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Parsons Brinckerhoff Quade &, Douglas, Inc.

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12th Street Over ACDC

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Parsons Brinckerhoff **Computation Sheet**

Subject 12 Street OVER ACDC

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12th Street OVER ACDC

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Subject 12th st. OVER AGDC

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Date 10-26-89

DESIGN CRITERIA

1. AASHTO standard 13th Edition (1983) with 1984-1987 interims

2. AASHTO Guide Specifications for Seismic Design of Highway Bridges 1983 Edition with 1985 supplement

LOADINGS

A - DEAD LOAD

Allowance for future wearing surface 25 #/ft²

B - LIVE LOAD

1. AASHTO HS20-44
2. All other Loads as specified by AASTHO

DESIGN METHODS

1. prestressed concrete superstructure - service Load Design, with capacity check by Load factor Design

2. Drilled shafts Load factor Design

3. All others by service Load Design

MATERIALS

A - Concrete

1. superstructure concrete $f_c' = 5000$ PSI
2. Drilled shafts & Approach slabs $f_c' = 4000$ PSI
3. All others $f_c' = 3000$ PSI

Subject 12th st. OVER ACDC

B - Reinforced steel

GRADE 60 Reinforcing steel $f_s = 24000$ psi

GRADE 40 $f_s = 20,000$ psi

C - Prestressing steel

Low Relaxation $1\frac{1}{2}$" ϕ strands $f_s' = 270,000$ psi

DESIGN CRITERIA - DECK SLAB

1. Service Load Design
2. Allow 25 #/ft² for future wearing surface
3. $f_c' = 5000$ psi, design purpose only $f_c = 0.4 f_c' = 2000$ psi
4. $f_s = 20,000$ psi, $n = 8$
5. Concrete Cover = 1" MIN

Transverse Design

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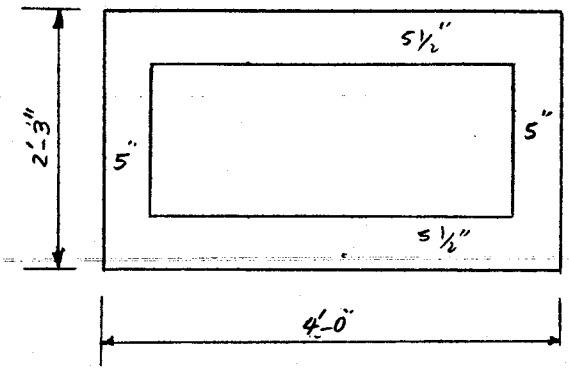
Subject 12th st. over ACDC

TRANSVERSE DESIGN



SPAN = 48" - 5" = $\frac{43}{12}$ = 3.583' (AASTHO 3.24.1.1)

LOADS

1. Dead Load = $\frac{7.5"}{12} \times 0.15$ k/ft³ + .025 = 0.1188 k/ft

2. Live Load = P_{20} = 16k (AASTHO 3.24.3.1)

3. Impact = $\frac{50}{L+125}$ USE 0.3 TRANSV. (AASTHO 3.8.2)

SERVICE MOMENTS

* see MOMENT DIST. by ALI 9-8-89

$\overset{+}{M}_D = (.808)\,w = .808(.1188) = .096^{k-'}$

$\overset{-}{M}_D = (.796)\,w = .796(.1188) = .095^{k-'}$

	AASHTO Moment Positive	negative
Simple	.1745P	0
intermediate	.15P	.103P
Fixed	.14P	.14P

$M_{LL} = \left(\frac{S+2}{32}\right)P = 0.1745\,P$ (NO IMPACT), (AASTHO 3.24.3.1)

MOD. LL MOMENT by MOMENT DIST

$\overset{+}{M} = \frac{PL}{4}$ (simple support) = 0.8958 P

$\overset{+}{M} = .566\,P$ (MOMENT DIST.)

