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Structural Calculations

GUADALUPE 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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Tucson, Arizona 85701
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STRUCTURAL CALCULATIONS GUADALUPE ROAD BRIDGE OVER RWCD FLOODWAY

A121.805

Structural Calculations
GUADALUPE 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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Tucson, Arizona

Basis for Design

Ref.- 1) "Uniform Standard Specifications for Public Works Construction," MAG, 1979

2) "Scope of Work" by Maricopa Co. Highway Dept. Rec'd Feb 24, 1984

3) "Std. Spec. For Highway Bridges" by AASHTO, 1977, Updated

4) Technical Advisories - Referenced when used

5) Navdocks DM-7

Loading:

A) Design to Accomodate HS20-44 Loading

B) Calculations based on Service Loads & the working stress Method

C) Thermal Range: Rise : 30°F (per AASHTO)
Fall : 40°F
Mean : 70°F

D) Seismic Zone: AASHTO Risk Zone 2

E) Stream Flow: V = 7.0 ft/s at d = 7.2', Q = 4900 cfs

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

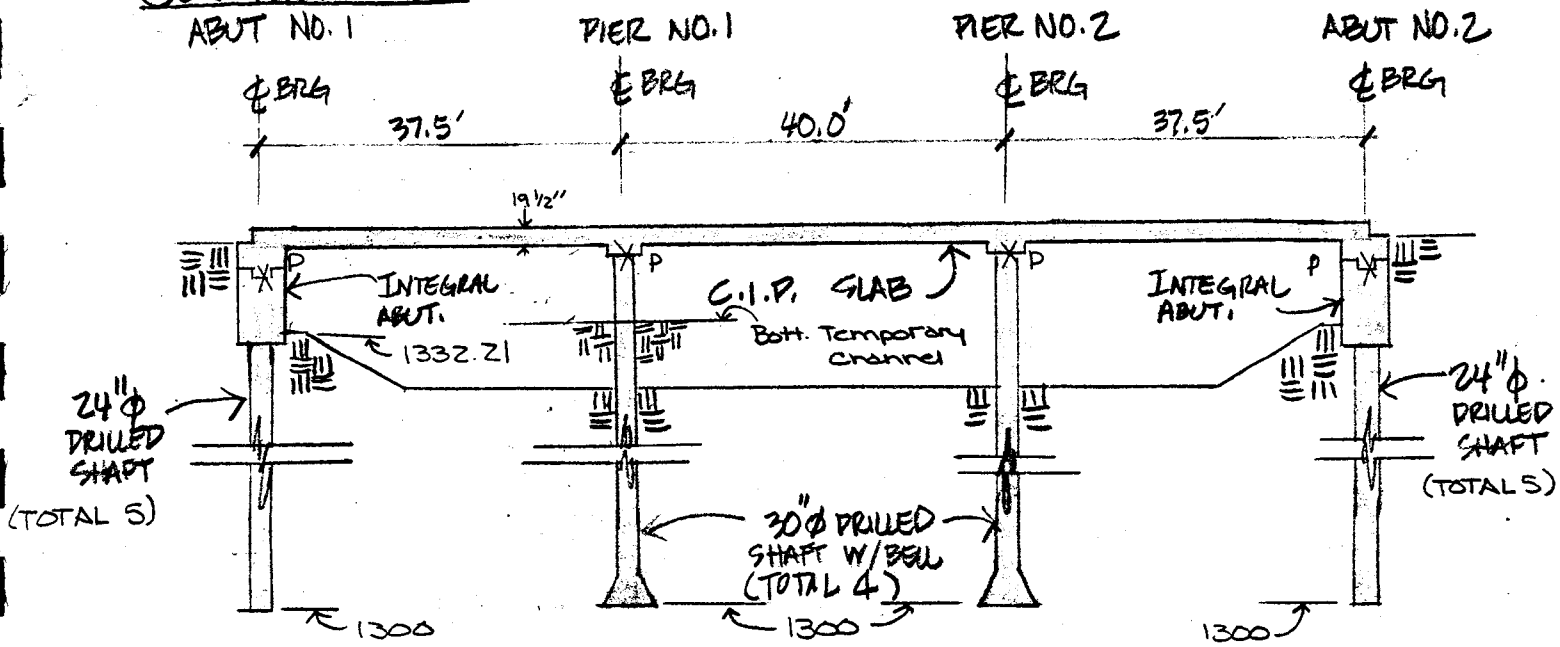
B.) Steel

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

Primary Assumptions

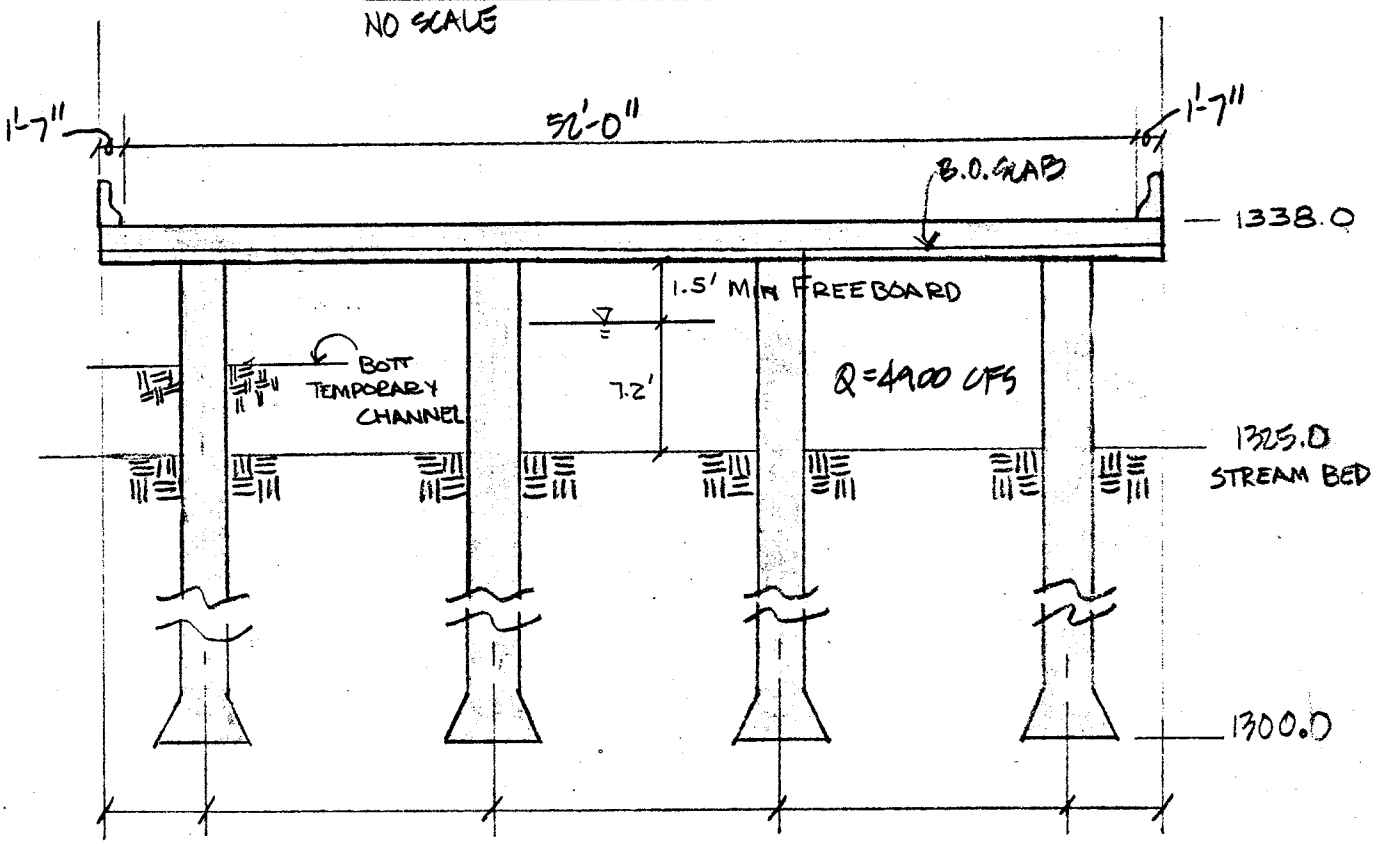
1) Pier columns are pinned to girders - slab Continuous
2) Integral Abutment - Abutment cap is pinned to caissons
3) Pier Design configuration shown on following page
4) Scour Bed Elev. @ 1315.0 -
   - Allows for 10' scour @ Piers

CONFIGURATION



ELEVATION THRU ℄ CONST.

NO SCALE



PIER ELEVATION

NO SCALE

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DECK DESIGN

Deck Design

Slab Design

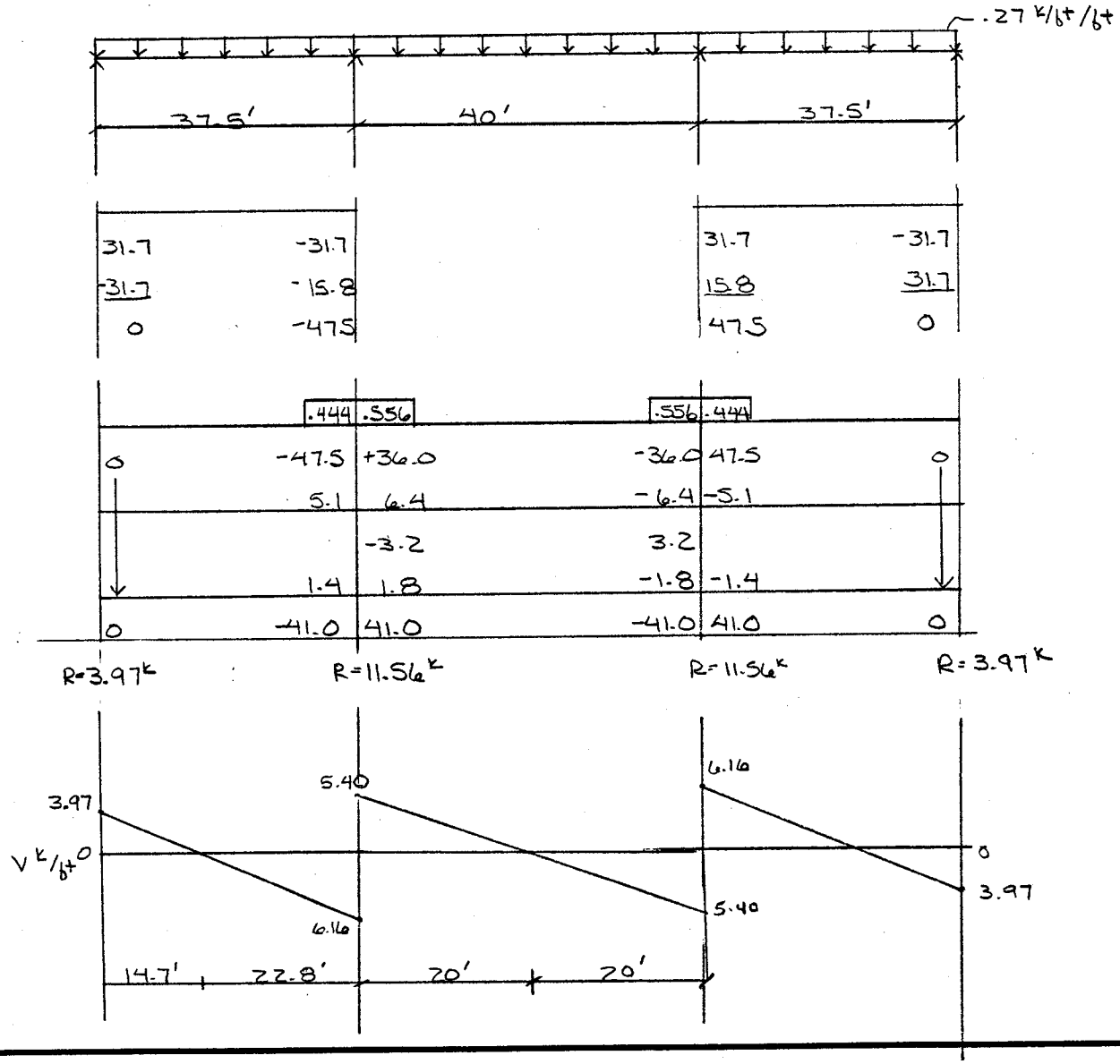
Dead Load: Wearing Surface = 0.025 ksf

Slab: $(19.5/12)(.150)$ = 0.244 ksf

0.269 ksf

Note: Barriers Supported by edge beam

Assume superstructure is pinned to substructure, typ.



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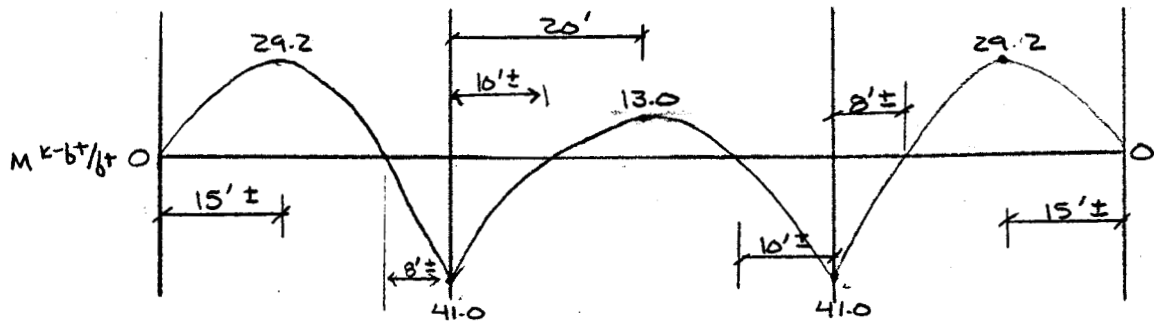
date 10/5/84 sh. 5 of

Deck Design

Dead Load Moment Diag:

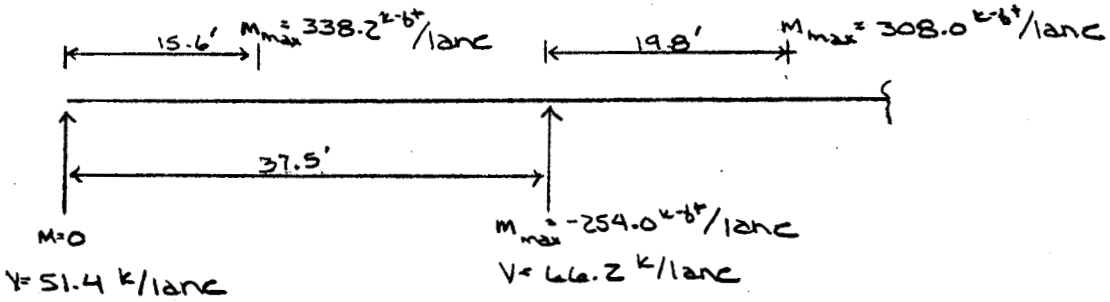


Live Load Moments

From 'Moments, Shears & Reactions For Continuous Highway Bridges'

$NL/L = 40/37.5 = 1.07$

Using Total Length = 120' & N = 1.1 (Table 3.1) - See Appendix



Impact Coefficient = 0.3

Distribution of Lane Loads: (AASHTO 1.3.2 (c))

$2E = 2(4+0.06(37.5)) = 12.5'$ (37.5' Span)

$= 2(4+0.06(40)) = 12.8'$ (40' Span)

Deck Design

DL+LL+I (Moments / ft width)

① Exterior Spans:

$DL = 29.2^{k\text{-}ft}$

$LL+I = 1.3(338.2/12.5) = 35.2^{k\text{-}ft}/ft$

$D+L+I = 64.4^{k\text{-}ft}/ft$

② @ Pier Supports:

$DL = -41.0^{k\text{-}ft}$

$LL+I = 1.3(-254.0/12.5) = -26.4^{k\text{-}ft}/ft$

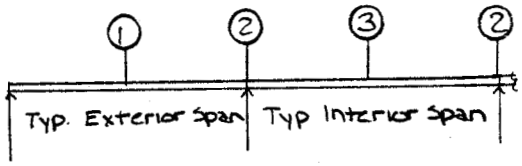
$D+L+I = -67.4^{k\text{-}ft}/ft$

③ Interior Spans

$DL = 13.0^{k\text{-}ft}$

$LL+I = 1.3(308/12.8) = 31.3^{k\text{-}ft}/ft$

$D+L+I = 44.3^{k\text{-}ft}$



① Exterior Span - Pos. Reinforcement

Try (#9 & #11)/LF $\quad A_s/LF = (1.00+1.56) = 2.56\ in^2$

$d = 19\tfrac{1}{2} - 1 - 1.41/2 = 17.8''$ Use $d = 17''$ $\quad n = \frac{29000}{57\sqrt{4000}} = 8$

$n\frac{A_s}{bd} = 8\frac{2.56}{(12)(17)} = 0.100 \quad k = 0.358\ ;\ j = 0.880$

Req'd $d = \sqrt{\frac{64.4(12)}{1/2(1.6)(.358)(.880)(12)}} = 16'' < 17''$ ∴ O.K.

$f_s = \frac{M}{A_s j d} = \frac{64.4(12)}{2.56(.88)(17)} = 20.2\ ksi < 24.0\ ksi$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.358(20.2)}{8(1-.358)} = 1.41\ ksi < 1.60\ ksi$

∴ O.K. Use #9 & #11 /LF

Deck Design

② Interior Supports - Neg. Reinforcement

$M = 67.4$ k-ft Try #9 & #11 /LF $A_s = (1.00 + 1.56) = 2.56$ in²/LF

$d = 19\frac{1}{2} - 2 - 1.41/2 = 16.8''$ Use $d = 16\frac{1}{2}''$ $n = 8$

$n \frac{A_s}{bd} = 8 \frac{2.56}{(12)(16.5)} = 0.103$ $k = 0.362$; $j = 0.879$

Req'd $d = \sqrt{\frac{67.4(12)}{\frac{1}{2}(1.6)(.362)(.879)(12)}} = 16.3'' < 16.5''$ ∴ O.K.

$f_s = \frac{67.4(12)}{(2.56)(.879)(16.5)} = 21.8$ ksi < 24.0 ksi

$f_c = \frac{0.362(21.8)}{8(1-.362)} = 1.55$ ksi < 1.60 ksi

∴ O.K. Use #9 & #11 /LF

③ Interior Span - Pos. Reinforcement

$M = 44.3$ k-ft Try #9 @ 6" O.C. $A_s = 2.00$ in²/LF ; $d = 17''$; $n = 8$

$n \frac{A_s}{bd} = 8 \frac{2.00}{(12)(17)} = 0.0784$ $k = 0.325$; $j = 0.892$

$f_s = \frac{44.3(12)}{(2.00)(.892)(17)} = 17.5$ ksi < 24.0 ksi

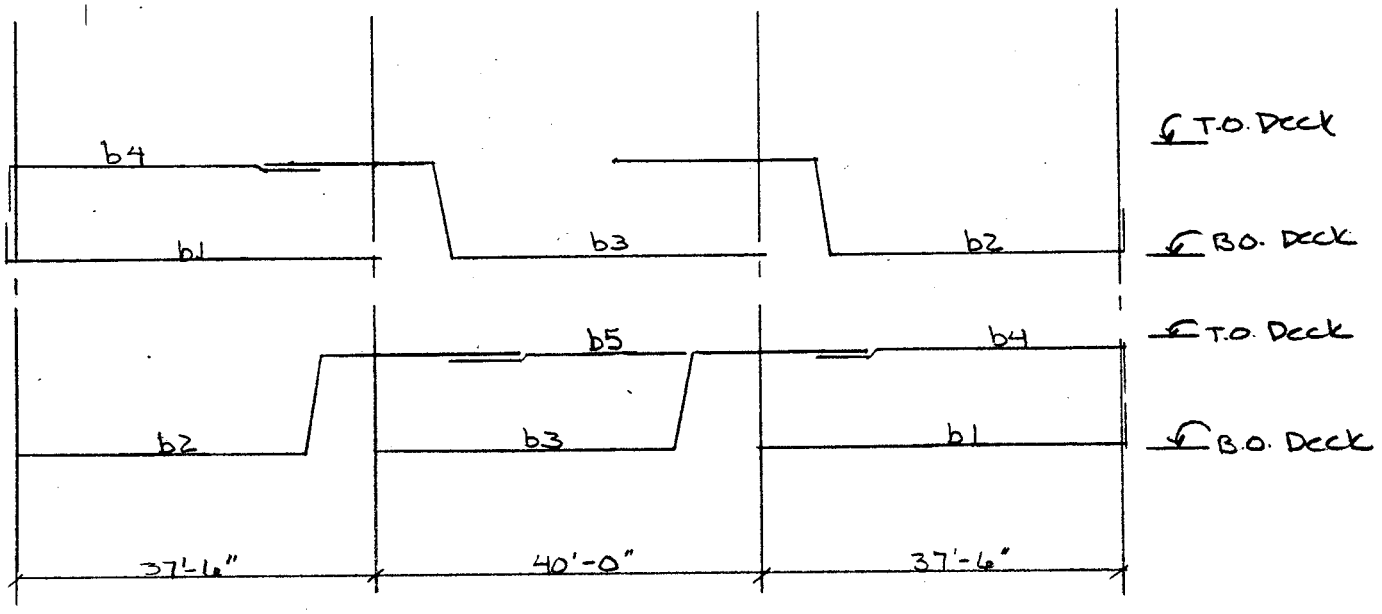
$f_c = \frac{0.325(17.5)}{8(1-0.325)} = 1.05$ ksi < 1.60 ksi

∴ O.K. Use #9 @ 6" O.C.

