade 60

Slab

- Long. Deck Reinf.

b1 #8 x 34.42' ⇒ (79+80)(34.5')(2.67 #/ft) = 14,646 #

b2 #10 x 46.29' ⇒ (79+80)(46.5')(4.303 #/ft) = 31,814 #

b3 #10 x 51.38' ⇒ 4(159)(51.5')(4.303 #/ft) = 140,940 #

187,400 #

Piers

- #11 Top Bars - 5[10(151') + 10(3)(7')] 5.313 #/ft = 45,692 #
- #9 Bott Bars - 5[10(151') + 10(3)(3.5')] 3.400 #/ft = 27,455 #

73,147 #

Edge Beam -

- 2 - #10 - 2(220.5' + 466.5')) 4.303 #/ft = 2121 #

Quantities

Superstructure - cont.

Reinforcing Steel - Grade 40

Slab

- Long. Deck Reinf.

b4 #6 x 24.0' ⇒ (79+80)(24')(1.502 #/ft) = 5732 #

b5 #6 x 21.75' ⇒ 4(80)(22')(1.502 #/ft) = 10574 #

- Transverse Reinf.

#5 Top Bars - 2[61(79'+2')]1.043 #/ft = 10,307#
#4 Top Bars - 105(79'+1.5')0.6668 #/ft = 5646#
#5 Bott Bars - 331 (79'+2')1.043 #/ft = 27964#
60,223#

Abutment 1-

5 #5 Str Bars - 5(151'+2(2'))(1.043 #/ft) = 808#
#4 - 152(5')(0.6668 #/ft) = 508#
#4 - 152(6')(0.6668 #/ft) = 609#
#6 Dowels - 152(4')(1.502 #/ft) = 913#
2838#

Abutment 2- = 2838#

Piers 1-5

- #5 Stirrups - 5[302(9')](1.043 #/ft) = 14174#

Additional Slab Reinf (Grade 40) under Curb & Barriers.

#4 x 5' - 442(5')(.6668 #/ft) = 1476#
#5 Cont - 2(220'+4(2'))1.043 #/ft = 476#
#4 Cont - 6[(220')+4(1.5')].6668 #/ft = 906#
#4 x 6.6' - 590(6.6').6668 #/ft = 2601#
#4 x 5.5' - 221(5.5).6668 #/ft = 812#
} 6271

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Quantities

Abutment Caps

Note: See Revisions at end of Section for Abutment 1

3000 psi Concrete

Abut 1 - [(151.29')(2.5')(7') + (14.05')(2.5')(9.8')] / 27 cu yd/cu ft = 111 cu yd

Abut 2 - [(151.29')(2.5')(7') + (14.05')(2.5')(10.2')] / 27 cu yd/cu ft = 111 cu yd

222 cu yd

Reinforcing Steel - Grade 60

Abut 1

#7 Top & Bott - 6(165' + 3(2')) 2.044 #/ft = 2097#

Abut 2 = 2097#

Reinforcing Steel - Grade 40

Abut 1

10 #5 Str.	- 10(165 + 3(2'))(1.043 #/ft)	= 1784#
#5 [] x 16.2'	- 152(16.2')(1.043 #/ft)	= 2568#
#5 ⊓ x 4'	- 152(4')(1.043 #/ft)	= 634#
#5 [] x 22'	- 14(22')(1.043 #/ft)	= 321#
#5 ⊓ x 4'	- 14(4')(1.043 #/ft)	= 58#
14 #5 Str	- 14(7')(1.043 #/ft)	= 102#
#4 ⊏ x 6.5'	- 22(6.5')(0.668 #/ft)	= 96#
#4 ⊏ x 5'	- 22(5')(0.668 #/ft)	= 73#
		5636#

Abut 2

Same as 1 except:

#5 [] x 22' ⇒ #5 [] x 23' - 14(23')(1.043 #/ft) = 336#

Total 5651#

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Quantities:

Shafts -

Abut 1 -

Note : See Revisions at end of section

Drilled Shafts L.F - 13(1330.25' - 1299') = 406.25 LF

3000 psi Concrete - $\pi(1')^2(406.25)/(27$ cu ft/cu yd) = 47.3 cu yd

Reinforcing - Grade 60

#7 Verts - 13(8)(35')(2.044 #/LF) = 7440#

Reinforcing - Grade 40

1/2" ø Spiral Ties - 13[59(π)(19/12) + 2(.5') + 5(3')].668 #/ft = 2688#

#4 Ties - 13[10(π(1.5) + 1.5)] 0.668 #/ft = 540#

3228#

Pier 1

Drilled Shafts L.F - 1337.02' + 1337.42' + 1337.82' + 1338.20' + 1338.25' + 1338.12' + 1337.97' - 7(1299')

= 271.8 LF

3000 psi Conc - $[\pi(1.25')^2(271.8' - 7(2.5')) + 7(.5'\pi(2.25)^2 + 2'\pi(1.75)^2)]/27$

= 53.3 cu yd.

Reinforcing - Grade 60

#11 Verts - 8[271.8 - 7(1.25')] 5.313 #/LF = 11,181#

#8 Pin - 7[6(6')] 2.67 #/LF = 673#

11,854#

Reinforcing - Grade 40

1/2" ø Spiral Ties - 7[76π(2.1') + 2(.5') + 9(3')].668 #/ft = 2475#

Quantities

Shafts-

Pier 2

Drilled Shafts L.F. - 1336.70 + 1337.10 + 1337.50 + 1337.90
+ 1337.97 + 1337.88 + 1337.77 - 7(1299.0) = 269.8 LF

3000 psi conc. - $[\pi(1.25)^2(269.8' - 7(2.5')) + 7(.5'\pi(2.25)^2 + 2'\pi(1.75')^2)]/27$
= 53.00 yd.

Reinforcing - Grade 60
#11 Verts - 8[269.8 - 7(1.25')] 5.313 #/LF = 11,096#
#8 Pin - 7(6)(6')(2.67 #/LF) = 673#
11,769#

Reinforcing - Grade 40
½" Ø Spiral Ties - 7[75π(2.1') + 2(.5') + 9(3')](.668 #/ft) = 2445#

Pier 3

Drilled Shafts L.F. - 1336.38 + 1336.78 + 1337.18 + 1337.58
+ 1337.66 + 1337.58 + 1337.50 - 7(1299.0) = 267.7 LF

3000 psi conc - $[\pi(1.25)^2(267.7 - 7(2.5')) + 7(.5'\pi(2.25)^2 + 2'(\pi)(1.75')^2)]/27$
= 53.00 yd

Reinforcing - Grade 60
#11 Verts - 8[267.7 - 7(1.25)] 5.313 #/LF = 11,006#
#8 Pin - 7(6)(6')(2.67 #/LF = 673#
11,679#

Reinforcing - Grade 40
½" Ø Spiral Ties - 7[75π(2.1)' + 2(.5') + 9(3')](.668 #/LF) = 2445#

Quantities

Shafts

Pier 4

Drilled Shafts L.F. - 1336.06 + 1336.46 + 1336.86 + 1337.26
+ 1337.34 + 1337.26 + 1337.18 - 7(1299.') = 265.4 L.F.