$\overset{+}{M}_{LL} = \frac{.566P}{.8958P} * 0.1745 \times 16 \times \overset{\text{IMPACT}}{1.3} = 2.29^{k-'}$

$\overset{+}{M}_{AASHTO(LL)} = .15(1.3)(16) = 3.12^{k-'}$

$\overset{-}{M}_{LL} = \frac{.33P}{.556P} * 0.1745 \times 16 \times 1.3 = 2.15^{k-'}$

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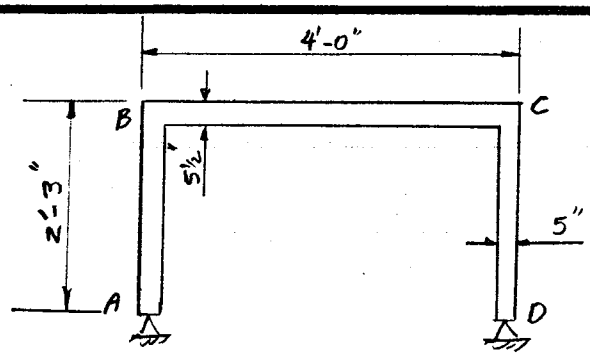
Subject 12th street over ACDC

MOMENT DISTURBUTION FOR SLAB DESIGN



MEMBER 1: $I = \frac{bh^3}{12} = \frac{12(5)^3}{12} = 125 \text{ in}^4$ $L_1 = 27 - 5.5 = 21.5"$ $I/L_1 = 5.81$ $DF_1 = \frac{5.81}{5.81 + 3.87} = 0.6 = BA$

MEMBER 2: $I = \frac{12(5.5)^3}{12} = 166.375 \text{ in}^4$ $L_2 = 48 - 5 = 43"$ $I/L_2 = 3.87$ $DF_2 = 1 - 0.6 = 0.4 = BC$

FEM FOR UNIFORM LOAD W $FEM = \frac{WL^2}{12} = W\frac{(3.583)^2}{12} = W(1.07)^{k-'}$

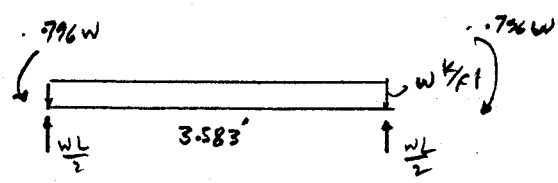
FEM FOR concentrated Load P $FEM = \frac{PL}{8} = P\frac{(3.583)}{8} = P(0.448)^{k-'}$

UNIFORM LOAD

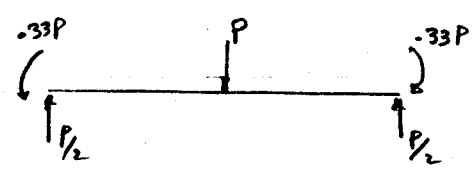
	BA	BC	CB	CD
DF	-0.6	-0.4	-.4	
FEM	0	1.07	-1.07	
	-0.642	-.428	.428	
		.214	-.214	
	-.128	-.0856	.0856	
		.0428	-.0428	
	-.0257	-.0171	.0171	
Σ	-.796W	.796W	-.79P	

Concentrated Load

	BA	BC	CB	CD
DF	-0.6	-.4	-.4	
FEM	0	.448	-.448	
	-.268	-.1792	.1792	
		.0896	-.0896	
	-.0538	-.0358	.0358	
		.0179	-.0179	
	-.011	-.007		
Σ	-.332P	.332P	-.33P	



$\overset{+}{M}_{@center} = \frac{WL}{2}\left(\frac{L}{2}\right) - .796W - \frac{WL}{2}\left(\frac{L}{4}\right) = 0.808W^{k-'}$



$\overset{+}{M}_{@center} = \frac{P}{2}\left(\frac{L}{2}\right) - .33P = .566P^{k-'}$