Deck Design

Calculation of bar cut-offs & bends

REBAR PLACING DIAGRAM:



bar	size	spacing
b1	#9	12"
b2	#11	12"
b3	#9	6"
b4	#6	12"
b5	#6	12"

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

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

Top Reinf.: Req'd $A_s/ft = 0.002(17")(12") = 0.41 \text{ in}^2/\text{LF}$

use #6 @ 12" o.c. (b4 & b5)

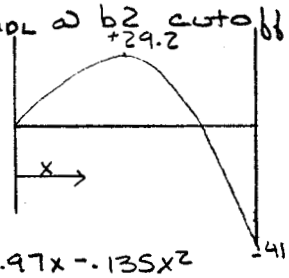
Deck Design

Bar b2

Pos. Reinf: Assume LL Mom. Distr is the same as DL Mom Distr.

Max M_{DL} @ b2 cutoff $= \left(\frac{1.00}{2.56}\right)(29.2)\left(\frac{24.0}{20.2}\right) = 13.5^{k\text{-}ft}$

M_{DL}:



$M_{DL} = 3.97x - .135x^2$

$M_{DL} = 13.5^{k\text{-}ft}$ @ $x = \dfrac{3.97 + \sqrt{(3.97)^2 - 4(.135)(13.5)}}{2(.135)} = 25.5'$

Bar Bend @ $25.5' + {}^{37.5'}/_{20} = 27.4'$ min from ℄ Abut

Neg. Reinf: Assume LL Mom Distr is the same as DL Mom Distr.

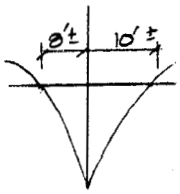
Max M_{DL} @ b2 cutoff bend $= \left(\frac{1.00}{2.56}\right)(-41)\left(\frac{24.0}{21.8}\right) = -17.6^{k\text{-}ft}$

$M_D = 3.97x - .135x^2 = -17.6$ @ $x = \dfrac{3.97 \pm \sqrt{(3.97)^2 + 4(.135)(17.6)}}{2(.135)} = 33.3'$

Bar Bend @ $37.5' - 33.3' + {}^{37.5'}/_{20} = 6.1'$ min from ℄ Pier

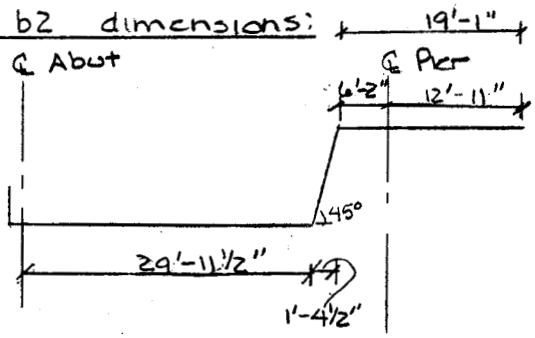
AASHTO 1.5.13(C)

bar b1 must extend $\ell/16$ beyond inflection pt



$10' + {}^{40}/_{16} = 12.5'$

Bar b2 dimensions:



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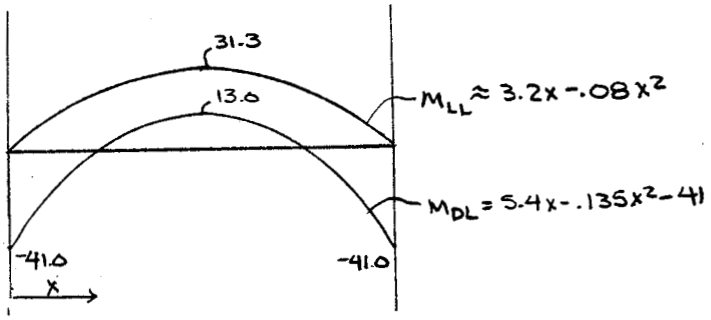
Deck Design

Bar b3 -

Pos. Reinf. - Max M_{D+L+I} @ b3 cutoff $= \frac{1}{2}(44.3)\left(\frac{24}{17.5}\right) = 30.3$ k-ft

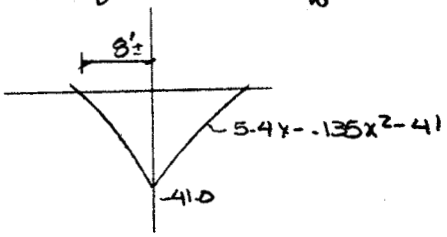


$M_{D+L+I} = 8.6x - .215x^2 - 41 = 30.3$ @ $x = \dfrac{8.6 \pm \sqrt{(8.6)^2 - 4(.215)(71.3)}}{2(.215)} = 11.7', 28.3'$

b3 cutoff @ $28.3' + 40'/20 = 30.3'$ Min.

Neg. Reinf. Assume LL Mom Distr = DL Mom Distr.

Max M_D @ b3 cutoff $= \left(\frac{1.56}{2.56}\right)(-41.0)\left(\frac{24}{21.8}\right) = -27.5$ k-ft

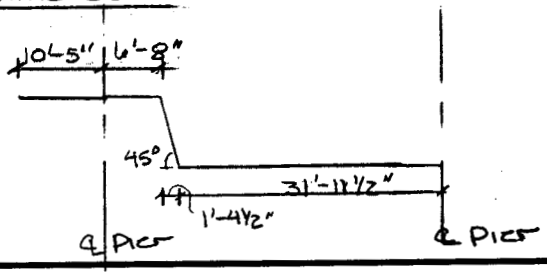


$M_{DL} = 5.4x - .135x^2 - 41 = -27.5$ @ $x = \dfrac{5.4 \pm \sqrt{(5.4)^2 - 4(.135)(13.5)}}{2(.135)} = 2.7'; 37.3'$

Bar bend @ $2.7' + 40/20 = 4.7'$ min from ℄ Pier

Extension beyond inflection pt. $= 8' + 37.5'/16 = 10.3'$

Bar b3 Dimensions



Deck Design

DL Camber - Calculation of EI

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

AASHTO (1.5.23(G))

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

Neglecting top ½" of slab: $I_g = 12(19)^3/12 = 6859 \text{ in}^4$

$M_{cr} = f_r I_g / y_t = 7.5\sqrt{4000}(6859)/9.5 = 342{,}500$ #-in $= 28.5$ k-ft

$I_{cr} = \frac{bc^3}{3} + nA_s(d-c)^2$ where $c = kd$; $n = 8$

① $M_a = M_{D+L+I} = 64.4$ k-ft/ft ; $A_s = 2.56 \text{ in}^2/\text{ft}$

$k = 0.358$; $d = 17''$ & $c = 0.358(17) = 6.1''$ $I_{cr} = \frac{12(6.1)^3}{3} + 8(2.56)(17-6.1)^2 = 3341 \text{ in}^4$

$$I_e = \left(\frac{28.5}{64.4}\right)^3 6859 + \left[1 - \left(\frac{28.5}{64.4}\right)^3\right] 3341 = 3646 \text{ in}^4$$

② $M_a = 67.4$ k-ft/ft ; $A_s = 2.56 \text{ in}^2/\text{ft}$; $k = 0.362$, $d = 16\frac{1}{2}''$ $kd = .362(16.5) = 6.0''$

$I_{cr} = \frac{12(6)^3}{3} + 8(2.56)(16.5-6)^2 = 3122 \text{ in}^4$

$$I_e = \left(\frac{28.5}{67.4}\right)^3 6859 + \left[1 - \left(\frac{28.5}{67.4}\right)^3\right] 3122 = 3405 \text{ in}^4$$

③ $M_a = 44.3$ k-ft/ft ; $A_s = 2.00 \text{ in}^2/\text{ft}$; $k = 0.325$; $d = 17''$ $c = kd = .325(17) = 5.5''$

$I_{cr} = \frac{12(5.5)^3}{3} + 8(2.00)(17-5.5)^2 = 2782 \text{ in}^4$

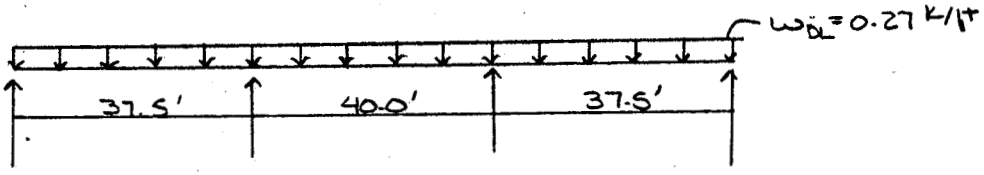
$$I_e = \left(\frac{28.5}{44.3}\right)^3 6859 + \left[1 - \left(\frac{28.5}{44.3}\right)^3\right] 2782 = 3868 \text{ in}^4$$

Exterior Span use $I_e = \frac{1}{2}(3646 + 3405) = 3526 \text{ in}^4$
Interior Span use $I_e = \frac{1}{2}(3405 + 3868) = 3637 \text{ in}^4$

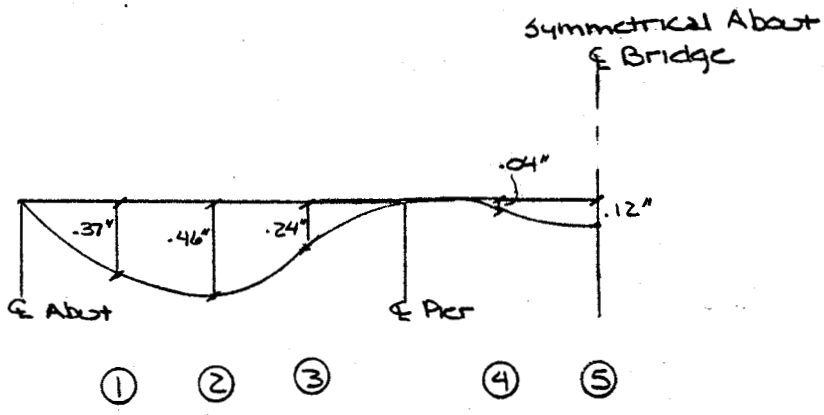
Deck Design

DL Camber (cont.)

Dead Loading



Resulting Initial DL Deflections: (@ Span quarter points)



Long Term Deflections

AASHTO 1.5.23 (G)

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

Point	A_s'	A_s	Factor $[2-1.2(A_s'/A_s)]$	$\Delta_{DL}(1+\text{Factor})$	Use Camber
1	0.44	2.56	1.79	1.03"	1"
2	0.44	2.56	1.79	1.28"	1 1/4"
3	1.00	2.56	1.53	0.61"	5/8"
4	1.56	2.00	1.06	0.08"	1/8"
5	0.44	2.00	1.74	0.33"	5/16"

Deck Design

Live Load Deflection:

M_{max} LL+I = 35.2 k-ft/ft @ M_{D+L+I} = 64.4 k-ft/ft

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

Neglect top 1/2" Wear Surface $t = 19''$

For 1' of Slab: $I_g = 12(19)^3/12 = 6859 \text{ in}^4/\text{ft}$

$y_T = 19/2 = 9.5''$; $f_r = 7.5\sqrt{4000} = 474$ psi

$M_{cr} = f_r I_g / y_T = 474(6859)/9.5 = 342228$ *-in = 28.5 k-ft/ft

$$I_{cr} = \frac{bc^3}{3} + n A_s (d-c)^2 \qquad n = 8 \quad A_s = 2.56 \text{ in}^2/\text{ft} \quad c = kd = .358(17) = 6.09''$$

$$= \frac{12(6.09)^3}{3} + 8(2.56)(17-6.09)^2 = 3341 \text{ in}^4/\text{ft}$$

$$I_e = \left(\frac{28.5}{64.4}\right)^3 6859 + \left(1-\left(\frac{28.5}{64.4}\right)^3\right) 3341 = 3646 \text{ in}^4$$

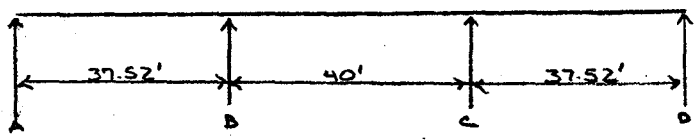
$$\Delta_{LL} = \frac{M\ell^2}{14.5EI} = \frac{35.2(40)^2(12)^3}{14.5(3605)(3646)} = 0.51''$$

AASHTO (1.7.6) $\ell/800 = 40(12)/800 = 0.60'' > 0.51''$

∴ Deflection OK

Deck Design
Edge Beam Design

width of beam = 1'-0" depth = $19\frac{1}{2}'' + 10'' = 29.5''$



From Deck calculations:

DL Moments due to slab:

at ≈0.4 AB $M = (29.2\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = 26.5\ k\text{-}ft$

at B $M = (-41.0\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = -37.2\ k\text{-}ft$

at ≈0.5 BC $M = (13.0\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = 11.8\ k\text{-}ft$

DL Moments due to barrier:

barrier = 0.4 k/ft at ≈0.4 AB $M = \left(\frac{0.4}{0.27}\right)(29.2) = 43.2\ k\text{-}ft$

at B $M = \left(\frac{0.4}{0.27}\right)(-41.0) = -60.7\ k\text{-}ft$

at 0.5 BC $M = \left(\frac{0.4}{0.27}\right)(13.0) = 19.2\ k\text{-}ft$

Total D.L Moments: at 0.4AD $M_D = 69.7\ k\text{-}ft$

at B $M_D = -97.9\ k\text{-}ft$

at 0.5BC $M_D = 31.0\ k\text{-}ft$

Deck Design

Live Load moment of 0.10 PS P = 16^k

Influence Coefficients from 'Moments, Shears & Reactions - - -'

$^{NL}/_L = {}^{40}/37.52 = 1.07'$ Use Tables A3.0 & A3.1 - See Appendix

Max + mom @ 0.4 AB:

Table A 3.0 - Load @ 0.4AB Coeff = 0.2042

Table A 3.1 - Load @ 0.4AB Coeff = 0.2056

Coeff = 0.2042 + .7(.2056 - .2042) = .2052

$M_{0.4AB}$ = .2052(16)(37.52) = 123.2 $^{k-ft}$

0.1(123.2 $^{k-ft}$) = 12.3 $^{k-ft}$

Max - mom @ B

Table A 3.0 - Load @ 0.6 AB Coeff = -.1024

Table A 3.1 - Load @ 0.6 AB Coeff = -.0982

Coeff = -.0995

M = -.0995(16)(37.52) = -59.73

0.1(59.73) = -6.0 $^{k-ft}$

Max + mom @ 0.5 BC

Table A3.0 Load @ 0.5 BC Coeff = 0.1750

Table A3.1 Load @ 0.5 BC Coeff = 0.1894

Coeff = 0.1851

M = .1851(16)(40') = 118 $^{k-ft}$

0.1(118) = 11.8 $^{k-ft}$

Total Moments:

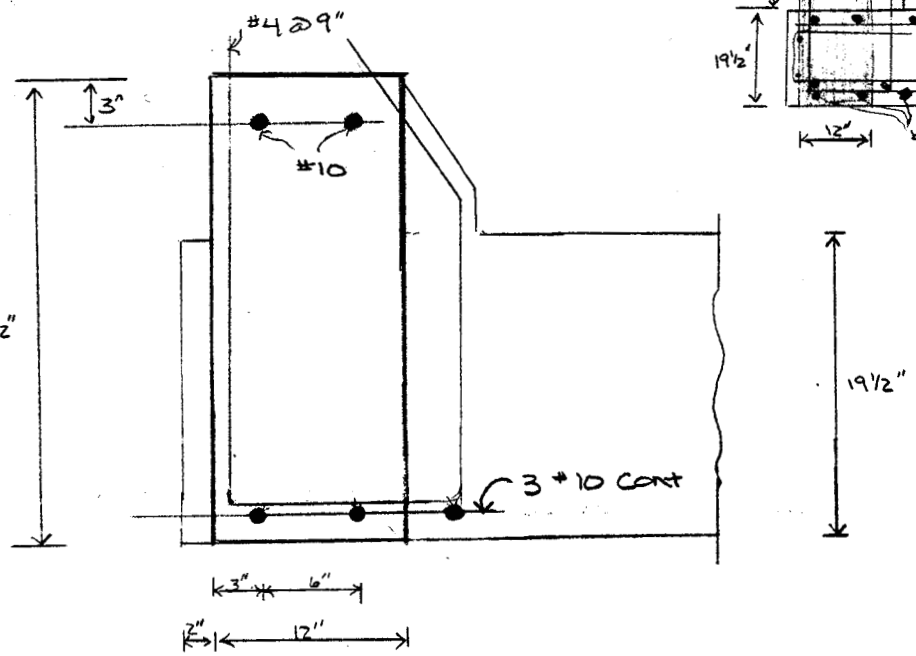
@ 0.4 AB M_T = 69.7 + 12.3 = 82.0 $^{k-ft}$

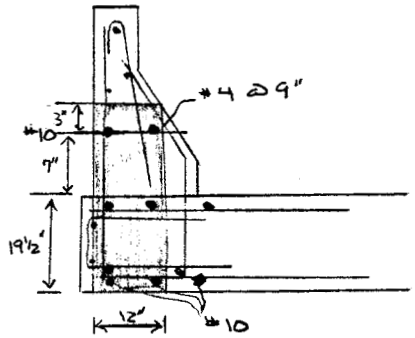
@ B M_T = -97.9 - 6.0 = -103.9 $^{k-ft}$

@ 0.5 BC M_T = 31.0 + 11.8 = 42.8 $^{k-ft}$

Deck Design

Edge Beam:





Design for $M = -103.9$ k-ft Use 2 #10

$d = 29\frac{1}{2} - 3 = 26\frac{1}{2}"$ use $d = 26"$ $A_s = 2(1.27) = 2.54$ in²

$n\frac{A_s}{bd} = 8\frac{2.54}{(12)(26)} = 0.0651$ $k = 0.301$ $j = 0.899$ req'd $d = \sqrt{\frac{103.9(12)}{\frac{1}{2}(1.6)(.301)(.899)(12)}} = 21.9"$

$21.9" < 26"$ ∴ o.k

$f_s = \frac{103.9(12)}{(2.54)(.899)(26)} = 21.0$ ksi < 24.0 ksi

$f_c = \frac{0.301(21.0)}{8(1-0.301)} = 1.13$ ksi < 1.60 ksi ∴ Section o.k. for moment

Posit. Moments – Use 2 #10

$M = 82.0$ k-ft < 103 k-ft ∴ O.K.