3000 psi Conc. - $[\pi(1.25)^2(265.4-7(2.5')) + 7(.5\pi(2.25)^2 + 2\pi(1.75)^2)]/27$
= 52 cu yd

Reinforcing - Grade 60
#11 Verts - 8[265.4' - 7(1.25')]5.313 #/ft = 10,909 #
#8 Pin - 7(6)(6')(2.67 #/ft) = 673 #
11,582 #

Reinforcing - Grade 40
1/2" Ø Spiral Ties - 7[74π(2.1') + 2(.5') + 9(3')].668 #/LF = 2414 #

Pier 5

Drilled Shafts L.F. - 1335.74' + 1336.14' + 1336.54' + 1336.94'
+ 1337.02' + 1336.94' + 1336.86' - 7(1299.0') = 263.2 L.F.

3000 psi Conc. - $[\pi(1.25)^2(263.2-7(2.5')) + 7(.5\pi(2.25')^2 + 2\pi(1.75')^2)]/27$
= 52 cu yd

Reinforcing - Grade 60
#11 Verts - 8[263.2 - 7(1.25')]5.313 #/ft = 10 815 #
#8 Pin - 7(6)(6')(2.67 #/ft) = 673 #
11,488 #

Reinforcing - Grade 40
1/2" Ø Spiral Ties - 7[74π(2.1') + 2(.5') + 9(3')].668 #/LF = 2414 #

Quantities

Shafts

Note: See Revisions @ End of section

Abut 2

Drilled Shafts L.F. - 13(1328.25' - 1299') = 380.25 LF

3000 psi Concrete - $\pi(1')^2(380.25')/27$ = 44 cu yd.

Reinforcing - Grade 60

#7 Verts - 13(8)(33')(2.044 #/ft) = 7015 #

Reinforcing - Grade 40

1/2" Ø Spiral Ties - 13 [56(π)(19/12) + 2(1.5') + 5(3')] · .668 #/ft = 2558 #

#4 Ties - 13[10(π(1.5) + 1.5)] 0.668 #/ft = 540 #

3098 #

Misc. Items

Conc Barriers

3000 psi Conc. - (2(220')[(17")(3") + (10")(10") + (5")(7") + (19")(8") + (9.5")(2")]/144)/27

- 8 cu yd (Edge Bm.) = 32 cu yd

Reinforcing - Grade 40

#6 cont - 2[220' + 4(2.5')] 1.502 #/ft = 691 #

#4 cont - 10 [220' + 4(1.5')] 0.668 #/ft = 1510 #

#4 | x 2.5' - 296 (2.5) .668 #/ft = 494 #

#4 ⌒ x 3' - 590 (3') .668 #/ft = 1182 #

3877 #

Quantities

Misc. Items

Conc. Curb

3000 psi Conc - (220.84' + 4.79')(10/12)(9/12)/27 ft³/yd³ = 5 cu yd

Reinforcing - Grade 40

#5 cont 2[225' + 4(2')] 1.043 #/ft = 486#

Approach Slabs

3000 psi Conc. - 2(15')(71')(1')/27 = 79 cu yd

Reinforcing - Grade 60

#8 str - 192(16')(2.67 #/ft) = 8202#

Reinforcing - Grade 40

#5 str - 18(136' + 3(2')) 1.043 #/ft = 2666#

#4 str - 12(136' + 3(1.5')) 0.668 #/ft = 1126#

#4 str - 96(15')(0.668 #/ft) = 962#

#4 ⊓ - 96(5.375')(0.668 #/ft) = 345#

#4 ⊓ - 274(3')(0.668 #/ft) = 549#

5648#

Chain Link Fence

L.F. = 220'

Quantities

Excavation & Backfill

Abut 1 - Existing grade ≈ 4' Below Finished grade
Avg ht of abut = 10.3'

Excavation = [(6.3')(165.33')(3' + 2.5')]/27 cu ft / cu yd = 212 cu.yd

Backfill = 212 cu yd. + 4'(29')(3')/27 = 225 cu yd

Abut 2 - Existing grade ≈ Finished Grade
Avg. ht of abut = 10.6'

Excavation = 10.6'(165.33')(3' + 2.5')/27 = 357 cu yd

Backfill = 357 cu yd − [136'(1')(1.5') + 151.3'(1.5')(1.5')]/27
= 337 cu yd

Approach slabs

Excavation = [(71')(15' − 1.5/cos 58.52°)(1')]/27 = 32 cu yd.

Backfill = [(71')(15')(3')]/27 = 118 cu yd

Agg. Base Course = 2(71')(15' − 8/12)(.5')/27 = 38 cu yd

Quantities

Revise Abut. 1 Quantities for 2' Cap Depth Increase:

Cap: 3000 psi Concrete: (2')(2.5')(165.4') / 27 cuft/cuyd = 31 cuyd

Abut 1 Cap: 3000 psi Concrete = 111 + 31 = 142 cu yd

Reinforcing Steel (In Addition)

#5 []	- 152(4')(1.043 #/ft)	= 634#
#5 []	- 14 (4') (1.043 #/ft)	= 58#
#5 Cont	- 4 (165 + 3(2'))(1.043 #/ft)	= 713#
#4 L	- 4 (6.5)(0.668 #/ft)	= 17#
#4 [	- 4 (5')(0.668 #/ft)	= 13#

1435# + 5636# = 7071#

Drilled Shafts:
- LF & Conc. Quantities same as For Abut 2 Drilled Shafts

Reinf. Steel - Verts - 13(8)(33')(5.313 #/ft) = 18,234# (for Abuts 1 & 2)

Excavation & Backfill

Increase Excavation by [(2')(165.4')(3' + 2.5')]/27 = 67 cu yd
Excavation = 67 + 212 = 279 cu yd

Increase Backfill by [(2')(165.4')(3')]/27 = 37 cu yd
Backfill = 37 + 225 = 262 cu yd

Quantities

Wingwalls - Abutment 1

24" Ø Drilled Shafts

Wingwall 1: 4(15.25') = 61.0'
Wingwall 2: 5(15.25') = 76.25'
137.25'

Concrete (3000 psi):

Shafts : π(1')²(137.25') = 431 ft³
Caps : (2.5')(2.5')(25' + 35') = 375 ft³
Wall 1 : (10.5/12)(7.15')(25') = 156 ft³
Wall 2 : (10.5/12)(6.71')(35') = 205 ft³
1167 ft³ = 43.2 yd³

Reinforcement

Shafts (Both walls)

#7 Verts : 9(12)(16.25')(2.044 #/ft) = 3587#
½" Ø Spirals : 9(144' + 2(2.5'))(0.668 #/ft) = 896#
#4 Ties ○ : 9(3)(16.25')(0.668 #/ft) = 113#

Cap (Both walls)

#9 Cont : 6(25' + 35')(3.400 #/ft) = 1224#
#5 Cont : 2(25' + 35')(1.043 #/ft) = 125#
⊓ #4 Stirrups : (50 + 70)(85')(0.668 #/ft) = 681#
L #4 Dowels : (18 + 24)(2.5')(0.668 #/ft) = 70#

Wall 1

#5 Verts : 38(10')(1.043 #/ft) = 396#
#4 Verts : 18(7')(0.668 #/ft) = 84#
#4 Horiz : 16(25')(0.668 #/ft) = 267#