Subject 12TH street ACDC

TRANSVERSE DESIGN

$d = 5\frac{1}{2}'' - 1'' - \frac{.625}{2} = 4.18''$

$f_c' = 5000$ psi, $f_s = 20{,}000$ psi $\quad f_c = 2000$ psi $\quad k = \frac{2000}{2000 + \frac{24000}{7}} = 0.368 \quad j = 0.877$

$\overset{+}{M}_{TOTAL} = \overset{+}{M}_D + \overset{+}{M}_L = 0.12 + 3.120 = 3.24^{k-'}$ AASHTO moment is larger than the estimated moment by moment distribution

$\overset{+}{A}_s = \frac{M}{f_s j d} = \frac{3.24 \times 12}{20(.877)(4.18)} = 0.53$ in²/ft $\quad$ #5 @ 6" = 0.61 in² (BOT STEEL) GRADE 40

$f_c = \frac{2M}{bd^2kj} = \frac{2 \times 3.24 \times 12000}{12(4.18)^2(.877)(.368)} = 1149$ psi < 2000 psi ok.

$f_s = \frac{M}{A_s j d} = \frac{3.24 \times 12}{.61(.877)(4.18)} = 17.7$ ksi < 20 ksi ok

$\bar{M}_{TOTAL} = .095 + 2.15 = 2.245^{k-'}$

$\bar{A}_s = \frac{2.245 \times 12}{20(.877)(4.18)} = 0.37$ in²/ft $\quad$ #4 @ 6" = 0.39 in² (TOP STEEL) GRADE 40

#4 @ 7" GRADE 60

$f_c = 796$ psi < 2000 psi ok

$f_s = 16.$ ksi < 20 ksi ok

MINIMUM REINFORCEMENT AASTHO 8.17.1

$1.2 M_{cr} = 1.2 f_r I_g / y_t = 1.2 \times 7.5 \frac{\sqrt{5000}}{12000} \times 12 \frac{(5.5)^2}{6} = 3.208^{k-'}$

$M_u = 1.3(0.096 + \frac{5}{3} \times 2.29) = 5.08^{k-'} > 1.2 M_{cr}$ ok $\quad a = \frac{A_s f_y}{.85 f_c' b} = \frac{.39(40)}{.85(5)(12)} = .30$

$M_s = \frac{f_s A_s j d}{12} = \frac{20(.39)(.877)(4.18)}{12} = 2.383^{k-'}$ $\quad M_n = A_s f_y (d - \frac{a}{2}) \times 0.9$

$M_n = 0.9 \times \frac{.39(40)}{12}(4.18 - \frac{.3}{2}) = 5.40^{k-'} > 1.2 M_{cr}$ OK

$M_c = \frac{f_c b d^2 k j}{2 \times 12} = \frac{2(12)(4.18)^2(.877)(.368)}{24} = 5.64^{k-'} > 1.2 M_{cr}$ ok

Subject 12th st. over ACDC
Transverse Design

TRANSVERSE DESIGN

Bottom Slab Reinforcement (AASHTO 8.17.2.3), (AASHTO, 9.24)

Parallel to girder 0.4% of flange area

$A_{s_{para}} = \frac{0.4}{100}(5.5")(12" \times 3.583) = 0.946 \text{ in}^2$ 5#4 OR 3#5 @ BOT. SLAB

NOT REQ. Due to exist. of P/S cables.

Transverse to girder 0.3% of Flange area

$A_{s_T} = \frac{0.3}{100}(5.5)(12") = 0.198 \text{ in}^2/\text{ft}$ #4@12" (PT)

$A_{s_T} = \frac{0.5}{100}(5.5)(12") = 0.33 \text{ in}^2/\text{ft}$ #4@7" (REINF.)

TEMPERATURE & shrinkage AASHTO (8.20)

$\frac{1}{8}$" in^2/ft #4@12" = 0.2 in^2/ft NOT REQ.