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Deck Design

Shear

From Deck Calc

$V_{max\ D.L} = (6.16)(1)\left(\frac{0.245}{0.27}\right) + \left(\frac{0.4}{0.27}\right)(6.16) = 14.7^k$; $V_{LL} = 0$ (Section Under Barrier)

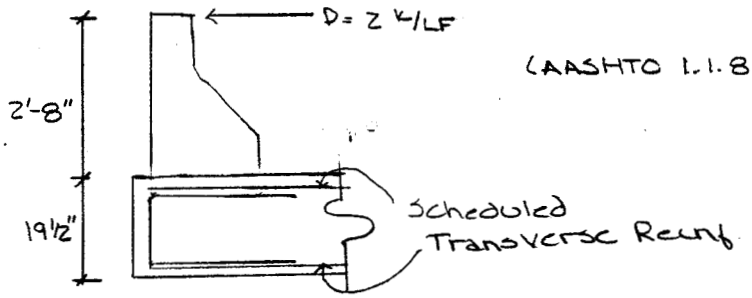
$v = \frac{V}{bd} = \frac{14.7}{(12)(26)} = 0.047$ ksi

$v_c = 0.95\sqrt{4000} = 60.1$ psi > 47 psi > 60.1/2 psi

AASHTO 1.5.10 : req'd $A_v = \frac{50 b_w s}{F_y}$; For s = 9" $A_v = \frac{50(12)(9)}{40{,}000} = 0.135$ in²

#4 bar $A_b = 0.20$ in² > 0.135 in² ∴ Section O.K. for Shear

Check Resistance to Barrier Load



(AASHTO 1.1.8 (A))

Deck - $M/ft = (2^k/LF)(2.667') = 5.3^{k\text{-}'}/LF$

Gross Concrete Section:

$S/ft = 12(19.5)^2/6 = 760$ in³

$f_b = M/S = 5.3(12)/760 = 0.084$ ksi

AASHTO 1.5.26: $F_b = 0.21(7.5\sqrt{4000}) = 99.6$ psi > 84 psi

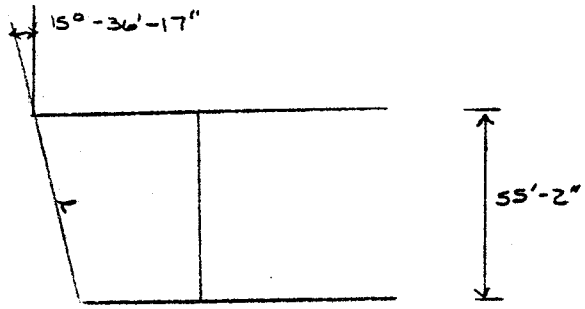
∴ Section O.K to Resist Barrier Load

Pier Design

Pier Design

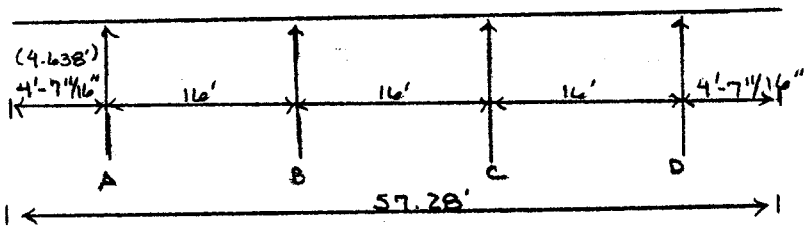
Pier Cap Design

Bridge Skew



Length of Pier Girder = 57.28'

Place piers 16' apart



From Slab Calculations:

$R_{DL} = 11.56$ k/ft Due to skew $R_{DL} = 0.963(11.56) = 11.13$ k/ft

Use pier girder $h = 7\frac{1}{2}''$ $w = 4.0'$

Load from barrier = 0.4 k/ft
from slab calc. concentrated load = $0.4/0.27(11.56) = 17.12$ k

$w_{DL} = 11.13 + 2(17.12)/57.3' + (.150\,k/ft^3)(7.5/12)(4') = 12.1$ k/ft

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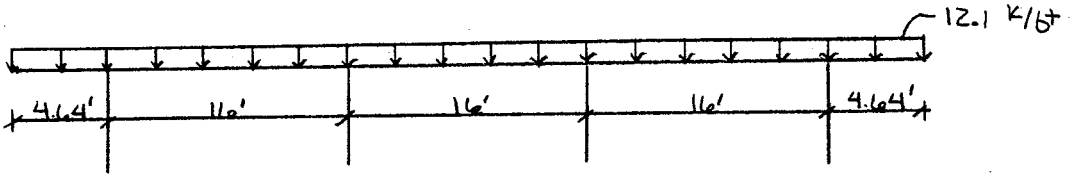
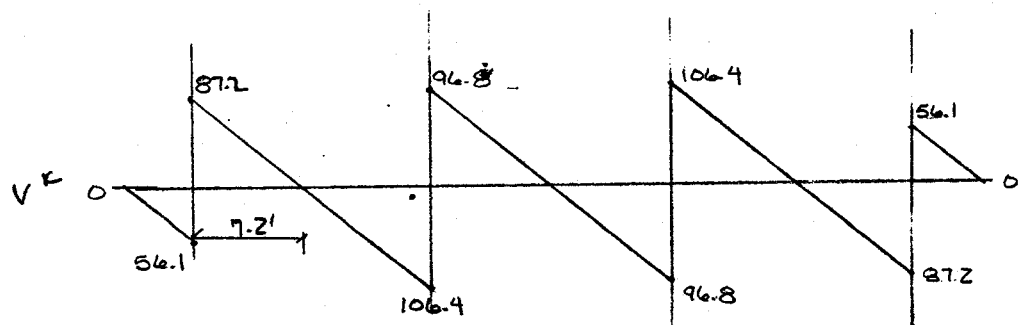
Pier Design

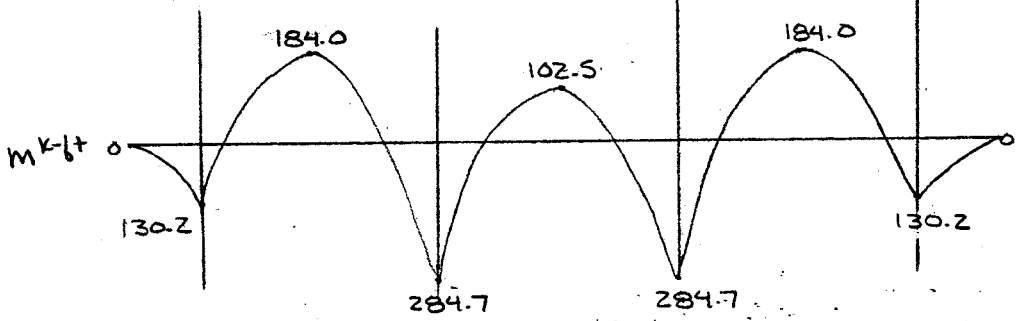
DL Mom Distr.:



130.2	-258.1	258.1		-258.1	258.1	-130.2
	127.9	64.0		-64.0	-127.9	
	-130.2	322.1		322.1	130.2	

		3/7	4/7	4/7	3/7	
	-130.2	322.1	-258.1	258.1	-322.1	130.2
		-27.4	-36.6	36.6	27.4	
			18.3	-18.3		
		-7.8	-10.5	10.5	7.8	
			5.2	-5.2		
		-2.2	-3.0	3.0	2.2	
Σ	-130.2	284.7	-284.7	284.7	-284.7	130.2
	R=143.3K		R=203.2K	R=203.2K		R=143.3K





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Pier Design

Live Load -

Ref: 'Moments, Shears and Reactions for Continuous Highway Bridges'

Table 3.1 Interpolate for "L" = 37.5 - (See Appendix)

Max LL Reaction = 65.8 k/lane

No. of lanes = 52/12 = 4.33 Use 4 lane (AASHTO 1.2.6)

$w_{LL} = 65.8(4)/(52'/\cos 15.60) = 4.88$ k/ft Use $w_{LL} = 5.0$ k/ft

$+M = \frac{5.0}{12.1}(184) = 76.0$ k-ft

$-M = \frac{5.0}{12.1}(285) = 118$ k-ft

D+L+I:

$+M = 184 + 1.3(76) = 283$ k-ft

$-M = 285 + 1.3(118) = 438$ k-ft

Top Reinf: Use 10 #10

$-M = 438$ k-ft Try 10 #10 $A_s = 10(1.27) = 12.7$ in^2

$d = 27'' - 2'' - 1.41'' - .625'' - .625'' - 1.27''/2 = 21.7''$ Use $d = 21''$

$n\frac{A_s}{bd} = 8\frac{12.7}{(48)(21)} = 0.1008$; $k = 0.359$; $j = 0.880$

req'd $d = \sqrt{\frac{438(12)}{\frac{1}{2}(1.6)(.359)(.880)(48)}} = 20.8'' < 21''$ ✓

$f_s = \frac{438(12)}{12.7(.880)(21)} = 22.4$ ksi < 24.0 ksi

$f_c = \frac{.359(22.4)}{8(1-.359)} = 1.57$ ksi < 1.60 ksi ∴ O.K.

Pier Design

Bottom Reinf Use 10 #8

$+M = 283^{k\text{-}ft}$ Try 10 #8 $A_s = 10(0.79) = 7.9 \text{ in}^2$

$d = 27'' - 1\tfrac{1}{2}'' - .625'' - 1.0/2 = 24.4''$ Use $d = 24''$

$n \frac{A_s}{bd} = 8 \frac{7.9}{(48)(24)} = 0.0549$ $k = 0.281$; $j = 0.906$

$f_s = \frac{283(12)}{7.9(.906)(24)} = 19.8 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{0.281(19.8)}{8(1-.281)} = 1.0 \text{ ksi} < 1.60 \text{ ksi}$ ∴ O.K

Shear Reinforcement:

@ d (22") from support: $V_D = 106.4^k - 12.1(22/12) = 84.2^k$

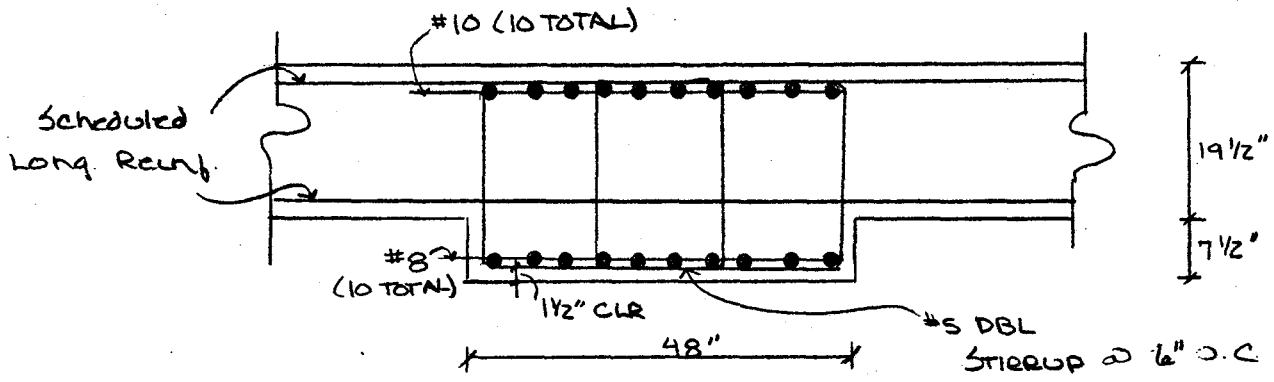
$V_L = 5.0^k/ft (14/2 - 22/12) = 30.8^k$

$V_{D+L+I} = 84.2^k + 1.3(30.8^k) = 125^k$

$v = V/bd = \frac{125^k}{(48)(22)} = 0.118 \text{ ksi}$ $v_c = 0.95\sqrt{4000} = 60.1 \text{ ksi}$

#5 DBL Stirrups: $A_v = 4(0.31) = 1.24 \text{ in}^2$

Req'd $s = \frac{1.24(20{,}000)}{(118-60)(48)} = 8.9''$ Use $s = 6''$



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Pier Design

Pier Column

- Lateral Loading

Analysis Using NAVDOCKS DM-7 - Lateral Loads on Piles - See Appendix

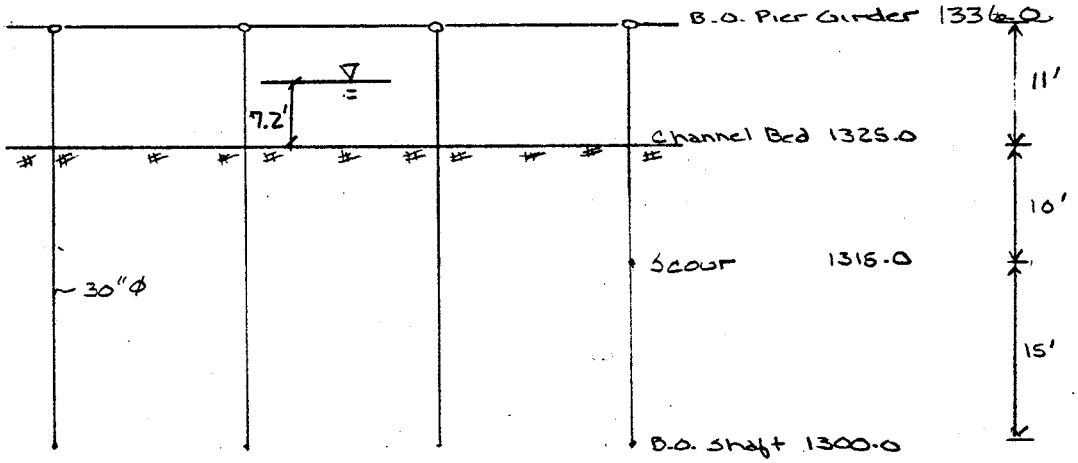
From Soil Report:

Coeff. of var. of lateral subgrade reaction $F = 60\ \#/in^3$

$F = 30\ \#/in^3$ w/ EQ

- Flexible Cap, Elevated Position Case

Transverse Model:



Stream Flow Forces:

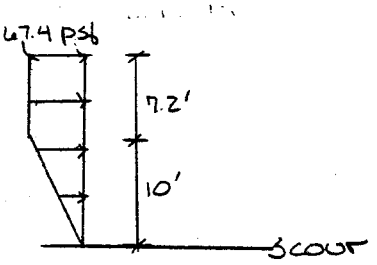
(AASHTO 1.2.17) $P = KV^2$

$V = 7.0$ ft/s $K = 1.375$ (assuming flat face from debris)

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

water level @ 1332.2'

Assumed SF loading:



Pier Design

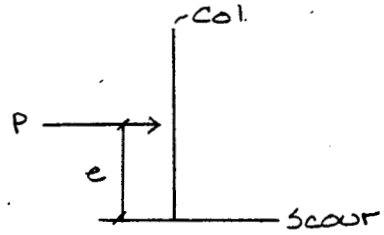
Stream Flow Forces (cont)

@ Debris laden pier effective col. width = 6.5'
(Per Corp of Engrs Recommendation)

$P = (67.4)(7.2)(6.5) + 1/2(67.4)(10)(6.5)$

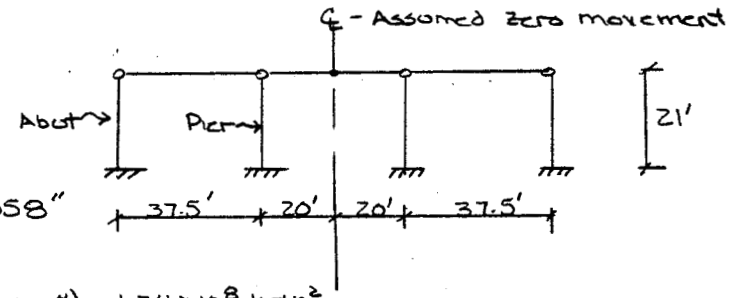
$= 3154^{\#} + 2190^{\#} = 5344^{\#}$

$$e = \frac{3154(10 + 7.2/2) + 2190(2(10)/3)}{5344} = 10.76'$$



Temperature Forces

Longitudinal Direction:

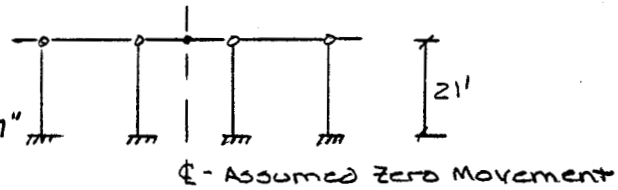


$\Delta_{pier} = (0.000006)(20')(12)(40°) = 0.058''$

$P = \frac{3EI\Delta}{L^3}$ $EI = 57\sqrt{3000}\,(39{,}761\ in^4) = 1.24 \times 10^8\ k\text{-}in^2$

$$P = \frac{3(1.24 \times 10^8)(0.058'')}{(21 \times 12)^3} = 1.35^k \quad @\ e = 21'$$

Transverse Direction:



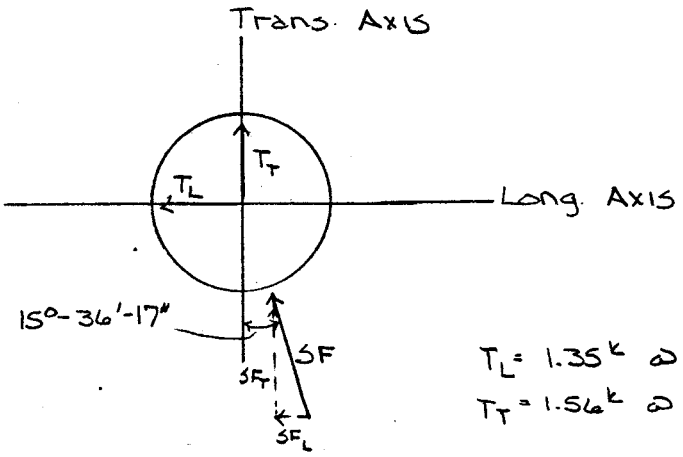
$\Delta_{ext\ pier} = (0.000006)(23.1')(12)(40) = 0.067''$

$$P = \frac{3(1.24 \times 10^8)(.067)}{(21 \times 12)^3} = 1.56^k$$

@ e = 21'

Pier Design

Load Case IV - Stream Flow & Temp.



$T_L = 1.35^k$ @ 21'

$T_T = 1.56^k$ @ 21'

$SF = 5.35^k$ $\quad SF_L = 5.35 \sin 15.6° = 1.44^k$, $SF_T = 5.35 \cos 15.6° = 5.15^k$ } @ 10.8'

$P_L = 1.35^k + 1.44^k = 2.79^k$ @ $\dfrac{1.35(21) + 1.44(10.8)}{2.79} = 15.7'$

$P_T = 1.56^k + 5.15^k = 6.71^k$ @ $\dfrac{1.56(21) + 5.15(10.8)}{6.71} = 13.2'$

$P = ((2.79)^2 + (6.71)^2)^{1/2} = 7.27^k$ @ $\dfrac{2.79(15.7) + 6.71(13.2)}{(2.79 + 6.71)} = 13.9'$

M @ scour $= 7.27^k (13.9') = 101.^{k\text{-}ft}$

Ref: NAVDOCKS DM-7 - See Appendix

$EI = 1.24 \times 10^8$ k-in^2 $\quad f = .06$ k/in^3 $\quad T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{1.24 \times 10^8}{.06}\right)^{1/5} = 73''$

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

Pier Design

Max Moment: (1982 DM-7, Figure 11 - See Appendix)

@ $z/T = 0$ $\quad M_m = 1.0(101) = 101^{k\text{-}ft}$

$M_p = 0.0$

@ $z/T = 1.0$ $\quad M_m = 0.75(101) = 76^{k\text{-}ft}$

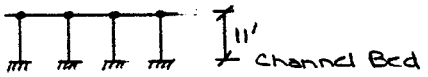
$M_p = 0.60(7.27)(73/12) = \underline{26^{k\text{-}ft}}$

$102^{k\text{-}ft}$ - controls

Earthquake Forces: (AASHTO 1.2.20)

EQ = CFW

F = 1.0



W = Dead Load:

Wearing Surface	$= (.025)(52)(115)$	$= 150^k$
Barriers	$= (.4^k/ft)(115)(2)$	$= 92^k$
Slab	$= (.150)(19.5/12)(115)(55.2)$	$= 1547^k$
Haunches	$= 2(.150)(4')(7.5/12)(57.3')$	$= 43^k$
Columns (to fun. grade)	$= 8(.150)(11)(\pi(1.25)^2)$	$= 65^k$
Abut. Diaphragm	$= 2(.150)(2.5')(3')(57.3')$	$= \underline{129^k}$
		2026^k

Determine P for structure $\quad P = P_{piers} + P_{abuts}$

$$P_{piers} = \frac{3EI\Delta}{L^3} \times 4 \text{ cols.} \times 2 \text{ piers} \qquad \Delta = 1'' \qquad EI = 1.24 \times 10^8 \ k\text{-}in^2$$

$$P_{piers} = \frac{3(1.24 \times 10^8)(1'')}{(11 \times 12)^3} \times 8 = 1294^k$$

Pier Design

Determine P_{abuts} using DM-7

$E = 3122$ ksi $\quad I = \pi(12'')^4/4 = 16286$ in^4 $\quad EI = 5.08 \times 10^7$ k-in^2

$f = 0.03$ k/in^3 $\quad T = \left(\frac{5.08 \times 10^7}{.03}\right)^{1/5} = 70''$

$L = 1328' - 1300' = 28'$ $\quad L/T = 28(12)/70 = 4.8$

$P_{abuts} = \frac{\Delta EI}{F_\Delta T^3} \times 5$ shafts $\times 2$ abuts $\quad \Delta = 1'' \quad @\ Z/T = 0 \quad F_\Delta = 2.3$

$P_{abuts} = \frac{1(5.08 \times 10^7)}{2.3\,(70)^3} \times 10 = 644^k$

$\therefore P = 1294^k + 644^k = 1938^k$

Determine EQ force / pier col.