Wall 2

#5 Verts : 53(9.5')(1.043 #/ft) = 525#
#4 Verts : 24(6.5')(0.668 #/ft) = 104#
#4 Horiz : 14(35')(0.668 #/ft) = 327#
8399#

Quantities

Wingwalls - Abutment 2

24" Ø Drilled Shafts

Wingwall 3: 2(15.25) = 30.5'
Wingwall 4: 2(15.25) = 30.5'
61.0'

Concrete (3000 psi)

Shafts: $\pi(1')^2(61')$ = 192 ft³
Caps : (2.5')(25')(15' + 15') = 188 ft³
Wall 3 : (10.5/12)(5.81')(15') = 76 ft³
Wall 4 : (10.5/12)(6.41')(15') = 84 ft³
540 ft³ = 20 yd³

Reinforcement

Shafts (Both Walls)

#7 Verts : 4(12)(16.25)(2.044 #/ft) = 1594#
½" Ø Spirals : 4(144' + 2(2.5))(0.668 #/ft) = 398#
#4 Ties @ : 4(3)(6.25')(0.668 #/ft) = 50#

Cap (Both Walls)

#9 Cont : 6(15' + 15')(3.400 #/ft) = 612#
#5 Cont : 2(15' + 15')(1.043 #/ft) = 63#
⊡ #4 Stirrups : (30 + 30)(8.5')(0.668 #/ft) = 341#
L #4 Dowels : (11 + 11)(2.5')(0.668 #/ft) = 37#

Wall 3

#5 Verts : 23(8.5')(1.043 #/ft) = 204#
#4 Verts : 11(5.5')(0.668 #/ft) = 40#
#4 Horiz : 14(15')(0.668 #/ft) = 140#

Wall 4

#5 Verts : 23(9.0')(1.043 #/ft) = 216#
#4 Verts : 11(6')(0.668 #/ft) = 44#
#4 Horiz : 14(15')(0.668 #/ft) = 140#
3879#

Quantities

Wingwalls - Excavation & Backfill

Abutment 1

Wingwall 1

Excavation = (1337' - 1328')(2.5' + 3')(25' + 1.5')/27 = 49 cu yd.

Backfill = {[(2.5')(3') + (2.25')(6.5')](25')
+ (1.5')(9')(5.5')]} /27 = 23 cu yd.

Wingwall 2

Excavation = (1330 - 1328)(2.5' + 3')(35' + 1.5')/27 = 15 cu yd

Backfill = [(2.5')(3')(35') + (1.5')(2')(5.5')]/27 = 10 cu yd.

Abutment 2

Wingwall 3

Excavation = (1335 - 1328)(2.5' + 3')(15' + 1.5')/27 = 24 cu yd

Backfill = {[(2.5')(3') + (2.25')(4.5')](15')
+ (1.5')(7')(5.5')} /27 = 12 cu yd

Wingwall 4

Excavation = (1337 - 1328)(2.5' + 3')(15' + 1.5')/27 = 30 cu yd.

Backfill = {[(2.5')(3') + (2.25')(6.5')](25')
+ (1.5')(9')(5.5')}/27 = 23 cu yd.

Quantities

ITEM	APPROX. QUANTITIES							
	EXCAVATION	BACKFILL	3000 CONC	4000 CONC	REINF STEEL	24"Ø SHAFTS	30"Ø SHAFTS	CHAIN LINK FENCE
	C.Y.	C.Y.	C.Y.	C.Y.	LBS	L.F.	L.F.	L.F.
ABUT 1	343	295	229		38,900	518		
PIER 1			54		14,330		272	
PIER 2			53		14,210		270	
PIER 3			53		14,120		268.	
PIER 4			52		14,000		265.	
PIER 5			52		13,900		263	
ABUT 2	411	372	175		32,960	441.		
SUPER-STRUCTURE			37	1185	353,380			220
APPROACH SLABS	32	118	79		13,850			
TOTAL	786	785	784	1185	509,650	959	1338	220

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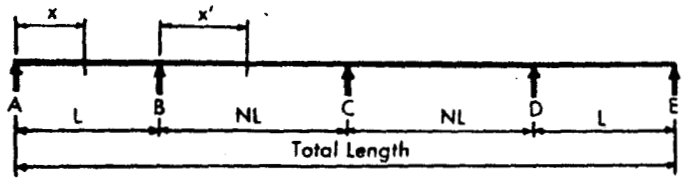
Appendix

TABLE 4.2

Symmetrical four-span continuous beam.
Constant moment of inertia.
AASHO HS20-44 loading.