Assumptions:

- Pier Columns Resist Entire Seismic Force
- Seismic Force Is Equally Distributed Between Columns
- C.G. of Structure @ B.O. Deck

$T = 0.32\sqrt{W/P} = 0.32\sqrt{2026/1938} = 0.33$ sec $\rightarrow C = 0.08$ (AASHTO Fig. 1.2.20D)

$EQ = 0.08(1.0)(2026^k) = 162^k$

$EQ/col = 162^k/8 = 20.2^k$

M_{EQ} @ Invert $= 20.2^k(11') = 222^{k-'}$

Determine Max M_{EQ} using DM-7 - See Appendix

$L = 25'$ $\quad f = .03$ k/in^3 $\quad T = \left(\frac{1.24 \times 10^8}{.03}\right)^{1/5} = 84''$ $\quad L/T = 25(12)/84 = 3.6$

@ $Z/T = 0$ $\quad M_m = 1.0(222) = 222$ $\quad M_p = 0.0$

@ $Z/T = 1.2$ $\quad M_m = 0.76(222) = 169^{k\text{-}ft}$

$M_p = 0.73(20.2)(84/12) = 103^{k\text{-}ft}$

Controls $\rightarrow 272^{k\text{-}ft}$

Pier Design

Load Cases - Check If Section Cracks

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

Load Case IV $1.0(D+L+SF+T) \leq 1.25$

$M = 102^{k\text{-}ft}/1.25 = 81.6^{k\text{-}ft}$; $S = \pi(15)^3/4 = 2651 \text{ in}^3$

$f_b = 81.6(12)/2651 = 0.369 \text{ ksi} < 0.411 \text{ ksi}$ ∴ O.K.

$M_{CASE\ IV} = 81.6^{k\text{-}ft}$

$P_D = 143^k + 16^k = 159^k$

$P_L = 54^k$

$P_{CASE\ IV} = (159^k + 54^k)/1.25 = 170^k$

Load Case VII $1.0(D+SF+EQ) \leq 1.33$

Assume EQ & SF Do Not Occur Simultaneously
- EQ Controls

$M = 272/1.33 = 205^{k\text{-}ft}$

$f_b = 205(12)/2651 = 0.928 \text{ ksi} > 0.411 \text{ ksi}$

∴ Recalculate M using Revised Stiffness for cracked section.

Cracked Section EI

Ref: AASHTO (1.5.34)

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

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

Ref: Helms & 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 12 #11 $\rho_t = \frac{12(1.56)}{\pi(15)^2} = 0.0265$

$EI = 3122(\pi)(15)^4/4 (0.2 + 1.2(.0265)\frac{29000}{3122}) = 61{,}494{,}000 \text{ k-in}^2$

Pier Design

$61,494,000\ k\text{-}in^2 > 49,653,000\ k\text{-}in^2$

$\therefore$ use $EI = 61,494,000$ (more conservative) for recalc. of EQ Mom.

$$P_{piers} = \frac{3EI\Delta}{L^3} \times 8 = \frac{3(61,494,000)(1")}{(11\times12)^3} \times 8 = 642^k$$

Calc. P_{abut} using DM-7

$EI_{abut\ shafts} = 20,338,000\ k\text{-}in^2$ (see Abut. Design)

$$T = \left(\frac{20,338,000}{.03}\right)^{1/5} = 58.4" \qquad L/T = 28(12)/58.4 = 5.75$$

@ $Z/T = 0$ $\quad F_\Delta = 2.3$ $\quad P_{abut} = \frac{1(20,338000)}{2.3(58.4)^3} \times 10 = 444^k$

$P = 642 + 444 = 1086^k$

$T = 0.32\sqrt{2026/1086} = 0.44 \rightarrow C = 0.09$ (AASHTO Fig. 1.2.20 D)

$EQ = 0.09(1.0)(2026) = 182^k$ $\quad EQ/col = 182/8 = 22.8^k$

M_{EQ} @ Invert $= 22.8^k(11') = 251\ k\text{-}ft$

Determine Max M_{EQ} using DM-7 - See Appendix

$L = 25'$ $\quad F = .03\ k/in^3$ $\quad T = \left(\frac{61494000}{.03}\right)^{1/5} = 72.8"$ $\quad L/T = 25(12)/72.8 = 4.1$

@ $Z/T = 0$ $\quad M_m = 1.0(251) = 251\ k\text{-}ft$

$M_p = 0.0$

@ $Z/T = 1.25$ $\quad M_m = 0.77(251) = 193\ k\text{-}ft$

$M_p = 0.76(22.8)(72.8/12) = \underline{105\ k\text{-}ft}$

$298\ k\text{-}ft \leftarrow$ Controls

Pier Design

$M_{CASE\ VII} = 298^{k\text{-}ft}/1.33 = 224^{k\text{-}ft}$

$P_D = 143^k + 8^k = 151^k \quad P_{CASE\ VII} = 151/1.33 = 114^k$

∴ Load Case VII will control design

Use $l_u = 11'$ (streambed to b.o. girder) per USDOT

$K = 2.1$ (fixed @ bott; pinned @ top)

$r = 0.25(30) = 7.5''$

$Kl_u/r = 2.1(11)(12)/7.5 = 37$

$\phi = 1.0$; $\beta_d = 0.0$ (No DL Mom transfer to colomns)

$C_m = 0.0$

$EI = 61{,}494{,}000\ k\text{-}in^2$

$$P_c = \frac{\pi^2(61\,494\,000)}{((2.1)(11)(12))^2} = 7900^k$$

$$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \qquad P_u = 2.5P \quad \text{(AASHTO 1.5.28)}$$

$$\delta = \frac{1.0}{1 - \frac{2.5(114)}{7900}} = 1.04$$

$M_c = \delta M = 1.04(224) = 233^{k\text{-}ft}$

Design Loads: $M = 233/0.35 = 666^{k\text{-}ft}$

$P = 114/0.35 = 326^{k\text{-}ft}$

Use 12 #11 Verts

(∴ O.K. See Circon output)

```
CIRCON

-P-

FY=60,000.00
Fc=3,000.00
D=30.00
D1=4.21
AS1=1.56
N=12.00
P=326.00
MX=666.00
MY=1.00E-8

REBAR COORDINATES
Rb=10.79 ∠=0.00
Rb=10.79 ∠=30.00
Rb=10.79 ∠=60.00
Rb=10.79 ∠=90.00
Rb=10.79 ∠=120.00
Rb=10.79 ∠=150.00
Rb=10.79 ∠=180.00
Rb=10.79 ∠=210.00
Rb=10.79 ∠=240.00
Rb=10.79 ∠=270.00
Rb=10.79 ∠=300.00
Rb=10.79 ∠=330.00

(.8)PMAX=1,611.65

ebR=15.44
MbR=8,107.26
PbR=525.18
cbR=15.26
B∠=8.60E-10

P=325.16
MX=668.93 >666
MY=1.00E-8
cR=13.29
ecR=2.06
```

Pier Design

Pier- Broms Analysis (Check Lateral Load Capability)

Broms Analysis - See Appendix

Per Soil Report - Need to Consider 2 cases:

1) Cohesive case: $k_h D = 400\#/in^2$ ($200\#/in^2$ for seismic)

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

For ultimate lateral resistance: $\phi = 30°$, $C_u = 900$ psf

Passive Earth Pressure Coeff. - $K_p = 3.0$

$\gamma = 120\#/ft^3$

Concrete Pier: $D = 30''$ $I = 39{,}761\ in^4$

$E = 3122$ ksi (3000 psi conc)

For Cohesive Case:

$$\beta = \sqrt[4]{\frac{k_h D}{4EI}} = \sqrt[4]{\frac{400}{4(3{,}122000)(39{,}761)}} = 0.0053 \quad (0.0045 \text{ for seismic})$$

L- Earthquake = 25' L- Stream Flow = 15'

SF: $\beta L = .0053(15)(12) = 0.95 < 2.25$

EQ: $\beta L = .0045(25)(12) = 1.35 < 2.25$ ∴ Rigid Pile Action

Earthquake: $L = 25'$

$P = 20^k$

$e = 11'$

$e/D = 11/2.5 = 4.4$

$\frac{P}{C_u D^2} = \frac{20000}{(900\ psf)(2.5)^2} = 3.56$

$L/D = 5.0 \pm$ $L = 5.0(2.5) = 12.5' < 25'$

$FS = 25/12.5 \times 1.33 = 2.66$

Stream Flow & Temp: $L = 15'$

$P = 7.3^k$

$e = 13.9'$

$e/D = 13.9/2.5 = 5.56$

$\frac{P}{C_u D^2} = \frac{7300}{900(2.5)^2} = 1.30$

$L/D = 4 \pm$ $L = 4.0(2.5) = 10 < 15'$

$FS = 15/10 \times 1.25 = 1.88$

Pier Design

Broms Analysis of Piers - Cont.

For Cohesionless Case:

$$\eta = \sqrt[5]{\frac{n_h}{EI}} = \sqrt{\frac{60\#/in^3}{(3122000)(39761)}} = 0.0137 \quad (0.0119 \text{ for seismic})$$

SF: $\eta L = 0.0137(15)(12) = 2.47 > 2.0$

EQ: $\eta L = 0.0119(25)(12) = 3.57 > 2.0$

∴ Flexible Pile Action

- Embedment length no longer a factor; Flexible Pile Action - for which the piles are designed - takes over.

∴ Embedment length o.k. for lateral load

Pier Design

Pier Top @ Elev. 1300.0'

Axial Load = $203^k + 80^k = 283^k$

From Fig. 4 - Sergent, Hauskins & Beckwith Soil Report (attached)

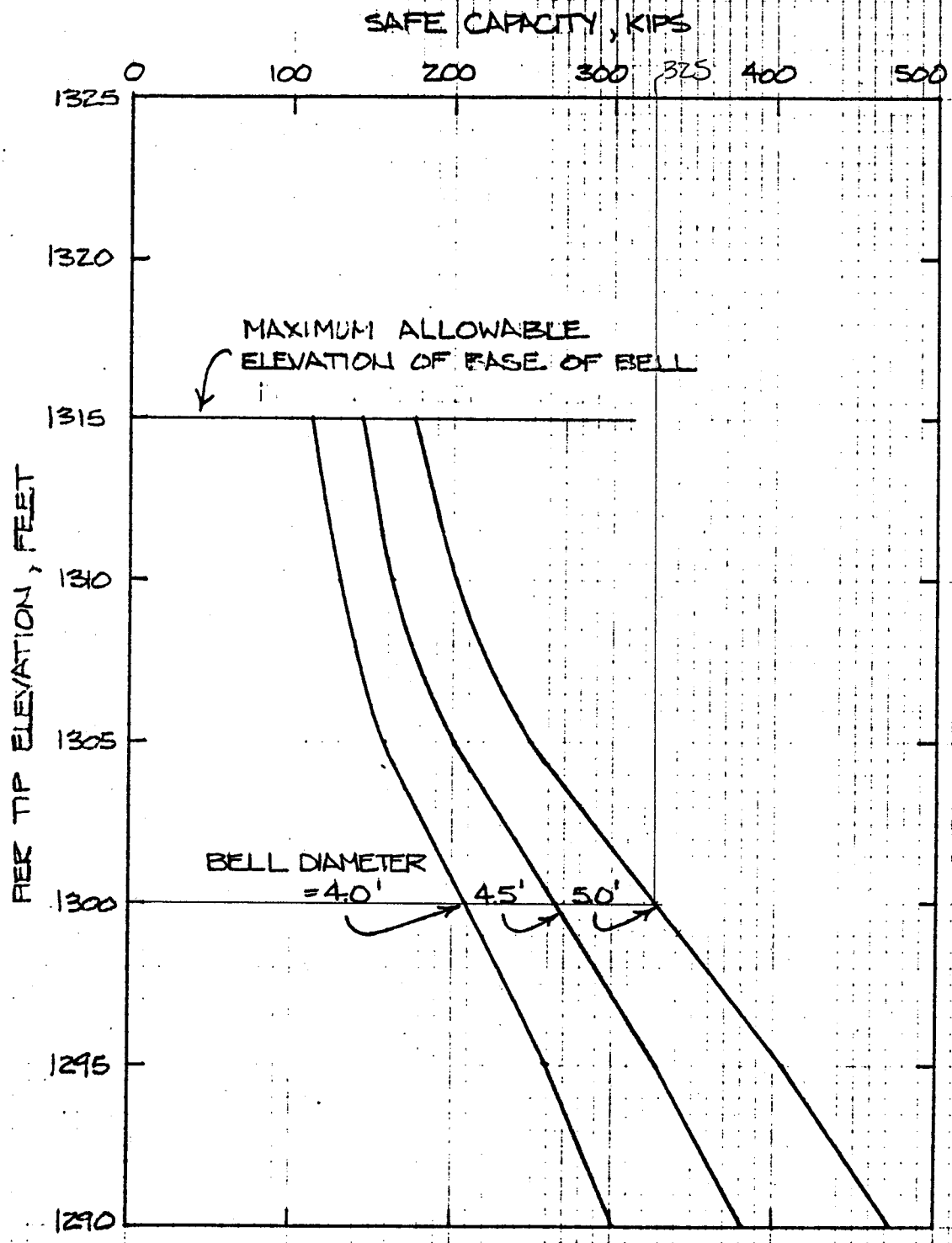
For 5.0' Bell Diam. @ 1300' Safe Capacity = $325^k > 283^k$

Use 30" Ø Piers w/ 12 #11 Verts & 1/2" Ø Spiral Ties @ 6" O.C. w/ 5.0' Ø Bell & pier top elev. @ 1300'

Pier Design

FIGURE 4

SAFE DOWNWARD CAPACITIES FOR BELLED PIERS.



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Job No: E84-Z
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Pier Design

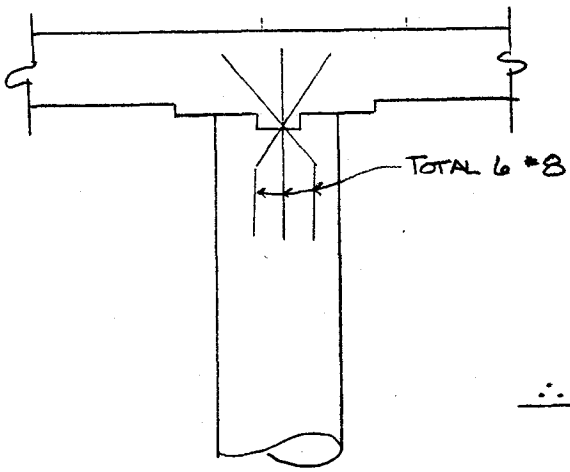
Pier Col. Pin Design

Temp. Force = $(1.35^2 + 1.56^2)^{1/2} = 2.06^k$

Earthquake Force = 22.8^k

$\therefore$ EQ controls Design Pin for $P = 22.8^k / 1.33 = 17.1^k$

AASHTO 1.5.29 (D)



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

$f_s = 20$ ksi $\mu = 1.4$

Req'd $A_{vf} = 17.1^k / 20(1.4) = 0.61 \text{ in}^2$

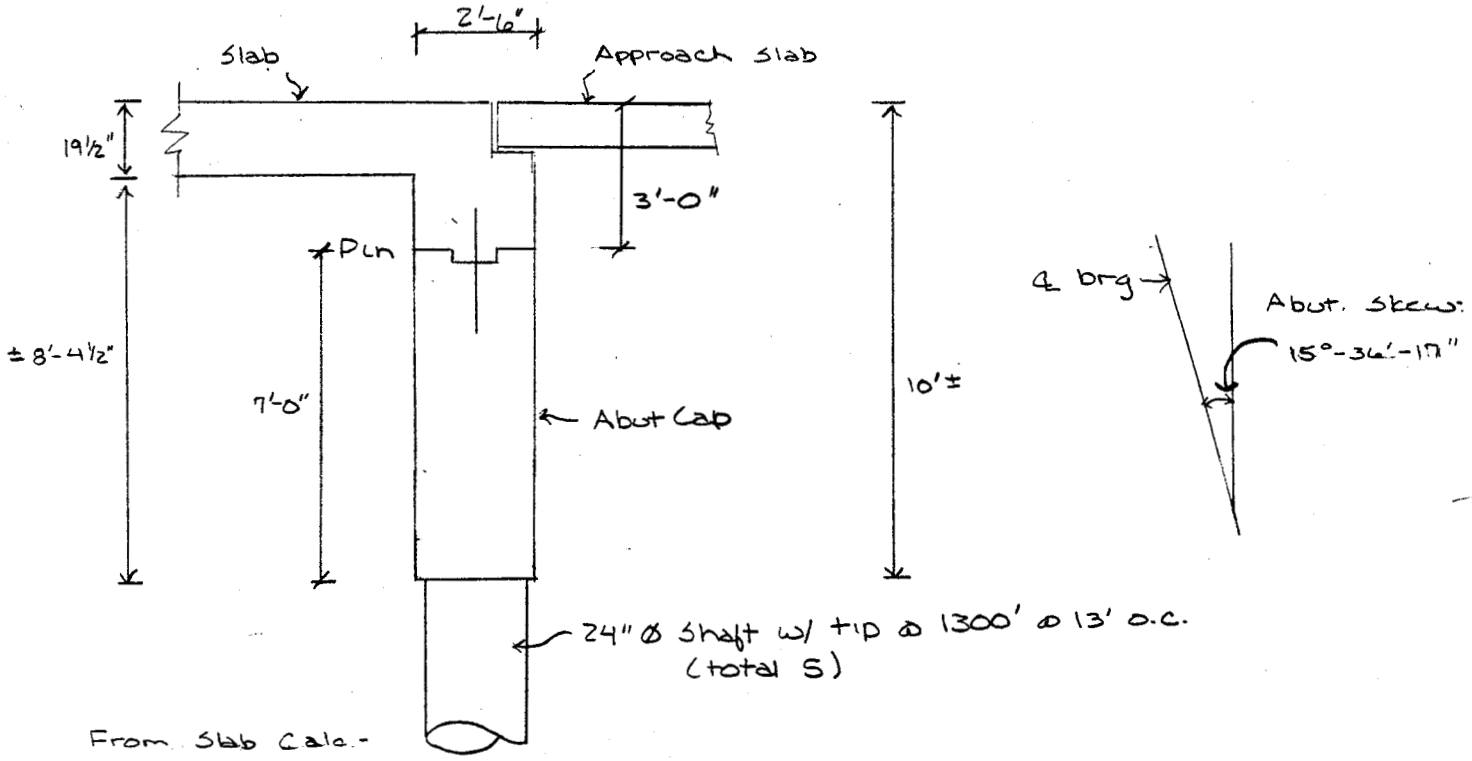
$A_s = 6(0.79) = 4.74 \text{ in}^2 > 0.61 \text{ in}^2$

$\therefore$ Use 6 #8 For Pin

Abutment Design

Abutment Design

Abutment Cap



From Slab Calc.-

$D_L = 3.97\ ^k/ft$ Due to Skew $D_L = 0.963(3.99\ ^k/ft) = 3.84\ ^k/ft$

weight of abutment $= (0.150\ ^k/ft^3)(10')(2.5') = 3.75\ ^k/ft$

Barrier : $0.4\ ^k/ft \times 2 = 0.8\ ^k/ft$ Load from barrier $= \frac{0.8}{0.27}(3.97)/(55.17/\cos 15.60)$
$= 0.21\ ^k/ft$

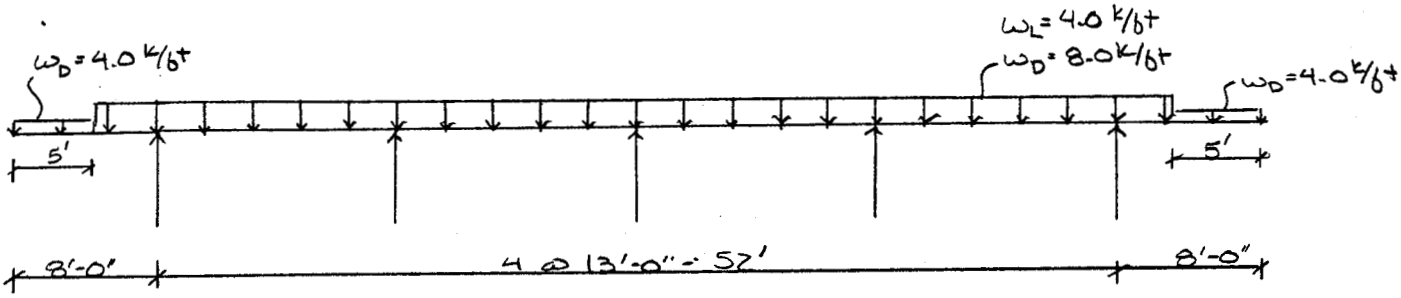
$w_D = 3.84 + 3.75 + 0.21 = 7.80\ ^k/ft$ use $w_D = 8.0\ ^k/ft$

Using Table 3.1 - 'Moments, Shears & Reactions for Continuous Highway Bridges'
N = 1.1 Interpolating for 'L' = 37.5'

Max Reaction @ A = 50.8 k/lane (4 lanes)

$w_L = \frac{50.8(4)}{55.17/\cos 15.6^\circ} = 3.55\ ^k/ft$ use $w_L = 4.0\ ^k/ft$

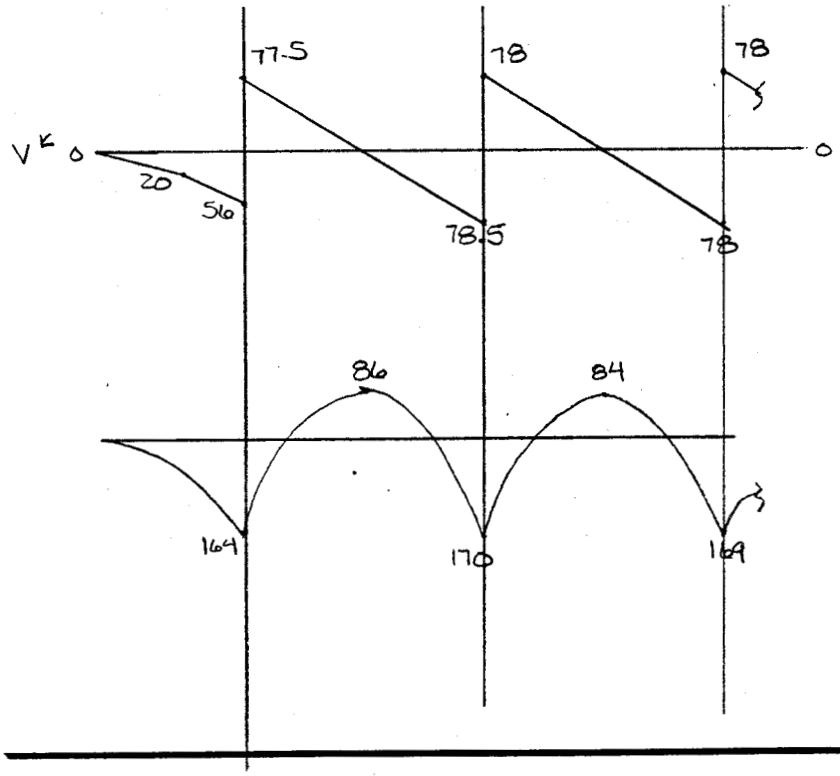
Abutment Design



(D+L)

164	-169	169			-169	169	-164
	5	2			-2	-5	
	-164	171			-171	164	

	3/7 | 4/7		1/2 | 1/2		4/7 | 3/4	
-164	171	-169	169	-169	169	-171	164
	-1	-1	0	0	1	1	
-164	170	-170	169	-169	170	-170	164
$P_D = 95.5^k$	$P_D = 104.5^k$		$P_D = 104^k$		$P_D = 104.5^k$		$P_D = 95.5$
$P_L = 38^k$	$P_L = 52^k$		$P_L = 52^k$		$P_L = 52^k$		$P_L = 38^k$



Max $+M_{D+L} = 86$ k-ft

Max $-M_{D+L} = 170$ k-ft

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Abutment Design

Design for $-M = 170^{k\text{-}ft}$

Try 3 #7 $A_s = 3(0.60) = 1.80 \text{ in}^2$ $d = 7'(12''/1) - 6'' - .625 - .875/2 = 76.9''$

use $d = 76''$

$n = \frac{29000}{57\sqrt{3000}} = 9$ $n\frac{A_s}{bd} = 9\frac{1.80}{(30)(76)} = 0.0071$ $k = 0.112$ $j = 0.962$

$f_s = \frac{170(12)}{1.80(.962)(76.9'')} = 15.3 \text{ ksi} < 20 \text{ ksi}$

$f_c = \frac{0.112(15.3)}{9(1-.112)} = 0.214 \text{ ksi} < 1.2 \text{ ksi}$

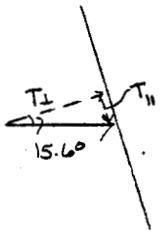
$\therefore$ o.k use 3 #7

Lateral Load Reinf.:

Lateral Loads - See Caisson Design Calcs

Earth Pressure: $.672^k/LF + .966^k/LF = 1.64^k/LF$

Temperature: 10.8^k/caisson



$T_\perp = 10.8^k \cos 15.6° = 10.4^k$

$w_\perp = 10.4^k/13' = 0.8^k/LF$

Lateral Load/LF $= 1.64^k/LF + 0.8^k/LF = 2.44^k/LF$

Load Case IV $w = 2.44/1.25 = 2.0^k/LF$

Use $w = 3.0^k/LF$

Abutment Design

Ref: 'Bridge Planning & Design Manual, Vol IV,' pg 6-22

Use #5 @ 12" $M/ft \approx \frac{3.0(7)}{4} = 5.25$ k-ft $A_s/ft = 0.31$ in²

$d = 30'' - 3'' - .625/2 = 26.7''$ Use $d = 26''$

$n\frac{A_s}{bd} = 9\frac{0.31}{(12)(26)} = 0.00894$ $k = 0.125$ $j = 0.958$

$f_s = \frac{5.25(12)}{0.31(.958)(26)} = 8.16$ ksi < 20.0 ksi

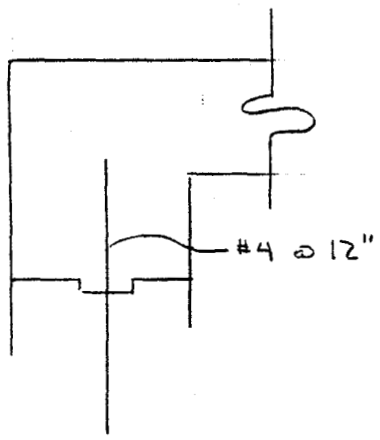
∴ o.k.

Check Shear:

$V = (0.966 + 1.2)/1.25 = 1.7^k$ $v = 1.7^k/(12)(26) = 0.005$ ksi $< \frac{1}{2}0.95\sqrt{3000} = 26$ psi

-No shear Reinf. Req'd:

Design Pin



AASHTO 1.5.29(D)

$A_{vf} = V/f_s\mu$

$f_s = 20{,}000$ psi $\times 1.25$...

$\mu = 1.4$

$V = .672^k/LF + 1.2^k/LF = 1.9^k/LF$

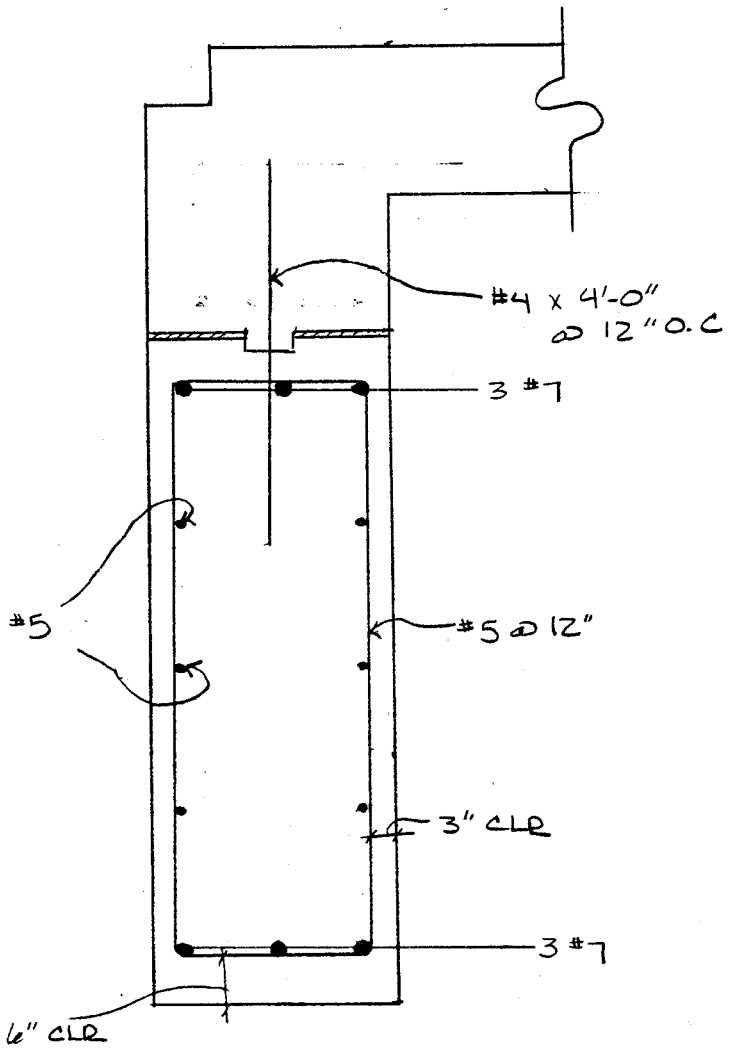
Use $V = 2.0^k/LF$

$A_{vf} = 2.0/20(1.25)(1.4) = 0.06$ in²/LF

#4 @ 12" o.k. $A_s = 0.20$ in²/LF

Abutment Design

Abutment Section



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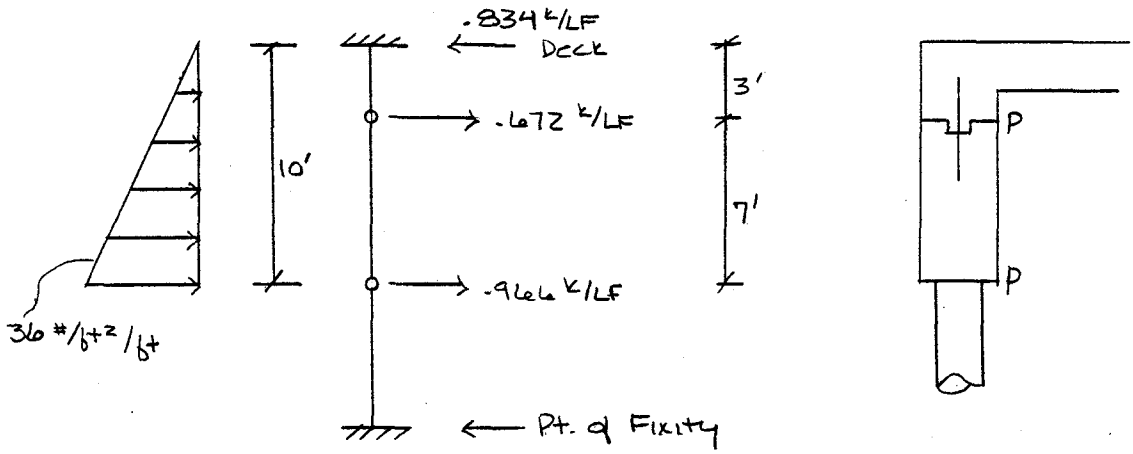
Abutment Design

Caisson Design

Assumptions:

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

Lateral Load due to Earth Pressure:

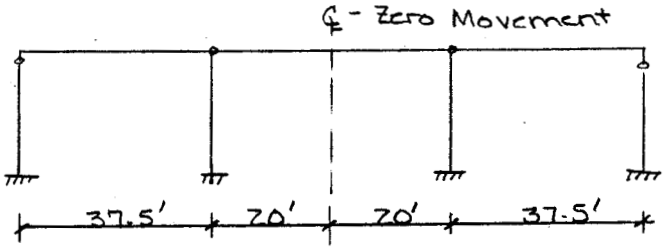


Ext. Caissons support 14.5'

Earth Pressure/ext. caisson = (.966 k/LF)(14.5') = 14.0k

Abutment Design

Lateral Load due to Temperature - Assuming Uncracked Section



$\Delta = (.0000006)(57.5')(12''/1)(40°) = 0.16''$

Use DM-7 to determine P_Δ (See Appendix)

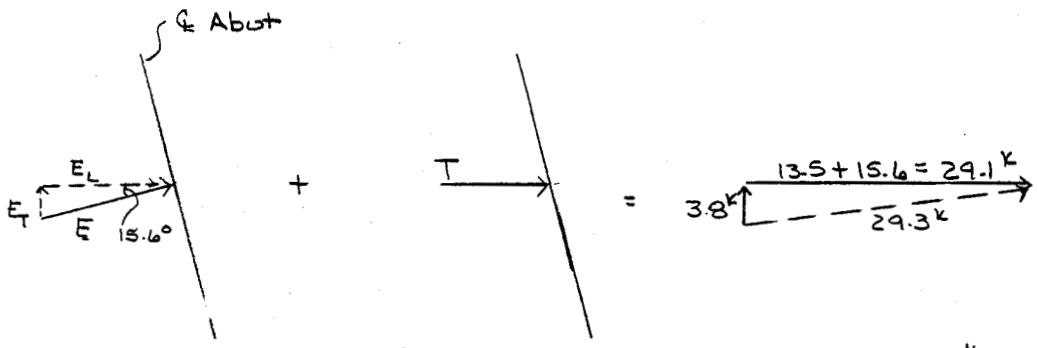
$f = .06\ ^k/in^3 \qquad EI = 3122\ ksi\,(\pi(12'')^4/4) = 5.08 \times 10^7\ ksi$

$T = \left(\frac{5.08 \times 10^7}{.06}\right)^{1/5} = 61'' \qquad L = 30' \qquad L/T = 30(12)/61 = 5.9$

$P_\Delta = \frac{\Delta EI}{F_\Delta T^3} \qquad @\ Z/T = 0 \qquad F_\Delta = 2.3 \qquad P_\Delta = \frac{0.16''(5.08 \times 10^7)}{2.3(61)^3} = 15.6^k$ /column

Load Case IV : $1.0(D+L+E+T) \leq 1.25$

E + T:



$E_L = 14.0^k(\cos 15.6°) = 13.5^k \qquad T = 15.6^k \qquad P = (3.8^2 + 29.1^2)^{1/2} = 29.3^k$

$E_T = 14.0^k(\sin 15.6°) = 3.8^k$

DM-7

$@\ Z/T = 1.3 \qquad M_p = 0.77(29.3^k)(61/12) = 115\ ^{k\text{-}ft}$

Abutment Design

- check if section cracks

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

Case IV $M = 115^{k\text{-}ft}/1.25 = 92^{k\text{-}ft}$; $S = \pi(12'')^3/4 = 1357$ in^3

$f_b = 92(12)/1357 = 0.814$ ksi > 0.411 ksi

∴ Recalculate M using Revised Stiffness representing cracked section

Cracked Section EI

Ref: AASHTO (1.5.34)

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

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

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

$$EI = E_c I_g \left(0.2 + 1.2\rho_t \frac{E_s}{E_c}\right)$$

For 8 #7 $\rho_t = 8(0.60)/\pi(12)^2 = 0.0106$

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

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

∴ Use $EI = 20{,}338{,}000$ k-in^2 for recalculation of Mom.

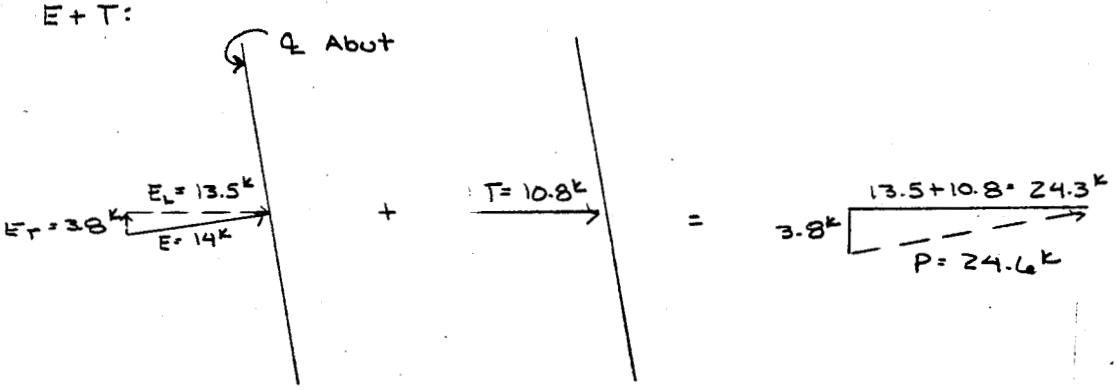
Use DM-7 to determine P_Δ due to temp $\Delta = 0.16''$

$T = \left(\frac{20{,}338{,}000}{.06}\right)^{1/5} = 50.8''$ $L/T = 30(12)/50.8 = 7.1$

@ $Z/T = 0$ $F_\Delta = 2.3$ $P_\Delta = \frac{0.16(20{,}338{,}000)}{2.3(50.8)^3} = 10.8^k$/caisson

Abutment Design

E + T:



DM-7 @ $z/T = 1.3$ $M_p = 0.77(24.6^k)(50.8''/12) = 80.2^{k\text{-}ft}$

- Slenderness Need Not be Considered -
Caissons fully embedded

Design Loads: $M_u = \frac{80.2^{k\text{-}ft}}{(1.25)(0.35)} = 183^{k\text{-}ft}$