N=1.2

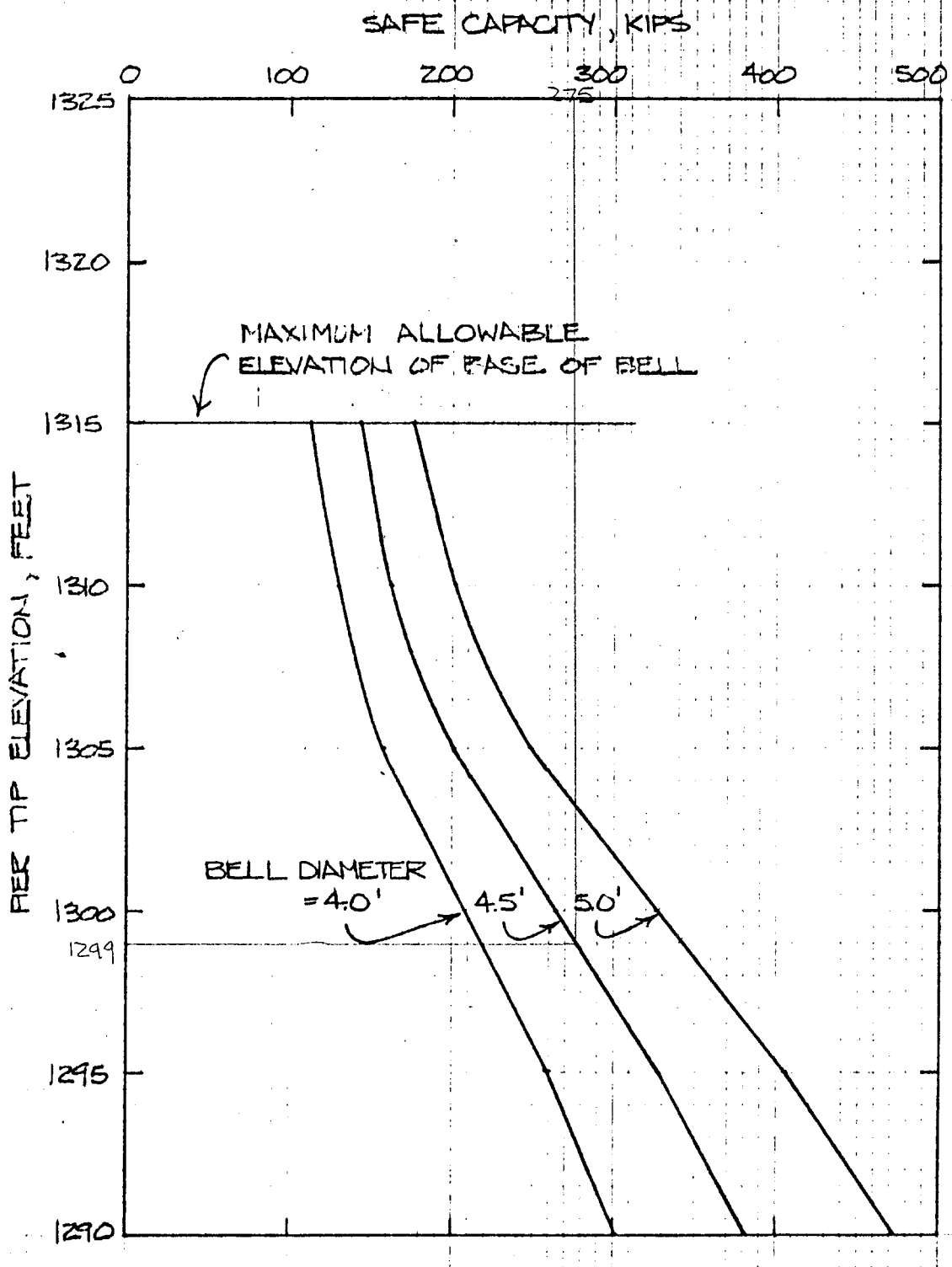
Total Length Ft.	Span Length "L"	Max. Reaction Kips.			Max. Shear Kips.			Max. Moment Kip.-ft.				Impact					Dist.-Ft.	
		at A	at B	at C	in AB at B	in BC at B	in BC at C	in AB at X	at B	in BC at X'	at C	I	II	III	IV	V	X	X'
120	27.3	44.5	61.8	61.9	-50.2	51.7	-51.6	200.8	-182.7	200.6	-187.1	.300	.300	.300	.270	.263	9.7	17.9
140	31.8	47.7	64.2	64.3	-54.1	54.9	-54.8	253.8	-216.6	256.3	-221.6	.300	.300	.300	.256	.248	11.5	20.6
160	36.4	50.3	65.9	65.9	-56.9	57.4	-57.3	308.4	-251.1	314.7	-261.5	.300	.300	.296	.244	.236	13.9	23.4
180	40.9	52.5	67.1	67.1	-59.1	59.3	-59.2	372.8	-299.6	374.9	-312.2	.300	.294	.287	.233	.224	16.6	26.1
200	45.5	54.3	68.0	67.9	-60.7	60.8	-60.7	437.6	-351.8	436.5	-367.0	.293	.286	.278	.222	.214	18.4	28.8
220	50.0	55.8	68.6	69.5	-62.0	62.0	-61.9	503.0	-407.8	498.9	-425.7	.286	.278	.270	.213	.204	20.3	31.5
240	54.5	57.1	72.3	73.4	-63.0	63.0	-62.9	568.9	-467.6	562.2	-488.5	.278	.270	.263	.204	.195	22.3	34.3
260	59.1	58.2	76.2	77.4	-63.9	63.9	-63.8	635.1	-531.2	625.9	-555.2	.272	.263	.255	.196	.187	24.2	37.0
280	63.6	59.1	80.0	81.3	-64.6	64.6	-64.5	701.6	-598.6	690.1	-626.0	.265	.256	.248	.189	.180	26.2	39.7
300	68.2	59.9	83.9	85.3	-65.2	65.1	-65.1	768.3	-669.8	754.6	-700.8	.259	.250	.242	.182	.173	28.0	42.5
320	72.7	60.7	87.8	89.2	-65.7	65.7	-65.6	835.2	-744.8	819.4	-779.6	.253	.244	.236	.175	.167	30.0	45.2
340	77.3	61.3	91.6	93.2	-66.2	66.1	-66.0	902.3	-823.5	884.5	-862.4	.247	.238	.230	.169	.161	31.9	48.0
360	81.8	61.9	95.5	97.1	-66.6	66.5	-66.4	969.4	-906.1	949.8	-949.2	.242	.233	.224	.164	.156	33.9	50.8
380	86.4	62.4	99.3	101.1	-66.9	66.8	-66.8	1036.7	-992.4	1015.2	-1040.0	.237	.227	.219	.159	.150	35.9	53.5
400	90.9	62.9	103.2	105.0	-67.2	67.1	-67.1	1104.0	-1082.5	1080.7	-1134.8	.232	.222	.214	.154	.146	37.9	56.2
420	95.5	63.3	107.1	109.0	-67.5	67.8	-67.5	1171.5	-1176.5	1146.4	-1233.7	.227	.217	.209	.149	.141	39.8	59.0
440	100.0	63.7	110.9	112.9	-67.7	69.8	-69.5	1238.9	-1274.2	1212.2	-1336.5	.222	.213	.204	.145	.137	41.8	61.6
460	104.5	64.1	114.8	116.9	-69.0	71.7	-71.4	1306.5	-1375.7	1278.1	-1443.4	.218	.208	.200	.141	.133	43.7	64.4
480	109.1	64.4	118.6	120.8	-70.9	73.7	-73.4	1374.1	-1481.0	1344.0	-1554.2	.214	.204	.195	.137	.129	45.6	67.1
Impact		I	IV	V	I	III	III	I	II	III	III							
Dead Load		.3774 x wL	1.2259 x wL	1.1934 x wL	-.6226 x wL	.6033 x wL	-.5967 x wL	.0712 x wL^2	-.1226 x wL^2	.0593 x wL^2	-.1187 x wL^2						.3774 x L	.6033 x L

From 'Moments, Shears and Reactions For Continuous Highway Bridges' AISC 1966

Power Rd @ Floodway 83-059

FIGURE 4

SAFE DOWNWARD CAPACITIES FOR BELLED PIERS.



POWER RD @ FIDDLEWOOD 83-58

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

Project GUADALUPE & POWER RD. BRIDGES
Job No: E84-Z
Computed by: JRF Ckd. by:
Date 2/13/84 Page ___ of ___

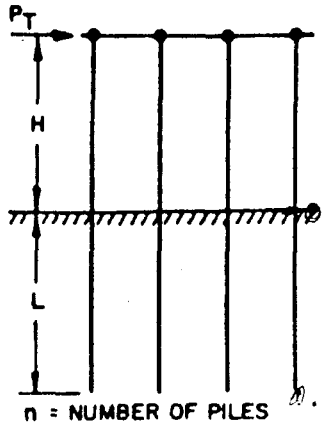
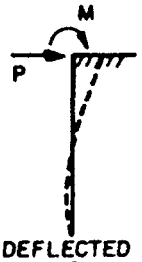
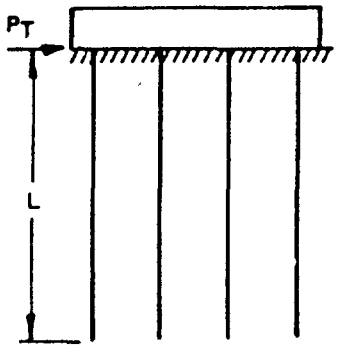
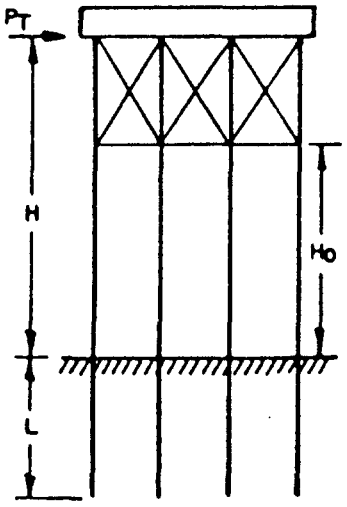
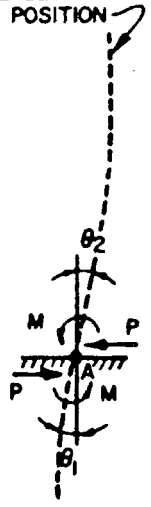
CASE I. FLEXIBLE CAP, ELEVATED POSITION		
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$ 	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.
CASE II. PILES WITH RIGID CAP AT GROUND SURFACE		
		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.
CASE III. RIGID CAP, ELEVATED POSITION		
		1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta (\frac{PT^2}{EI}) + F_\theta (\frac{MT}{EI})$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_0^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