$P_u = 307^k$

∴ O.K. Use 8 #7 - See Circon Ouput

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

                                                    -p-

FY=60,000.00
Fc=3,000.00
D=24.00
D1=4.00
AS1=0.60
N=8.00
P=307.00
MX=183.00
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=800.44

ebR=8.75
MbR=2,745.92
PbR=313.82
cbR=11.84
B∠=3.13E-9

P=306.54
MX=228.33   > 183'k
MY=1.25E-8
cR=11.71
ecR=0.74
```

O.K.

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Abutment Design

Abutment Caissons - Broms Analysis (See Appendix)

- Check Lateral Load Capability

Ref - Sergent, Hauskins & Beckwith Soil Report, SHB Job No E84-2

Need to consider 2 cases:

1) Cohesive Case $k_h D = 400\ \#/in^2$
2) Cohesionless Case $n_h = 60\ \#/in^3$

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

Concrete Pier $D = 24''$; $I_g = \pi(12)^4/4 = 16286\ in^4$; $E_c = 3122$ ksi

Cohesive Case:

$$\beta = \sqrt[4]{\frac{k_h D}{4EI}} = \sqrt[4]{\frac{.400}{4(3122)(16286)}} = 0.0067$$

$L = 30'$ Streamflow $\beta L = 0.0067(30)(12) = 2.41 > 2.25$

∴ Flexible Pile Action

Cohesionless Case:

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

$\eta L = 0.0164(30)(12) = 5.90 > 2.0$

∴ Flexible Pile Action

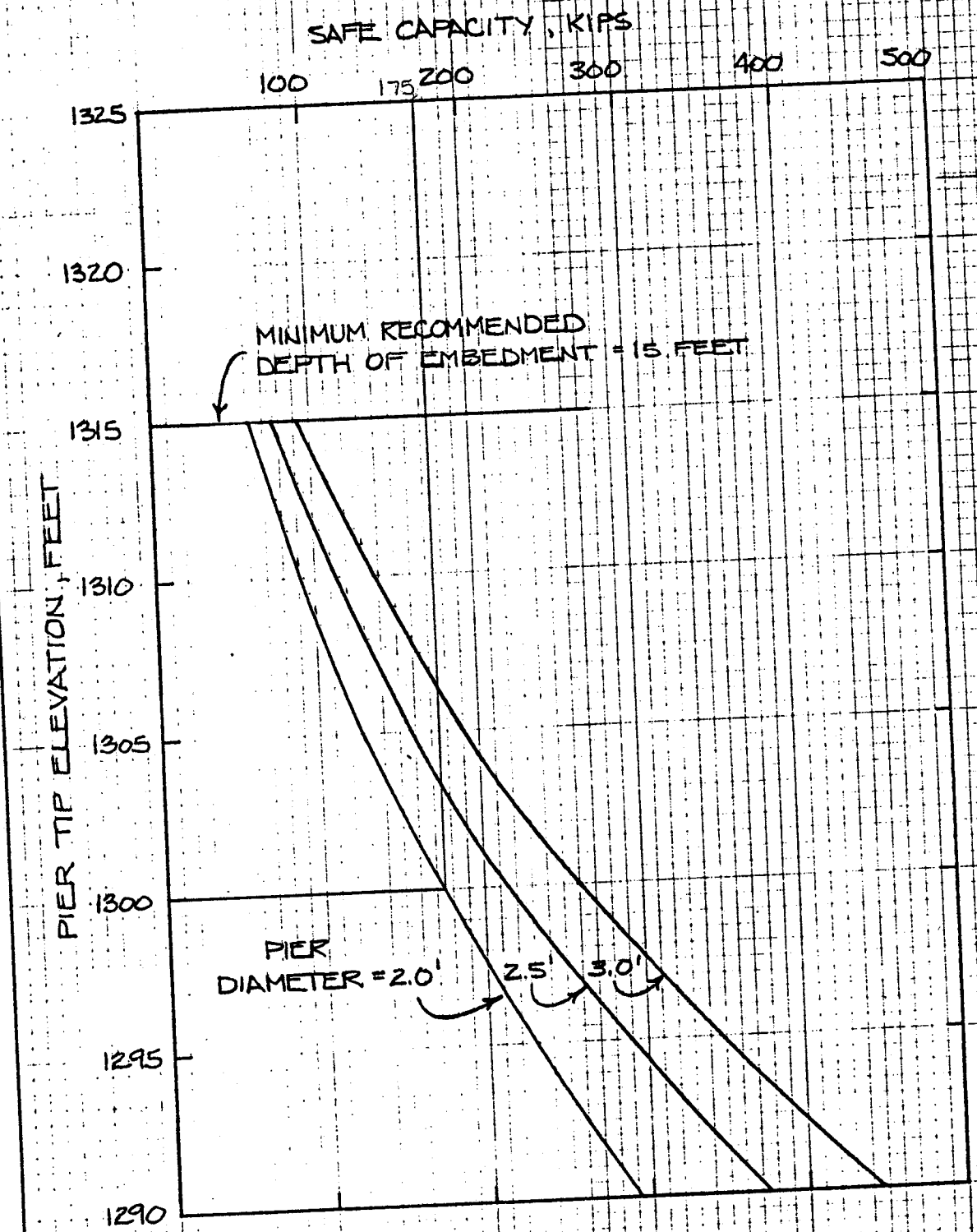
- Embedment length no longer a factor; Flexible pile action - for which the piles were designed - takes over

∴ Embedment length O.K. For Lateral Loads

Abutment Design

FIGURE 1

SAFE DOWNWARD CAPACITIES
FOR STRAIGHT PIERS
- ABUTMENT PIERS -



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Abutment Design

Axial Load = $P_D + P_L = 104.5^k + 52^k = 157^k$

Per SHB Soil Report, Figure 1 (attached)
@ 1300' Safe Bearing capacity = $175^k > 157^k$

Use 24" Ø Drilled Shafts @ 13'-0" o.c.
w/ 8 #7 Verts & 1/2" Ø Spiral Ties @ 6" pitch
Caisson Tip Elev. = 1300'

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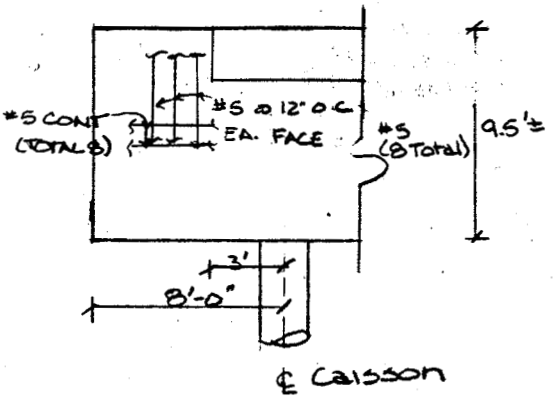
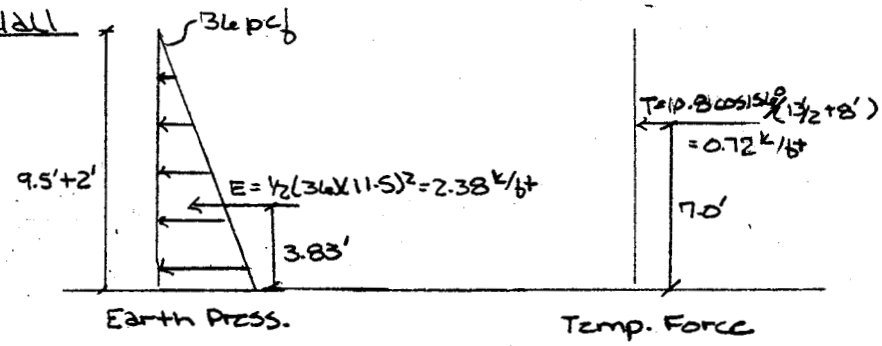
date 10/5/84 sh. 49 of
1/4/85

Abutment Design

Check Cantilevered End Wall





Lateral Loads

Vertical Cantilever

$M = [(2.38)(3.83) + 0.72(7)]/1.25 = 11.3^{k\text{-}ft}/ft$ $\quad n = \frac{29000}{57\sqrt{3000}} = 9$

$d = 30" - 2" - 0.625"/2 = 27.6"$ Use $d = 27."$; $A_s/ft = 0.31\ in^2$

$n\frac{A_s}{bd} = 9\frac{0.31}{(12)(27)} = 0.00861$ $\quad k = 0.123$ $\quad j = 0.959$

$f_s = \frac{11.3(12)}{(0.31)(0.959)(27)} = 16.9\ ksi < 20.0\ ksi$

$f_c = \frac{0.123(16.9)}{9(1-.123)} = 0.26\ ksi \leq 1.2\ ksi$

Horiz. Cantilever

$M = ((2.38 + 0.72)(8')^2/2)/1.25 = 79.4^{k\text{-}ft}$ $\quad A_s = 8(0.31) = 2.48\ in^2$; $d = 27"$; $b = 114"$

$n\frac{A_s}{bd} = 9\frac{2.48}{(114)(27)} = 0.00725$; $k = 0.113$; $j = 0.962$

$f_s = \frac{79.4(12)}{2.48(.962)(27)} = 14.8\ ksi < 20\ ksi$ $\therefore$ o.k.

$f_c = \frac{0.113(14.8)}{9(1-0.113)} = 0.21\ ksi < 1.2\ ksi$ $\therefore$ o.k.

$\therefore$ Cantilevered End Wall o.k.

Abutment Design

Check Caisson Supporting Cantilevered End Wall

Earth Load = $(0.966^k/LF \times 13'/2 + 3') + (2.38^k/ft)(5') = 21^k$ $\begin{cases} E_L = 20.3^k \\ E_T = 5.7^k \end{cases}$

Temp = 10.8^k

$(T+E) = [(10.8 + 20.3)^2 + (5.7)^2]^{1/2} = 31.6^k$

DM-7: @ $Z/T = 1.3$ $M_D = 0.77(31.6)(50.8''/12''/1) = 103^{k\text{-}ft}$

Design $M_D = \frac{103^{k\text{-}ft}}{(1.25)(0.35)} = 235^{k\text{-}ft} \approx M_x = 228.33$ (See CIRCON OUTPUT)

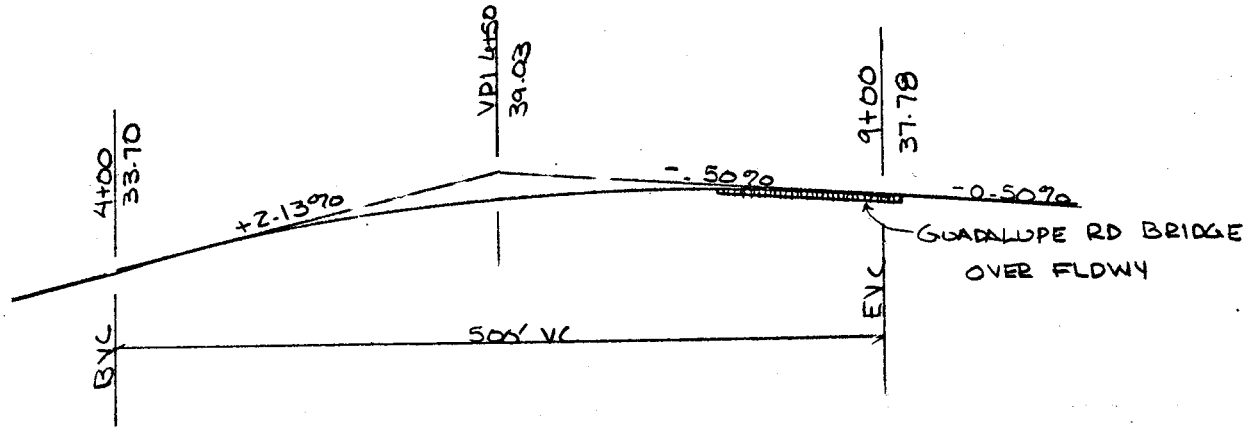
∴ Caisson O.K

Bridge Geometry

Geometry

Deck Elevations

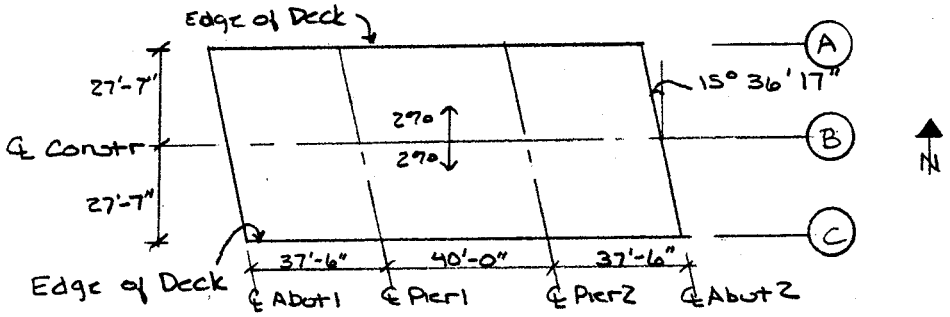
Vertical Curve Ref: The WLB Group Roadway Plans



Profile Grade @ ℄ Constr

@ ℄ Constr:

Location	Station	Elevation
℄ Abut 1	7+92.46	38.01
℄ Pier 1	8+29.96	38.00
℄ Pier 2	8+69.96	37.90
℄ Abut 2	9+07.46	37.74



Line A: Sta offset = -27.58' (tan 15.60°) = -7.70'
Vert. Offset = -27.58' (.02) = -0.55'

Line C: Sta. Offset = +7.70'
Vert. Offset = -0.55'

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
A	7+84.76	7+94.13	8+03.51	8+12.88	8+22.26
B	7+92.46	8+01.84	8+11.21	8+20.58	8+29.96
C	8+00.16	8+09.54	8+18.91	8+28.28	8+37.66

SPAN 2					
LINE	℄ PIER 1 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ PIER 2 STA
A	8+22.26	8+32.26	8+42.26	8+52.26	8+62.26
B	8+29.96	8+39.96	8+49.96	8+59.96	8+69.96
C	8+37.66	8+47.66	8+57.66	8+67.66	8+77.66

SPAN 3					
LINE	℄ PIER 2 STA	1/4 PT STA	1/2 PT STA	3/4 PT STA	℄ ABUT 2 STA
A	8+62.26	8+71.63	8+81.01	8+90.38	8+99.76
B	8+69.96	8+79.34	8+88.71	8+98.09	9+07.46
C	8+77.66	8+87.04	8+96.41	9+05.79	9+15.16

- Elevations

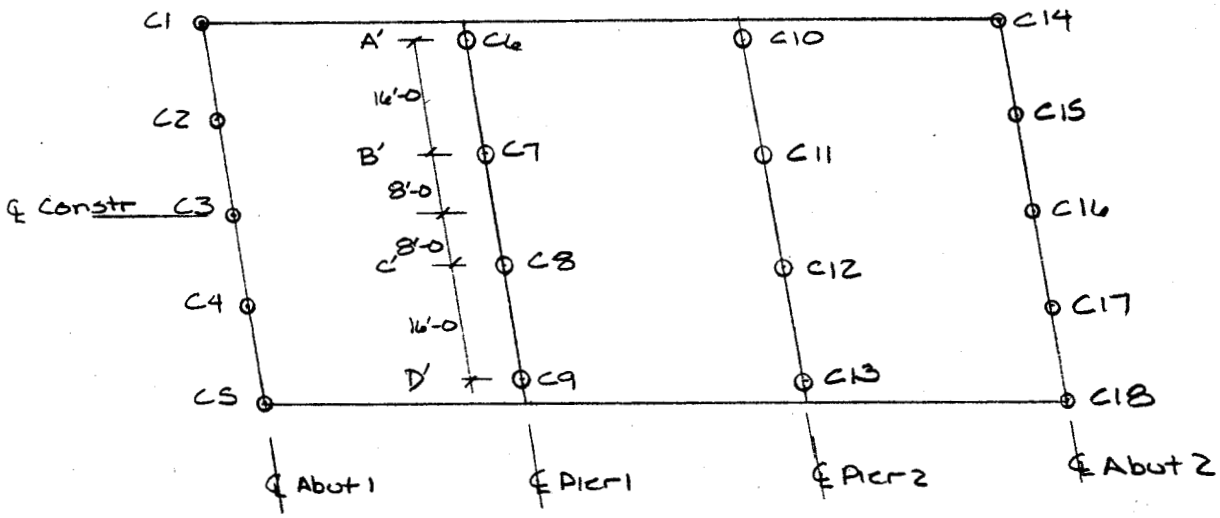
SPAN 1					
LINE	℄ ABUT 1 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 1 ELEV
A	37.45	37.46	37.46	37.46	37.45
B	38.01	38.01	38.01	38.01	38.00
C	37.46	37.46	37.46	37.45	37.43

SPAN 2					
LINE	℄ PIER 1 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ PIER 2 ELEV
A	37.45	37.44	37.42	37.40	37.37
B	38.00	37.98	37.96	37.93	37.90
C	37.43	37.41	37.39	37.36	37.32

SPAN 3					
LINE	℄ PIER 2 ELEV	1/4 PT ELEV	1/2 PT ELEV	3/4 PT ELEV	℄ ABUT 2 ELEV
A	37.37	37.34	37.31	37.27	37.22
B	37.90	37.87	37.83	37.78	37.74
C	37.32	37.28	37.24	37.19	37.15

Geometry

Caisson Cut-off Elevations



Abut 1 C1-C5

B.O. Abutment Cap Elev. = 1328.0'

Caisson Cutoff Elev. = 1328' + 3" = 1328.25'

Abut 2 C14-C18

B.O. Abutment Cap Elev. = 1328.0'

Caisson Cutoff Elev. = 1328' + 3" = 1328.25'

Pier cut-offs:

GRID LINE	STATION OFFSET	VERT OFFSET
A'	-6.46'	-2.80'
B'	-2.15'	-2.49'
C'	+2.15'	-2.49'
D'	+6.46'	-2.80'

Geometry

Pier 1:

CAISSON	STA.	CUT-OFF ELEV
C6	8+23.50	35.21
C7	8+27.81	35.51
C8	8+32.11	35.51
C9	8+36.42	35.19

Pier 2:

CAISSON	STA.	CUT-OFF ELEV.
C10	8+63.50	35.13
C11	8+67.81	35.42
C12	8+72.11	35.41
C13	8+76.42	35.08

Min. Freeboard

Min. Top of Deck Pier Elevation = 1337.32' @ Pier 2

1337.32 − 2.25' = 1335.07' ← Elev. @ Bott. of Pier Cap

Bott. of Proposed Streambed = 1325.0'

Depth of flow = 7.2'

Min Freeboard = 1335.07' − (1325' + 7.2') = 2.87' > 1.5' ∴ O.K.

QUANTITIES

Quantities

Superstructure

Concrete, $f'_c = 4000$ psi

Slab- $(115' - 2(\overset{\text{abut}}{1.3'}))(55.17')(19.5/12)/27\ ft^3/yd^3$ = 373 cu yd

Piers 1:2 - $2[57.28'(6')(7.5/12)]/27\ ft^3/yd^3$ = 16 cu. yd.

Abut. 1. - $57.28'(2.5')\left(\frac{2(38.01)+37.46+37.45}{4} - 35\right)/27$ = 15 cu. yd.

Abut 2 - $57.28'(2.5')\left(\frac{2(37.74)+37.15+37.22}{4} - 35\right)/27$ = 13 cu. yd.

Edge Bms. - $2(116')[(17")(3")+(14.5")(7")]/144\ in^2/ft^2/27\ ft^3/yd^3$ = 9 cu yd.

426 cu. yd.

Concrete $f'_c = 3000$ psi

Barriers - $2(116')[15"(9")+3"(11")]/144(27)$ = 10 cu yd.

Reinforcing Steel-Grade 60

Slab-

b1	$(105)(40')(3.40\#/ft)$	= 14,280#
b2	$(105)(53.5')(5.313\#/ft)$	= 29,846#
b3	$(105)(51.5')(3.40\#/ft)$	= 18,386#
16 #10 cont	$16(115' + 2(2') + 2(77/12))(4.303\#/ft)$	= 9076#
		71,588#

Piers 1:2

Top Bars	$2(10)(57')(4.303\#/ft)$	= 4,905#
Bott Bars	$2(10)(57')(2.670\#/ft)$	= 3044#
		7949#

Quantities

Superstructure (cont)

Reinforcing Steel - Grade 40

Slab-

b4	105(30')(1.502#/ft)	=	4731#
b5	53(22')(1.502#/ft)	=	1751#
#5 Tran. Bott	[(2(43)+47)(57')]1.043#/ft	=	3910#
#4 Tran Top	24(78)0.668#/ft	=	1250#
			15642#

Piers 1 & 2-

#5	- 2(114)(9')(1.043#/ft)	=	2140#
#5	-2(114)(7')(1.043#/ft)	=	1665#
			3805#

Edge of Slab & Barriers-

#6 cont	- 2(115'+2(2.5'))1.502#/ft	=	361#
#5 cont	- 2(115'+2(2'))1.043#/ft	=	248#
#4 cont	- 14(115'+2(1.5'))0.668#/ft	=	1164#
#4	- 2(116)(5')(.668#/ft)	=	775#
#4	- 2(155)(6.5')(.668#/ft)	=	1346#
#4	- 2(155)(3.5')(.668#/ft)	=	725#
#4	- 2(78)(2')(.668#/ft)	=	208#
			4767#

Abuts 1 & 2

#5 cont	- 10(57')(1.043#/ft)	=	595#
#4	- 2(58)(10')(0.668#/ft)	=	775#
#4	- 2(58)(2')(0.668#/ft)	=	155#
			1525#

Total Reinforcing (Superstructure) = 105,276#

Quantities

Abutment 1

Drilled Shafts -

5(1328.25' - 1300') = 141.25 L.F.

Concrete, f'_c = 3000 psi

Shafts - $\pi(1')^2(141.25)/27$ cu ft/cu yd = 17 cu yd

Cap - $[57.28'(2.5')(7') + 10.72'(2.5')(9.41')]/27$ cu ft/cu yd = 47 cu yd

64 cu yd.

Reinforcing Steel - Grade 60

Shafts

- #7 Verts - 5(8)(32')(2.044 #/ft) = 2616#

Cap

#7 Bott - 3(68' + 2') 2.044 #/ft = 429#

#7 Top - 3(68' + 2.5') 2.044 #/ft = 432#

861#

Reinforcing Steel - Grade 40

Shafts

- 1/2" Φ Spiral Ties: 5[54π(1.58') + 5(π(1.58')/2 + .5') + 2(.5')](.668 #/ft) = 948#

- #4 Ties : 5[10(π(1.5') + 1.5')](0.668 #/ft) = 207#

1155#

Cap

- 10 #5 : 10(68' + 2')(1.043 #/ft) = 730#
- #5 - 7 EA SIDE : 7(10')(1.043 #/ft) = 73#
- #5 [] : 57(16.5')(1.043 #/ft) = 981#
- #5 ⌐ : 69(3')(1.043 #/ft) = 216#
- #5 [] : 12(22')(1.043 #/ft) = 275#
- #5 - 6 Vert EA End : 12(9')(1.043 #/ft) = 113#
- #4 ⊏ : 36(5.5')(0.668 #/ft) = 133#
- #4 Dowels : 58(4')(0.668 #/ft) = 155#

2676#

Total Reinf. - Abut 1 = 7308#

Quantities

Abutment 2 - See Abut 1

Drilled Shafts - 141.25 L.F.

Concrete, $f'_c = 3000$ psi

Shafts = 17 cu yd

Diaph - $[57.28'(2.5')(7') + 10.72'(2.5')(9.13')]/27$ = 46 cu yd

63 cu yd

Reinforcing Steel - Grade 60