NAVDOCKS DM-7

Power Rd @ Floodway
83-58

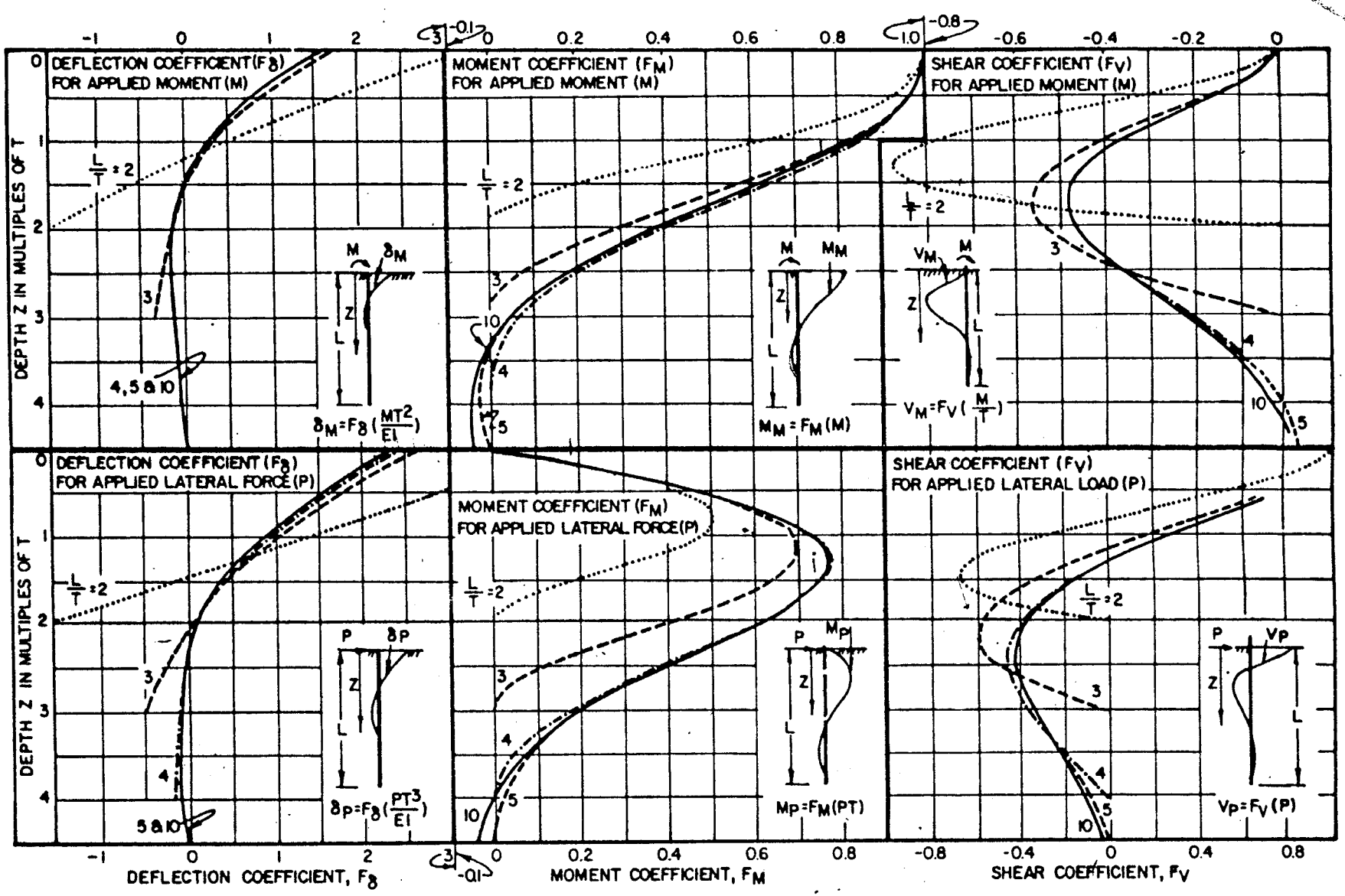


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

NAVDOCKS DM-7
7.2-238

Power Rd @ Floodway
83-53

I BASIC PRINCIPLES OF ANALYSIS

A. To behave as a flexible pile, the mechanism of failure must be such that as the failure load is reached, the moment resistance of the pile section is reached before the soil in front of the pile fails in passive pressure or in bearing

B. Flexible action is determined by the relative flexibility of the pile and the stiffness of the soil. There is no way to predict flexible action without actually analyzing the pile. However the following guidelines may be used.

1. If the pile length exceeds 10 pile diameters - flexible action very likely
2. If "rigidity factors" described below exceed the following limits: (See EG-4 for notation) (L = length of pile in inches, EI = modulus and inertia of pile psi-in^4)

Type of Soil →	Clay	Sand
"Rigidity Factor"	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$	$\eta = \sqrt[5]{\frac{n_h}{EI}}$
Flexible action if:	$\beta L > 2.5$	$\eta L > 2.0$
Example	12"ϕ Steel Pipe Pile 20' long EI = $3 \times 10^7 \times 361.5$	
	k_h = 80 pci (stiff clay)	n_h = 10 pci
	$\beta = \sqrt[4]{\frac{80 \times 12}{4 \times 108.45 \times 10^8}} = 0.0120$	$\eta = \sqrt[5]{\frac{10}{3 \times 10^7 \times 361.5}} = 0.0156$
	$\beta L = 0.0120 \times 240 = 2.88 > 2.5$ Flexible Action	$\eta L = 0.0156 \times 240 = 3.74 > 2.0$ Flexible Action

(A convenient table of roots is available in: Abbett - American Civil Eng'g Practice 1956 Vol 1 p 1-