Shafts - 2616 #

Diaph - 861 #

Reinforcing Steel - Grade 40

Shafts - 1155 #

Diaph - 2676 #

Total Reinf - Abut 2 - 7308 #

Pier 1

Drilled Shafts

L.F. = 35.21 + 35.51 + 35.51 + 35.19 = 141.42 L.F.

Concrete, $f'_c = 3000$ psi

Shafts - $[\pi(1.25)^2(141.42 - 4(3')) + 4[.5\pi(2.5')^2 + 2.5\pi(1.875)^2]]/27$ = 29 cu yd

Reinforcing Steel - Grade 60

#11 Verts - $12[141.42' - 4(1.25')]5.313$ #/ft = 8698 #

#8 Pin Reinf - $4(6)(6')$ 2.670 #/ft = 384 #

9082 #

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checked by ME 2.13.85
date 1/21/85 sh. 61 of

Quantities

Pier 1 (cont)

Reinforcing Steel - Grade 40

½" ø Spiral Ties - 4 [69π(2.1') + 8(π(2.1')/2 + .5) + 2(.5)] .668 #/ft = 1300#

Total Reinf. - Pier 1 - 10,382#

Pier 2

Drilled Shafts

L.F. = 35.13 + 35.42 + 35.41 + 35.08 = 141.04'

Concrete, f'_c = 3000 psi

Shafts - [π(1.25')²(141.04' - 4(3')) + 4[.5π(2.5)² + 2.5π(1.875)²]]/27 = 29 cu yd

Reinforcing Steel - Grade 60

#11 Verts - 12 [141.04' - 4(1.25')] 5.313 #/ft = 8673#

#8 Pin Reinf = 384#

9057#

Reinforcing Steel - Grade 40

½" ø Spiral Ties - 1300#

Total Reinf - Pier 2 - 10,357#

Quantities

Approach Slabs

Concrete, $f'_c = 3000$ psi

$2(55.17')(15')(1')/27$ cu ft/cu yd = 62 cu yd

Reinforcing Steel - Grade 60

#8 @ 9" o.c. - $2(75)(15')(2.67\#/ft) = 6008\#$

Reinforcing Steel - Grade 40

#5 @ 12" o.c. - $2(15)(57')1.043\#/ft = 1784\#$

#4 @ 18" o.c - $2(11)(57')0.6668\#/ft = 838\#$

#4 @ 18" o.c - $2(38)(15')0.6668\#/ft = 762\#$

#4 ⊓ - $2(38)(5.5')0.6668\#/ft = 279\#$

3663#

Total Reinforcing - Approach Slabs - 9671#

6" Base Course (ABC)

$2(55.17')(14.5')(.5')/27$ cu ft/cu yd = 30 cu yd

Excavation & Backfill

Existing Elev. @ Profile Grade ℄: Abutment 1 = 1334.0'

Abutment 2 = 1336.5'

Abut 1

Excavation = $(68' + 3')(6')(2.5' + 3')/27$ cu ft/cu yd = 87 cu yd

Backfill = $[68(6')(3') + 3'(6')(5.5')]/27$ cu ft/cu yd = 49 cu yd

Abut 2

Excavation = $(68' + 3')(8.5')(2.5' + 3')/27$ cu ft/cu yd = 123 cu yd

Backfill = $[68'(8.5')(3') + 3(8.5')(5.5')]/27$ cu ft/cu yd = 69 cu yd.

Approach Slabs

Backfill - $[(55.17')\cos 15.6°(15' + 2' + 2.5')(2.5')]/27$ = 96 cu yd

2.5'

Quantities

ITEM	APPROX. QUANTITIES							
	3000 CONC (CLASS A)	4000 CONC (CLASS AA)	REINF STEEL	24" Ø DRILLED SHAFTS	30" Ø DRILLED SHAFTS	Agg Base Course	STRUCT EXCAVATION	STRUCT. BACKFILL
	CY	CY	LBS	LF	LF	CY	CY	CY
ABUT 1	64		7310	141.25			87	49
PIER 1	29		10380		141.42			
PIER 2	29		10360		141.04			
ABUT 2	63		7310	141.25			123	69
SUPER-STRUCTURE	10	426	105280					
APPROACH SLABS	62		9670			30		96
TOTAL	257	426	150,310	282.50	282.46	30	210	214

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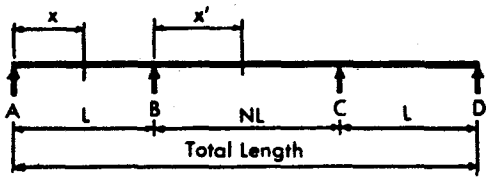
Appendix

TABLE 3.1

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

N=1.1



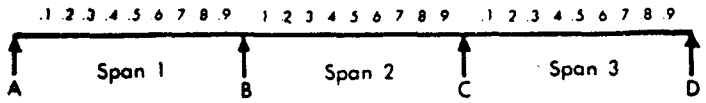
Total Length Ft.	Exter. Span Length "L"	Max. Reaction Kips.		Max. Shear Kips.		Max. Moment Kip.-ft.			Impact Coefficient				Dist.-Ft.	
		at A	at B	in AB at B	in BC at B or C	in AB at X	at B	in BC at X'	I	II	III	IV	X	X'
90	29.0	45.7	62.2	-52.0	51.1	219.1	-185.2	195.6	.300	.300	.300	.269	10.3	14.5
105	33.9	48.8	64.6	-55.7	54.4	275.9	-219.4	250.4	.300	.300	.300	.255	12.2	17.1
120	38.7	51.4	66.2	-58.3	56.9	338.2	-254.0	308.0	.300	.300	.298	.242	15.6	19.8
135	43.5	53.4	67.3	-60.2	58.8	406.2	-302.8	367.4	.297	.293	.289	.231	17.5	22.5
150	48.4	55.2	68.2	-61.7	60.4	475.0	-355.4	428.0	.288	.284	.281	.221	19.5	25.1
165	53.2	56.6	68.8	-62.9	61.6	544.3	-411.8	489.7	.281	.276	.272	.211	21.6	27.8
180	58.1	57.8	72.5	-63.8	62.7	614.2	-472.1	552.0	.273	.269	.265	.202	23.6	30.4
195	62.9	58.9	76.3	-64.6	63.5	684.3	-536.1	614.9	.266	.262	.257	.194	25.6	33.1
210	67.7	59.8	80.2	-65.3	64.2	754.7	-603.9	678.3	.259	.255	.251	.187	27.7	35.8
225	72.6	60.6	84.1	-65.8	64.8	825.3	-675.5	741.9	.253	.248	.244	.180	29.7	38.4
240	77.4	61.2	87.9	-66.3	65.4	896.1	-750.9	805.9	.247	.242	.238	.174	31.8	41.1
255	82.3	61.9	91.8	-66.7	65.8	967.0	-830.1	870.1	.241	.237	.232	.168	33.8	43.7
270	87.1	62.4	95.7	-67.1	66.2	1038.1	-913.2	934.5	.236	.231	.226	.162	35.9	46.4
285	91.9	62.9	99.5	-67.4	66.6	1109.2	-999.9	999.1	.230	.226	.221	.157	38.0	49.1
300	96.8	63.4	103.4	-67.7	66.9	1180.5	-1090.5	1063.8	.225	.221	.216	.152	40.0	51.7
315	101.6	63.8	107.3	-67.9	67.2	1251.8	-1184.9	1128.6	.221	.216	.211	.148	42.1	54.4
330	106.5	64.1	111.2	-68.7	68.5	1323.1	-1283.1	1193.6	.216	.211	.207	.143	44.2	57.1
345	111.3	64.5	115.0	-70.6	70.4	1394.5	-1385.1	1258.6	.212	.207	.202	.139	46.2	59.7
360	116.1	64.8	118.9	-72.6	72.3	1466.0	-1490.9	1323.7	.207	.202	.198	.136	48.3	62.4
Impact		I	IV	I	III	I	II	III						
Dead Load		.3900 x wL	1.1600 x wL	-.6100 x wL	.5500 x wL	.0761 x wL^2	-.1100 x wL^2	.0413 x wL^2					.3900 L	.5500 x L

From 'Moments, Shears, & Reactions for Continuous Highway Bridges', AISC 1966

Guadalupe Rd. Br. 83-58
RR

Unit load at		MOMENTS/PL																				
		SPAN 1											SPAN 2									
		A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
SPAN 1	A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	.1	0	.0874	.0747	.0621	.0494	.0368	.0242	.0115	-.0011	-.0138	-.0264	-.0231	-.0198	-.0165	-.0132	-.0099	-.0066	-.0033	.0000	.0033	.0066
	.2	0	.0749	.1498	.1246	.0995	.0744	.0493	.0242	-.0010	-.0261	-.0512	-.0448	-.0384	-.0320	-.0256	-.0192	-.0128	-.0064	.0000	.0064	.0128
	.3	0	.0627	.1254	.1882	.1509	.1136	.0763	.0390	.0018	-.0355	-.0728	-.0637	-.0546	-.0455	-.0364	-.0273	-.0182	-.0091	.0000	.0091	.0182
	.4	0	.0510	.1021	.1531	.2042	.1552	.1062	.0573	.0083	-.0406	-.0896	-.0784	-.0672	-.0560	-.0448	-.0336	-.0224	-.0112	.0000	.0112	.0224
	.5	0	.0400	.0800	.1200	.1600	.2000	.1400	.0800	.0200	-.0400	-.1000	-.0875	-.0750	-.0625	-.0500	-.0375	-.0250	-.0125	.0000	.0125	.0250
	.6	0	.0298	.0595	.0893	.1190	.1488	.1786	.1084	.0381	-.0322	-.1024	-.0896	-.0768	-.0640	-.0512	-.0384	-.0256	-.0128	.0000	.0128	.0256
	.7	0	.0205	.0410	.0614	.0819	.1024	.1229	.1434	.0638	-.0157	-.0952	-.0833	-.0714	-.0595	-.0476	-.0357	-.0238	-.0119	.0000	.0119	.0238
	.8	0	.0123	.0246	.0370	.0493	.0616	.0739	.0862	.0986	.0109	-.0768	-.0672	-.0576	-.0480	-.0384	-.0288	-.0192	-.0096	.0000	.0096	.0192
	.9	0	.0054	.0108	.0163	.0218	.0272	.0326	.0381	.0435	.0490	-.0456	-.0399	-.0342	-.0285	-.0228	-.0171	-.0114	-.0057	.0000	.0057	.0114
	B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SPAN 2	.1	0	-.0039	-.0078	-.0117	-.0156	-.0195	-.0234	-.0273	-.0312	-.0351	-.0390	.0534	.0458	.0382	.0306	.0230	.0154	.0073	.0002	-.0074	-.0150
	.2	0	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512	-.0576	-.0640	.0192	.1024	.0856	.0688	.0520	.0352	.0184	.0016	-.0152	-.0320
	.3	0	-.0077	-.0154	-.0231	-.0308	-.0385	-.0462	-.0539	-.0616	-.0691	-.0770	-.0042	.0686	.1414	.1142	.0870	.0598	.0326	.0054	-.0218	-.0490
	.4	0	-.0080	-.0160	-.0240	-.0320	-.0400	-.0480	-.0560	-.0640	-.0720	-.0800	-.0184	.0432	.1048	.1664	.1280	.0896	.0512	.0128	-.0256	-.0640
	.5	0	-.0075	-.0150	-.0225	-.0300	-.0375	-.0450	-.0525	-.0600	-.0675	-.0750	-.0250	.0250	.0750	.1250	.1750	.1250	.0750	.0250	-.0250	-.0750
	.6	0	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512	-.0576	-.0640	-.0256	.0128	.0512	.0896	.1280	.1664	.1048	.0432	-.0184	-.0800
	.7	0	-.0049	-.0098	-.0147	-.0196	-.0245	-.0294	-.0343	-.0392	-.0441	-.0490	-.0218	.0054	.0326	.0598	.0870	.1142	.1414	.0686	-.0042	-.0770
	.8	0	-.0032	-.0064	-.0096	-.0128	-.0160	-.0192	-.0224	-.0256	-.0288	-.0320	-.0152	.0016	.0184	.0352	.0520	.0688	.0856	.1024	.0192	-.0640
	.9	0	-.0015	-.0030	-.0045	-.0060	-.0075	-.0090	-.0105	-.0120	-.0135	-.0150	-.0074	.0002	.0078	.0154	.0230	.0306	.0382	.0458	.0534	-.0390
	C	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SPAN 3	.1	0	.0011	.0023	.0034	.0046	.0057	.0068	.0080	.0091	.0103	.0114	.0057	.0000	-.0057	-.0114	-.0171	-.0228	-.0285	-.0342	-.0399	-.0456
	.2	0	.0019	.0038	.0058	.0077	.0096	.0115	.0134	.0154	.0173	.0192	.0096	.0000	-.0096	-.0192	-.0288	-.0384	-.0480	-.0576	-.0672	-.0768
	.3	0	.0024	.0048	.0071	.0095	.0119	.0143	.0167	.0190	.0214	.0238	.0119	.0000	-.0119	-.0238	-.0357	-.0476	-.0595	-.0714	-.0833	-.0952
	.4	0	.0026	.0051	.0077	.0102	.0128	.0154	.0179	.0205	.0230	.0256	.0128	.0000	-.0128	-.0256	-.0384	-.0512	-.0640	-.0768	-.0896	-.1024
	.5	0	.0025	.0050	.0075	.0100	.0125	.0150	.0175	.0200	.0225	.0250	.0125	.0000	-.0125	-.0250	-.0375	-.0500	-.0625	-.0750	-.0875	-.1000
	.6	0	.0022	.0045	.0067	.0090	.0112	.0134	.0157	.0179	.0202	.0224	.0112	.0000	-.0112	-.0224	-.0336	-.0448	-.0560	-.0672	-.0784	-.0896
	.7	0	.0018	.0036	.0055	.0073	.0091	.0109	.0127	.0146	.0164	.0182	.0091	.0000	-.0091	-.0182	-.0273	-.0364	-.0455	-.0546	-.0637	-.0728
	.8	0	.0013	.0026	.0038	.0051	.0064	.0077	.0090	.0102	.0115	.0128	.0064	.0000	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512
	.9	0	.0007	.0013	.0020	.0026	.0033	.0040	.0046	.0053	.0059	.0066	.0033	.0000	-.0033	-.0066	-.0099	-.0132	-.0165	-.0198	-.0231	-.0264