II ULTIMATE ANALYSIS OF FLEXIBLE PILES

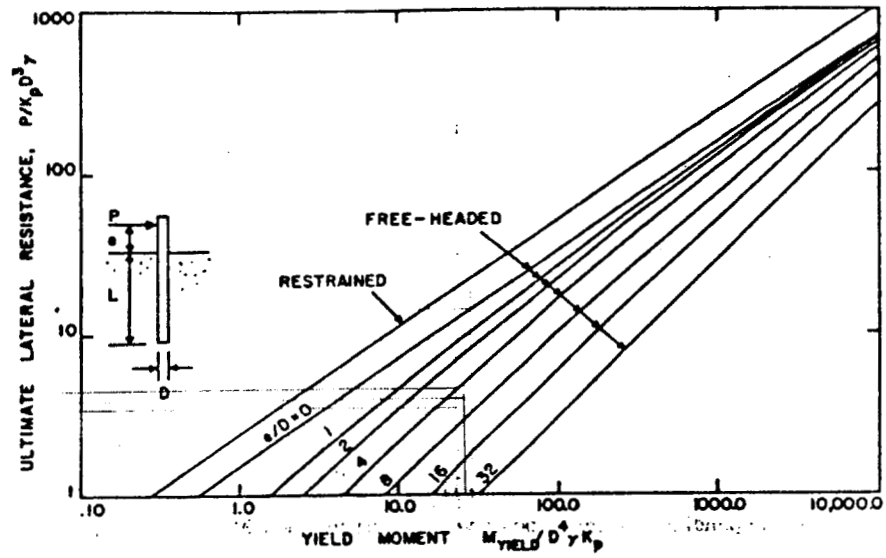
A. Analysis consists of computing the maximum bending moment producing failure in the pile section. Then the ultimate lateral load which produces the failure moment corresponds to the ultimate capacity of the pile. The margin of safety for a particular load is the ratio of the bending moment produced by the load to the resisting moment of the pile section.

B. The bending resistance of piles can be computed by structural analysis methods using elastic or plastic (ultimate) analyses methods. If the pile wall is thin, buckling may become critical

C. If the lateral pressure applied by the pile to the soil exceeds the passive resistance or the bearing capacity of the soil before the full bending resistance is reached, the use of "flexible pile analysis is invalid.

Broms Method

E. Design charts for Ultimate Analysis of Flexible Piles: (from Ref 5) These charts are based on the equations developed in Ref 3 and 4



-ULTIMATE LATERAL RESISTANCE FOR COHESIONLESS SOILS RELATED TO YIELD MOMENT

Example:

12"ϕ Pipe Pile 20 long
What is max P? $e = 0$
Soil = sand $\phi = 30°$ $K_p = 3$
$K_p = 3$ $\gamma = 100$ pcf

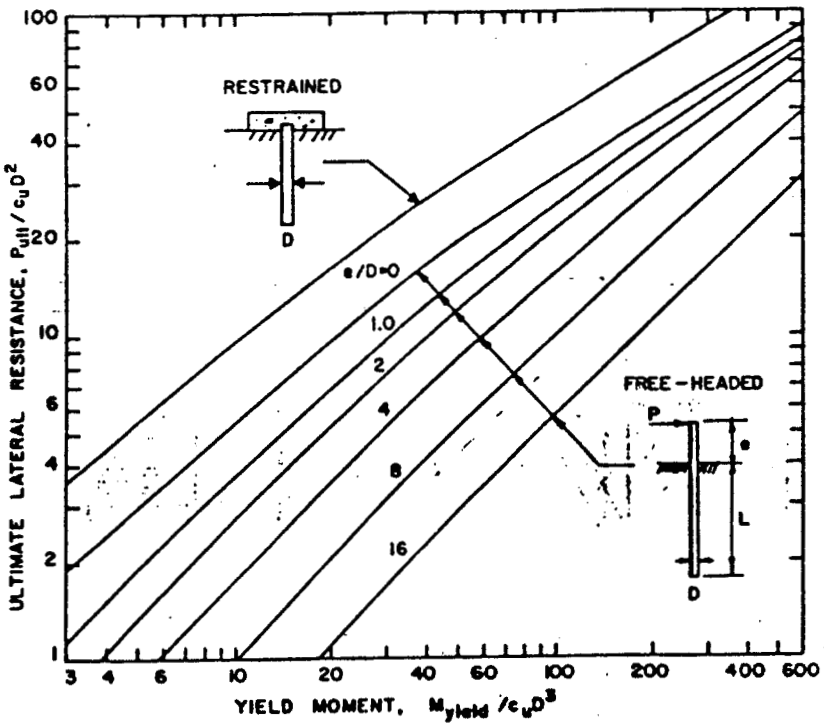
$$M_{resisting} = \frac{fI}{c} = \frac{20^{*} \times 361.5}{6_{in} \times 12^{in/ft}} = 100 \text{ ft-kips}$$

$$\frac{M_r}{D^4 \cdot \gamma \cdot K_p} = \frac{100}{1^4 \times 0.1 \times 3} = 333.3$$

$$\frac{P}{K_p D^3 \gamma} = 80 \text{ (from chart)}$$

$$P = 80 \times 3 \times 1 \times 0.1 = \underline{24 \text{ kips}}$$

* allowable yield stress of steel in Ksi



-ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO YIELD MOMENT

Example:

Same pile as before $c_u = 750$ psf

$$\frac{M_r}{c_u D^3} = \frac{100}{0.75 \times 1} = 133.3$$

$$\frac{P_{ult}}{c_u D^2} = 35$$

$$P_{ult} = 35 \times 0.75 \times 1^2 = \underline{26.25 \text{ kips}}$$

Broms Method

Power Rd @ Floodway
83-58

PILES: LATERALLY LOADED
Analysis of Rigid Piles

I BASIC PRINCIPLES OF ANALYSIS

A. To behave as a rigid pile, the mechanism of failure must be such that as the failure load is reached, the ultimate bearing capacity or the the passive resistance of the soil is reached before the pile fails in bending.

B. Rigid action is determined by the relative flexibility of the pile and the stiffness of the soil. There is no way to predict rigid action without actually analyzing the pile. However the following guidelines may be used:

1. If pile length is less than 10 pile diameters – rigid action very likely
2. If "rigidity factors" described below are smaller than indicated limits: (See EG-4 for notation) (L = pile length in inches) (E & I Modulus and Inertia of pile – psi and in⁴)

MAKE SURE YOU HAVE CONSISTENT UNITS

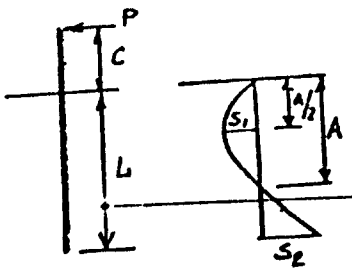
Type of Soil →	Clay	Sand
"Rigidity Factor	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$ $\frac{\#}{in^3}$ in / $\frac{\#}{in^2} \times in^4$ = $\sqrt[4]{\frac{1}{in^4}}$ (PILE VALUE)	$\eta = \sqrt[5]{\frac{n_h}{EI}}$ – PILE PROPERTIES
"Rigid action" if	$\beta L < 2.25$ (Ref 3)	$\eta L < 2.0$ (Ref 4)
Example	$k_h = 80$ pci (Firm clay) $D = 24"$ $L = 12'$ Concrete $\beta = \sqrt[4]{\frac{80 \times 24}{4 \times 3 \times 10^6 \times 16286}} = 0.010$ $\beta L = 0.010 \times 144 = 1.44 < 2.25$ Rigid Pile action	$n_h = 10$ pci 12" ϕ Steel Pipe Pile $L = 10'$ $\eta = \sqrt[5]{\frac{10}{3 \times 10^7 \times 361.5}} = 0.01563$ $\eta L = 0.01563 \times 120 = 1.87 < 2.0$ Rigid Pile Action

(A convenient table of roots is available in: Abbett – American Civil Eng'g Practice 1966 Vol I p 11-27

II ULTIMATE ANALYSIS OF RIGID PILES

A. Analysis consists of computing the pressure applied by the pile to the soil. Near the surface of the ground (D = 2–3 diameters) a passive failure will occur. Deeper, a bearing failure is expected. The margin of safety available is the ratio of the available resistance to the applied pressure

B. Pressure applied by the pile usually computed by "flagpole formulas"



$$A = \frac{L(3L + 4C)}{4L + 6C}$$

$$S_1 = \frac{1.5P(3L + 4C)^2}{L^2(4L + 6C)}$$ (usually governs design)

$$S_2 = \frac{6P}{L^2}(2C + L)$$

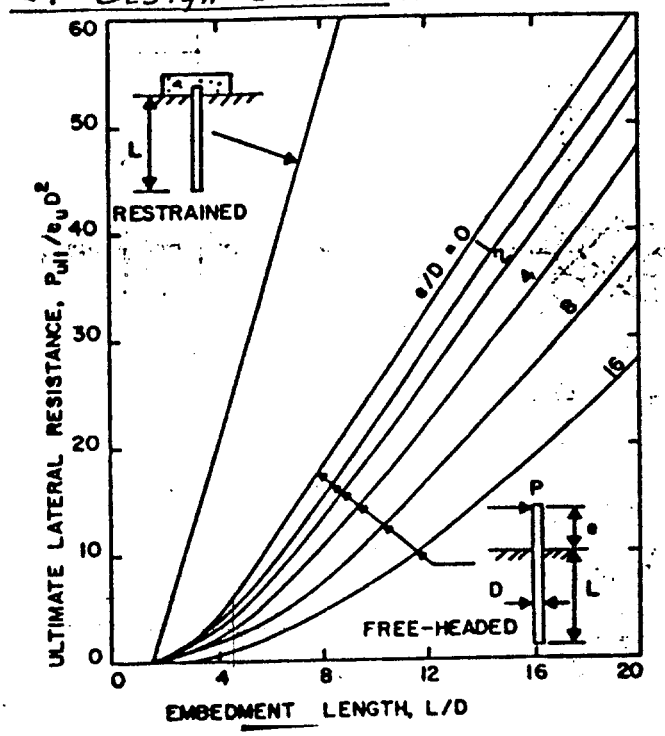
Additional equations for pile design available in Ref.

C. If bending moments developed in pile exceed strength of the pile the use of "rigid" pile analysis is invalid.

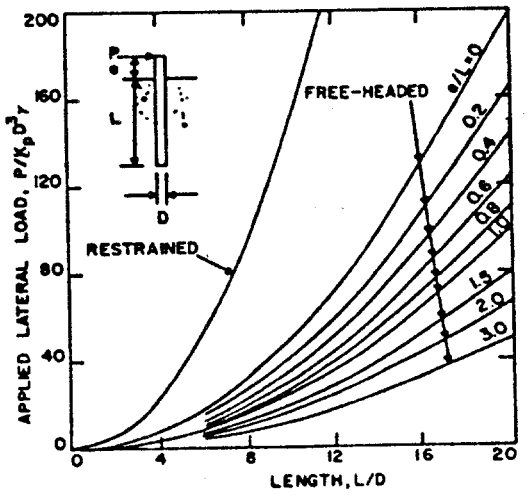
Broms Method

Power Rd @ Floodway 83-58

G. Design Charts for Ultimate Analysis of Rigid Piles: (From Ref 4)



-ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO EMBEDMENT LENGTH



-ULTIMATE LATERAL RESISTANCE FOR COHESIONLESS SOILS RELATED TO EMBEDMENT LENGTH

Example

$c = 750$ psf

$e = 0$ $D = 2'$

$P = 30$ kips

$$\frac{P}{c_u D^2} = \frac{30000}{750 \times 4} = 10$$

$\frac{L}{D} = 5.5$ $L = 11'$ (FS = 1.0)

Example:

$\phi = 30°$, $D = 1'$ $\gamma = 100$ pcf

$e = 0$ $P = 30$ kips.

$$k_p = \tan^2(45 + \phi/2) = 3.0$$

$$\frac{P}{k_p D^3 \gamma} = \frac{30000}{3 \times 1 \times 100} = 100.0$$

$\frac{L}{D} = 14$ $L = 14'$ (FS = 1.0)

III BASIC RELATIONSHIPS

Since a rigid pile fails by applying a pressure to the soil which exceed its passive resistance or its bearing capacity, the following is basic:

SOIL FAILS NOT PILE SO DEEPEN ~~DON'T INCREASE SIZE~~

A. Increasing the length of the rigid pile increases the load it can carry and reduces its lateral deflections

B. Increasing the stiffness of the rigid pile will not increase its lateral load carrying capacity and has only a small effect on its deflections.