	D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
+ Area		0	.0400	.0700	.0900	.1000	.1000	.0900	.0700	.0402	.0204	.0167	.0152	.0300	.0550	.0700	.0750	.0700	.0550	.0300	.0152	.0167
− Area		0	-.0050	-.0100	-.0150	-.0200	-.0250	-.0300	-.0350	-.0402	-.0654	-.1167	-.0702	-.0500	-.0500	-.0500	-.0500	-.0500	-.0500	-.0500	-.0702	-.1167
Total Area		0	.0350	.0600	.0750	.0800	.0750	.0600	.0350	.0000	-.0450	-.1000	-.0550	-.0200	.0050	.0200	.0250	.0200	.0050	-.0200	-.0550	-.1000

TABLE A3.0

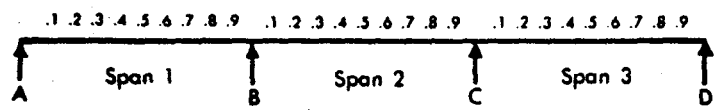


From 'Moments, Shears & Reactions for Continuous Highway Bridges', AISC 1966

Guadalupe Rd. Bridge 83-58
RR

Unit load at	MOMENTS/PL																				
	SPAN 1											SPAN 2									
	A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	.0875	.0749	.0624	.0499	.0373	.0248	.0123	-.0002	-.0128	-.0253	-.0221	-.0189	-.0157	-.0125	-.0093	-.0061	-.0030	.0002	.0034	.0066
.2	0	.0751	.1502	.1253	.1004	.0755	.0506	.0256	.0007	-.0242	-.0491	-.0429	-.0367	-.0305	-.0243	-.0181	-.0119	-.0057	.0005	.0067	.0129
.3	0	.0630	.1260	.1891	.1521	.1151	.0781	.0411	.0042	-.0328	-.0698	-.0610	-.0522	-.0434	-.0346	-.0258	-.0169	-.0081	.0007	.0095	.0183
.4	0	.0514	.1028	.1542	.2056	.1571	.1085	.0599	.0113	-.0373	-.0859	-.0751	-.0642	-.0534	-.0425	-.0317	-.0209	-.0100	.0008	.0117	.0225
.5	0	.0404	.0808	.1212	.1617	.2021	.1425	.0829	.0233	-.0363	-.0959	-.0838	-.0717	-.0596	-.0475	-.0354	-.0233	-.0112	.0009	.0130	.0251
.6	0	.0302	.0604	.0906	.1207	.1509	.1811	.1113	.0415	-.0283	-.0982	-.0858	-.0734	-.0610	-.0486	-.0362	-.0238	-.0115	.0009	.0133	.0257
.7	0	.0209	.0417	.0626	.0835	.1044	.1252	.1461	.0670	-.0121	-.0913	-.0797	-.0682	-.0567	-.0452	-.0337	-.0222	-.0106	.0009	.0124	.0239
.8	0	.0126	.0253	.0379	.0506	.0632	.0758	.0885	.1011	.0137	-.0736	-.0643	-.0550	-.0458	-.0365	-.0272	-.0179	-.0086	.0007	.0100	.0193
.9	0	.0056	.0113	.0169	.0225	.0281	.0338	.0394	.0450	.0507	-.0437	-.0382	-.0327	-.0272	-.0216	-.0161	-.0106	-.0051	.0004	.0059	.0114
B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	-.0045	-.0090	-.0135	-.0179	-.0224	-.0269	-.0314	-.0359	-.0404	-.0449	.0569	.0487	.0406	.0324	.0242	.0160	.0078	-.0004	-.0086	-.0168
.2	0	-.0074	-.0147	-.0221	-.0294	-.0368	-.0441	-.0515	-.0588	-.0662	-.0735	.0182	.1100	.0917	.0735	.0552	.0370	.0187	.0004	-.0178	-.0361
.3	0	-.0088	-.0177	-.0265	-.0353	-.0442	-.0530	-.0618	-.0706	-.0795	-.0883	-.0080	.0722	.1525	.1228	.0931	.0634	.0336	.0039	-.0258	-.0555
.4	0	-.0092	-.0183	-.0275	-.0366	-.0458	-.0549	-.0641	-.0732	-.0824	-.0916	-.0237	.0442	.1121	.1799	.1378	.0957	.0536	.0114	-.0307	-.0728
.5	0	-.0086	-.0171	-.0257	-.0342	-.0428	-.0514	-.0599	-.0685	-.0771	-.0856	-.0306	.0244	.0794	.1344	.1894	.1344	.0794	.0244	-.0306	-.0856
.6	0	-.0073	-.0146	-.0218	-.0291	-.0364	-.0437	-.0510	-.0583	-.0655	-.0728	-.0307	.0114	.0536	.0957	.1378	.1799	.1121	.0442	-.0237	-.0916
.7	0	-.0055	-.0111	-.0167	-.0222	-.0278	-.0333	-.0389	-.0444	-.0500	-.0555	-.0258	.0039	.0336	.0634	.0931	.1228	.1525	.0722	-.0080	-.0883
.8	0	-.0036	-.0072	-.0108	-.0144	-.0180	-.0216	-.0252	-.0288	-.0325	-.0361	-.0178	.0004	.0187	.0370	.0552	.0735	.0917	.1100	.0182	-.0735
.9	0	-.0017	-.0034	-.0050	-.0067	-.0084	-.0101	-.0117	-.0134	-.0151	-.0168	-.0086	-.0004	.0078	.0160	.0242	.0324	.0406	.0487	.0569	-.0449
C	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	.0011	.0023	.0034	.0046	.0057	.0068	.0080	.0091	.0103	.0114	.0059	.0004	-.0051	-.0106	-.0161	-.0216	-.0272	-.0327	-.0382	-.0437
.2	0	.0019	.0039	.0058	.0077	.0097	.0116	.0135	.0154	.0174	.0193	.0100	.0007	-.0086	-.0179	-.0272	-.0365	-.0458	-.0550	-.0643	-.0736
.3	0	.0024	.0048	.0072	.0096	.0120	.0143	.0167	.0191	.0215	.0239	.0124	.0009	-.0106	-.0222	-.0337	-.0452	-.0567	-.0682	-.0797	-.0913
.4	0	.0026	.0051	.0077	.0103	.0129	.0154	.0180	.0206	.0231	.0257	.0133	.0009	-.0115	-.0238	-.0362	-.0486	-.0610	-.0734	-.0858	-.0982
.5	0	.0025	.0050	.0075	.0100	.0126	.0151	.0176	.0201	.0226	.0251	.0130	.0009	-.0112	-.0233	-.0354	-.0475	-.0596	-.0717	-.0838	-.0959
.6	0	.0023	.0045	.0068	.0090	.0113	.0135	.0158	.0180	.0203	.0225	.0117	.0008	-.0100	-.0209	-.0317	-.0425	-.0534	-.0642	-.0751	-.0859
.7	0	.0018	.0037	.0055	.0073	.0092	.0110	.0128	.0146	.0165	.0183	.0095	.0007	-.0081	-.0169	-.0258	-.0346	-.0434	-.0522	-.0610	-.0698
.8	0	.0013	.0026	.0039	.0052	.0065	.0077	.0090	.0103	.0116	.0129	.0067	.0005	-.0057	-.0119	-.0181	-.0243	-.0305	-.0367	-.0429	-.0491
.9	0	.0007	.0013	.0020	.0026	.0033	.0040	.0046	.0053	.0059	.0066	.0034	.0002	-.0030	-.0061	-.0093	-.0125	-.0157	-.0189	-.0221	-.0253
D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Area	0	.0403	.0705	.0908	.1011	.1015	.0917	.0720	.0423	.0209	.0167	.0163	.0346	.0643	.0824	.0885	.0824	.0643	.0346	.0163	.0167
[illegible]	0	-.0063	-.0125	-.0188	-.0251	-.0315	-.0377	-.0440	-.0503	-.0749	-.1267	-.0718	-.0478	-.0472	-.0472	-.0472	-.0472	-.0472	-.0478	-.0718	-.1267
[illegible] Area	0	.0340	.0580	.0720	.0760	.0700	.0540	.0280	-.0080	-.0540	-.1100	-.0555	-.0132	.0171	.0352	.0413	.0352	.0171	-.0132	-.0555	-.1100

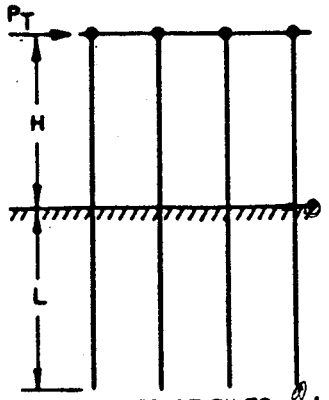
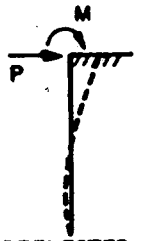
TABLE A3.1



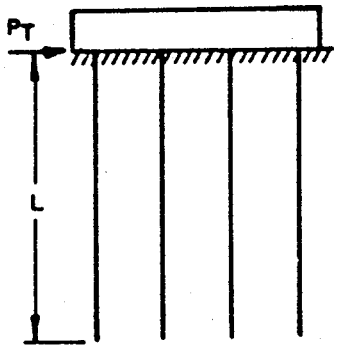
From 'Moments, Shears & Reactions for Continuous Highway Bridges,' AISC 1966

Guadalupe Rd. Bridge 83-58
RR

CASE I. FLEXIBLE CAP, ELEVATED POSITION

CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = \left(\frac{EI}{f}\right)^{1/5}$ 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

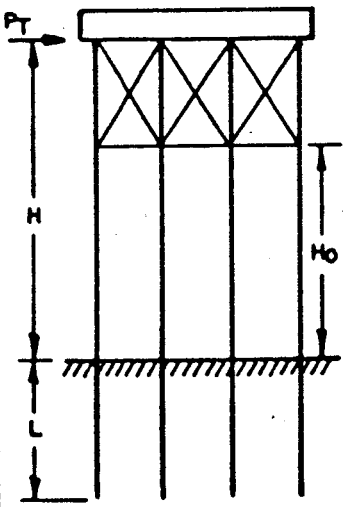
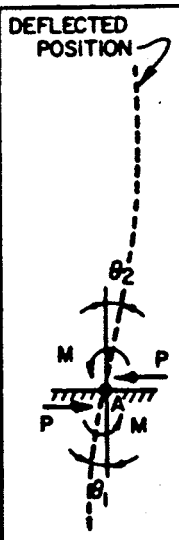
		1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta\left(\frac{PT^2}{EI}\right) + F_\theta\left(\frac{MT}{EI}\right)$ 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_o^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

NAVDOCKS DM-7

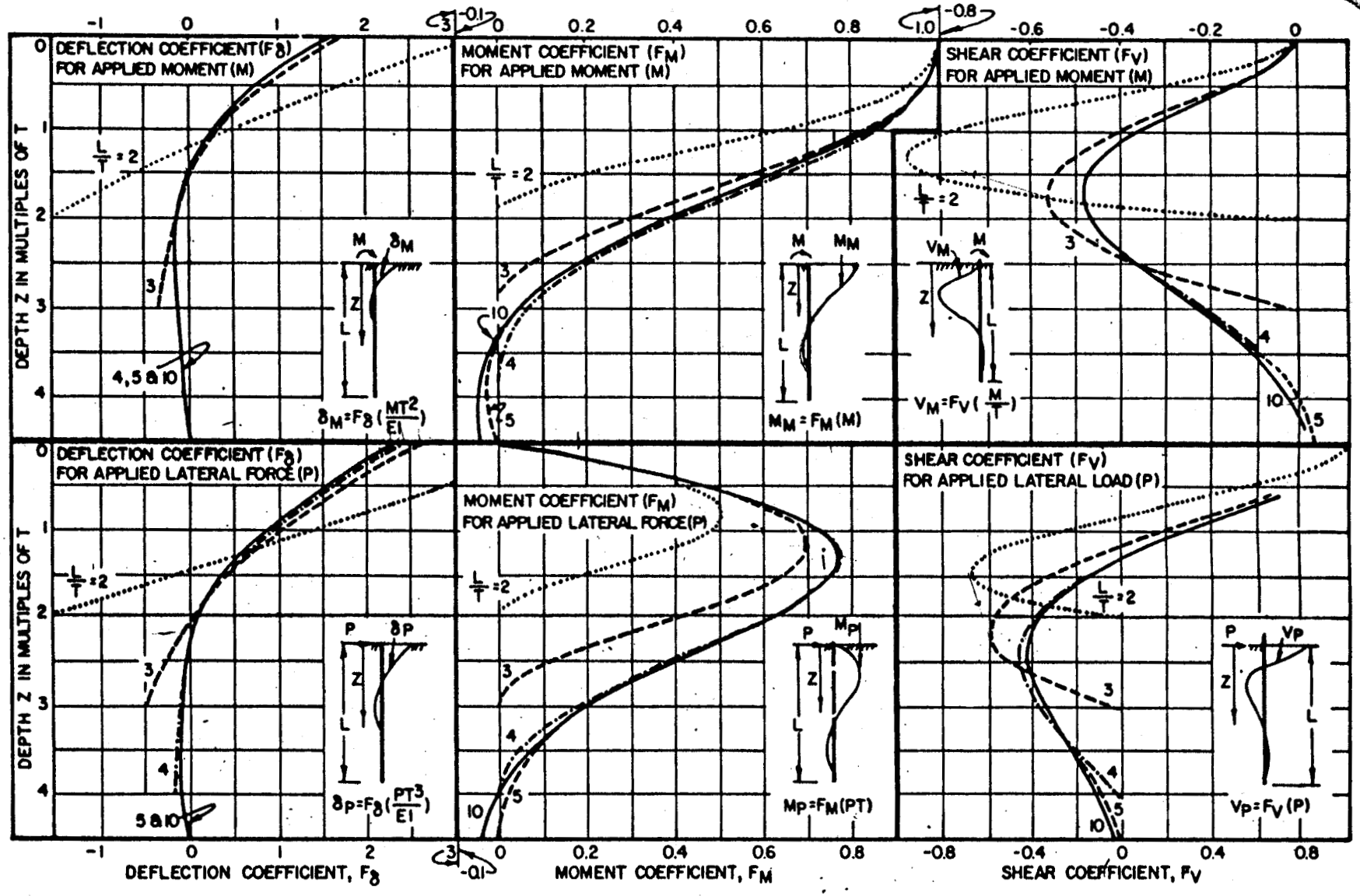


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

NAVDOCKS DM-7

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:	$\beta L > 2.25$	$\eta L > 2.0$
Example	12"φ Steel Pipe Pile 20' long $EI = 3\times10^7 \times 361.5$	
	$k_h = 80$ pci (stiff clay)	$n_h = 10$ pci
	$\beta = \sqrt[4]{\frac{80\times12}{4\times108.45\times10^8}} = 0.0120$	$\eta = \sqrt[5]{\frac{10}{3\times10^7\times361.5}} = 0.0156$
	$\beta L = 0.0120\times240 = 2.88 > 2.25$ Flexible Action	$\eta L = 0.0156\times240 = 3.74 > 2.0$ Flexible Action

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

II ULTIMATE ANALYSIS OF FLEXIBLE PILES

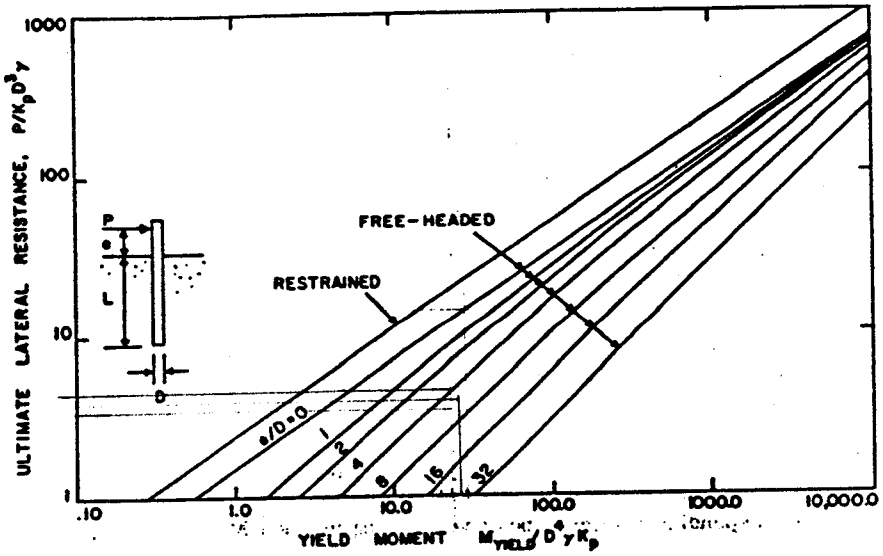
A. Analysis consists of computing the maximum bending moment producing failure in the pile section. Then the ultimate lateral load which produces the failure moment corresponds to the ultimate capacity of the pile. The margin of safety for a particular load is the ratio of the bending moment produced 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) analysis 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 ϕ = 30° K_p = 3
K_p = 3 γ = 100 pcf

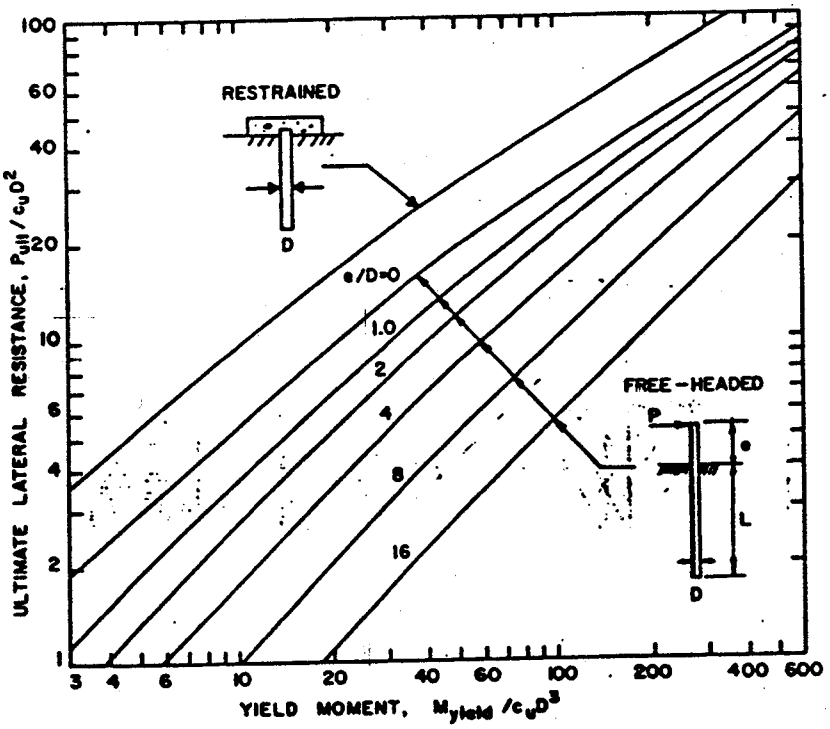
$M_{resisting} = \frac{fI}{c} = \frac{20^* \times 351.5}{6_{in} \times 12 \text{ in/ft}}$
= 100 ft-kips

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

$\frac{P}{K_p D^3 \gamma} = 80$ (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

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

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

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

.287

Brom's Method

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 in^4)

MAKE SURE YOU HAVE CONSISTENT UNITS

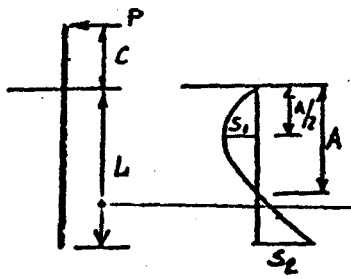
Type of Soil →	Clay	Sand
"Rigidity Factor"	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$ $\frac{\#}{in^3}$ in / $\frac{\#}{in^2} \times in^4$ = $\sqrt[4]{\frac{1}{in^4}}$ (PILE PROPERTIES)	$\eta = \sqrt[5]{\frac{n_h}{EI}}$ – PILE PROPERTIES
"Rigid action" if	$\beta L < 2.25$ (Ref 3)	$\eta L < 2.0$ (Ref 4)
Example	$k_h = 80$ 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 1956 Vol I p 11-21

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"



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

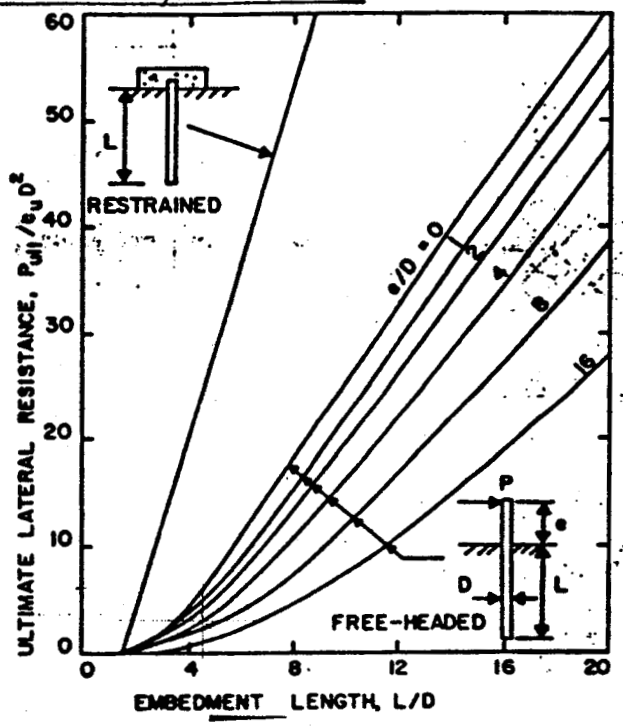
$$S_1 = \frac{1.5P(3L + 4C)^2}{L^2(4L + 6C)}$$ (usually governs design)

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

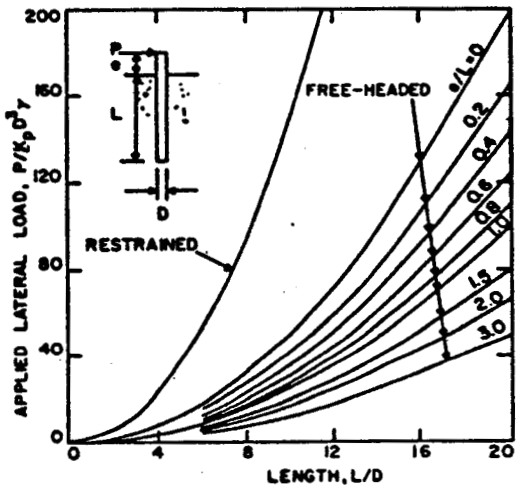
Additional equations for pile design available in Ref 1

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

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.

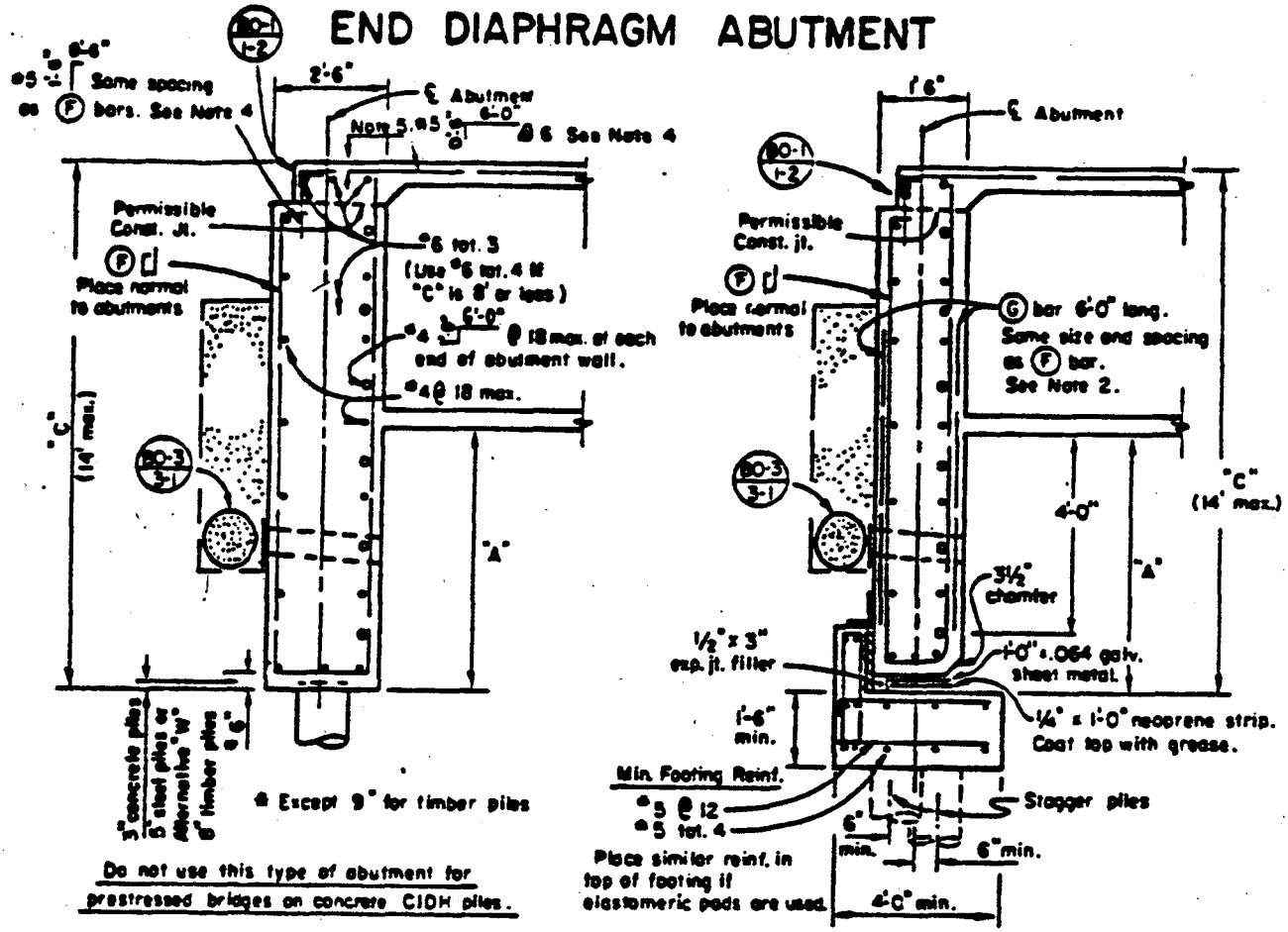
IV ELASTIC ANALYSIS OF RIGID PILES

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 Broms Method

END DIAPHRAGM ABUTMENT



(F) (G) BARS, #5 (F) 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 determine 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.

Guadalupe RD Br 83-58
RR