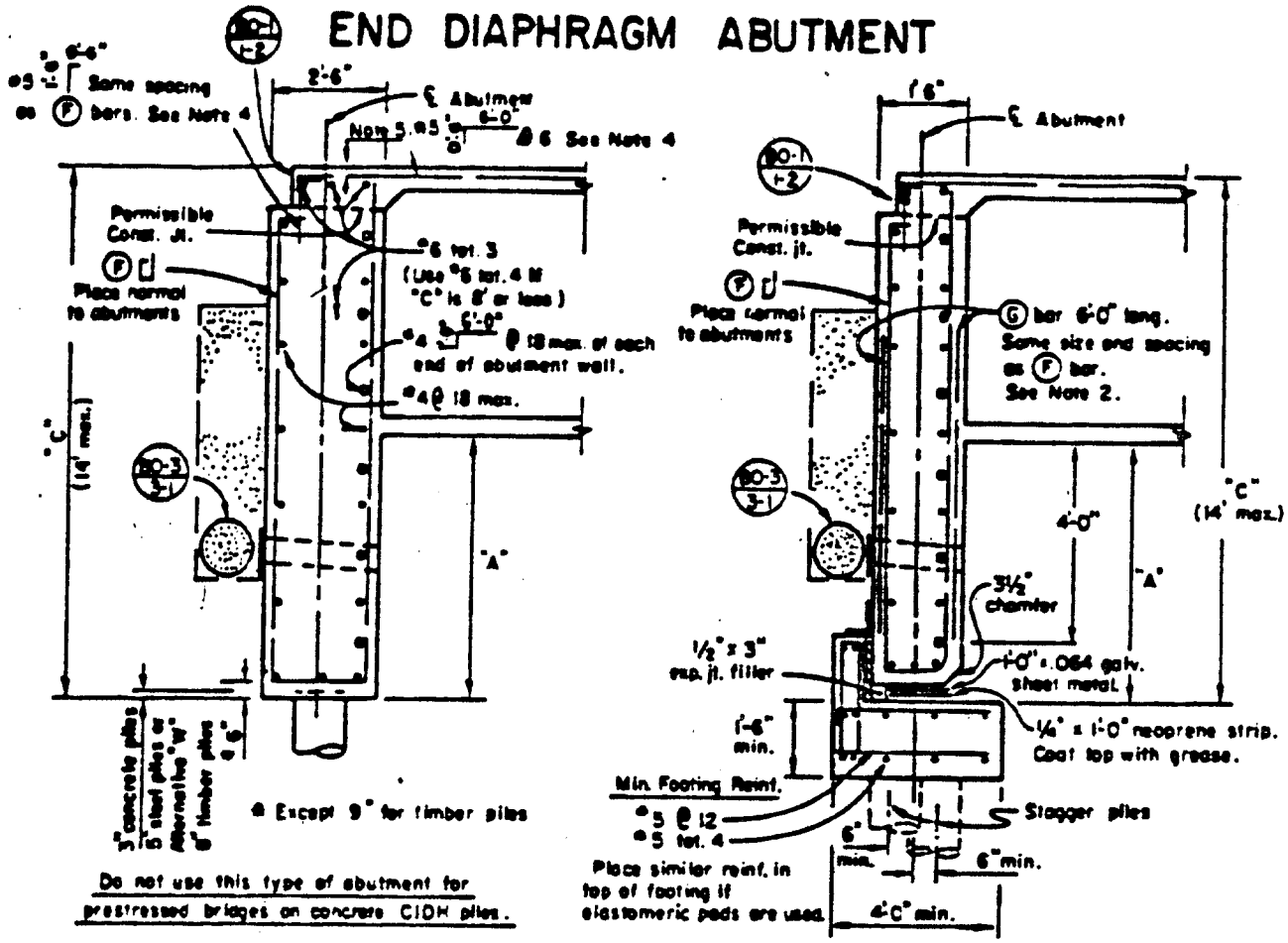
IV ELASTIC ANALYSIS OF RIGID PILES

A. Purpose: to compute deflections of a rigid pile under working loads.

B. Elastic analysis is based on soil stiffness in the elastic range. For this reason it is valid only up to about 1/3 to 1/2 of ultimate load.

C. Soil stiffness values are obtained as described in EG-4

END DIAPHRAGM ABUTMENT



(F) U BARS, #5 When (F) bars > than #8

LONGIT. FORCE IN KIPS / L.F.		1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
"A" in Feet	9	#5 @ 18	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#10 @ 12	#10 @ 12	#11 @ 12	#11 @ 12
	8	#5 @ 18	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12	#11 @ 12	#11 @ 12
	7	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12	#10 @ 12
	6	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#8 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12
	5	#5 @ 18	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#7 @ 12	#8 @ 12	#8 @ 12	#9 @ 12
	4	#5 @ 18	#5 @ 18	#5 @ 12	#6 @ 12	#6 @ 12	#7 @ 12	#7 @ 12	#8 @ 12	#8 @ 12

NOTES

1. Designer will determine longitudinal force based upon Memo to Designers 5-2. To determihe longitudinal force in kips/ft, divide total force by effective abutment width. Effective abutment width = total width less widths of utility openings.
2. Omit G bar when diaphragm width at girder soffit is 2'-6" or greater. Between face of exterior girder and edge of deck extend the G bar upward into the deck slab.
3. When bridge is to be post-tensioned, call for placing of wingwall concrete to be done after stressing is completed.
4. See page 8-32 for requirements on skews.
5. Specify clearance to allow for deck slab reinforcement.
6. Specify keys similar to that shown on page 6-23.

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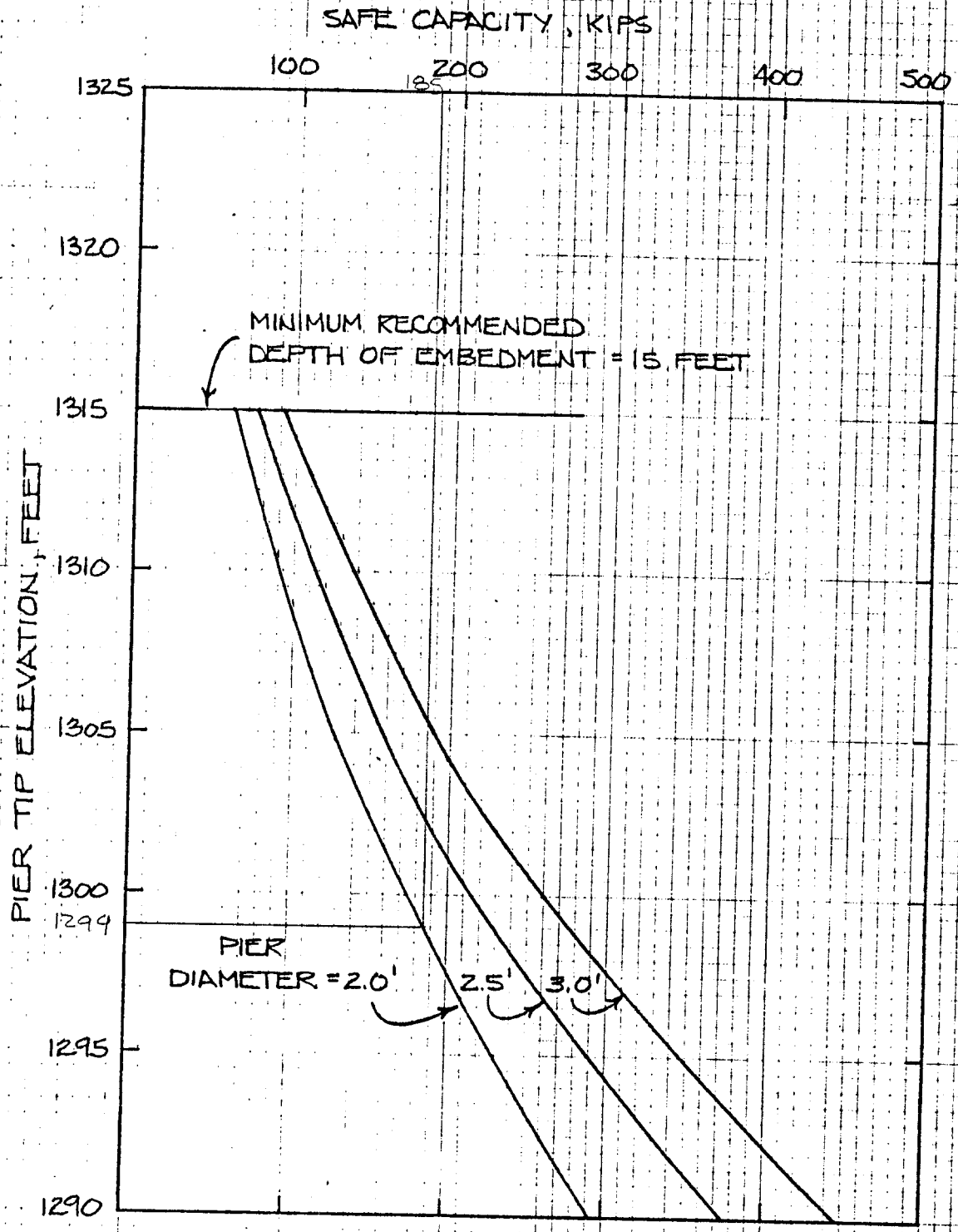
FIGURE 1

SAFE DOWNWARD CAPACITIES
FOR STRAIGHT PIERS.
- ABUTMENT PIERS -



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