Distribution Reinforcement (AASHTO 3.24.10.2)

$\text{Percentage} = \frac{220}{\sqrt{3.583}} = 116$ max 0.67

$A_s = 0.62(0.67) = 0.41 \text{ in}^2/\text{ft}$

#5 REQ. @ Mid half $= \frac{3.583}{2} \times \frac{0.41}{0.31} = 2.3$ USE 2#5

#5 REQ @ QUART'S $= \frac{2}{2} = 1$ USE 1#5

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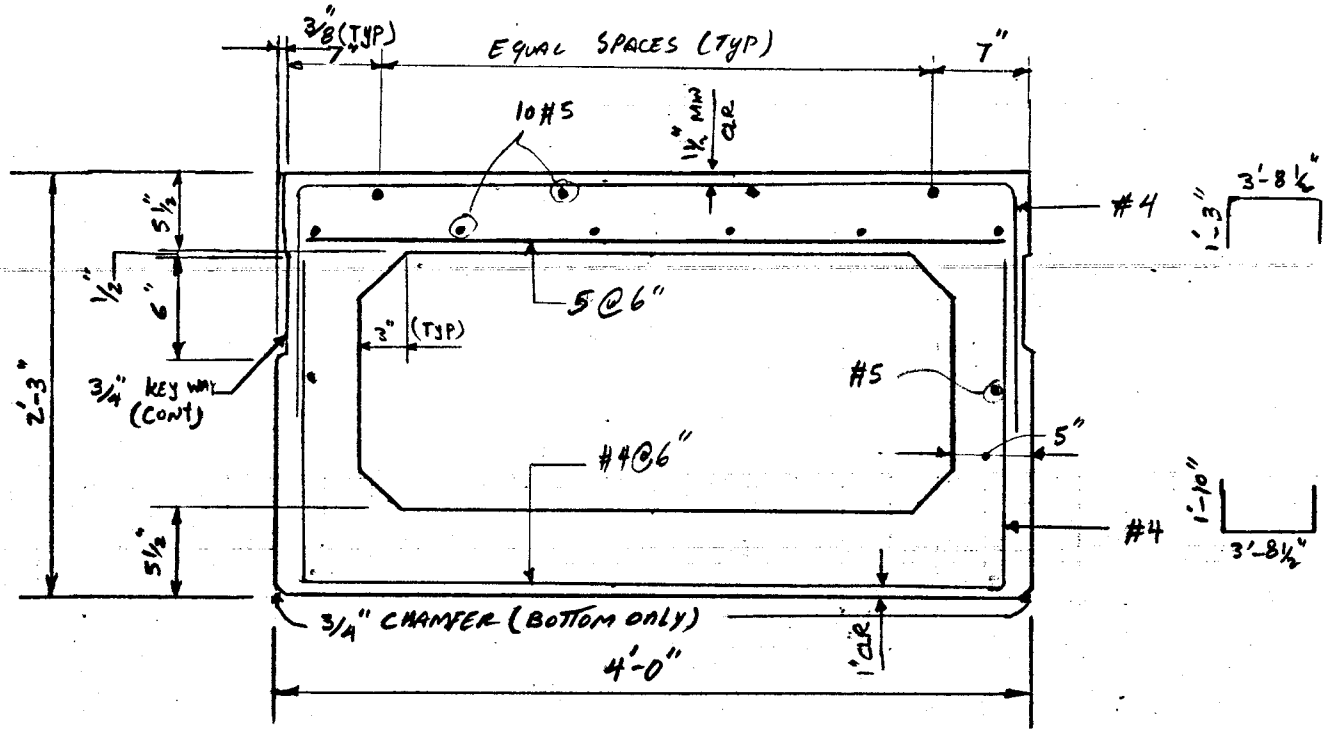

T — TYPICAL SECTION

SCALE : 1" = 1'-0"

Longitudinal Analysis

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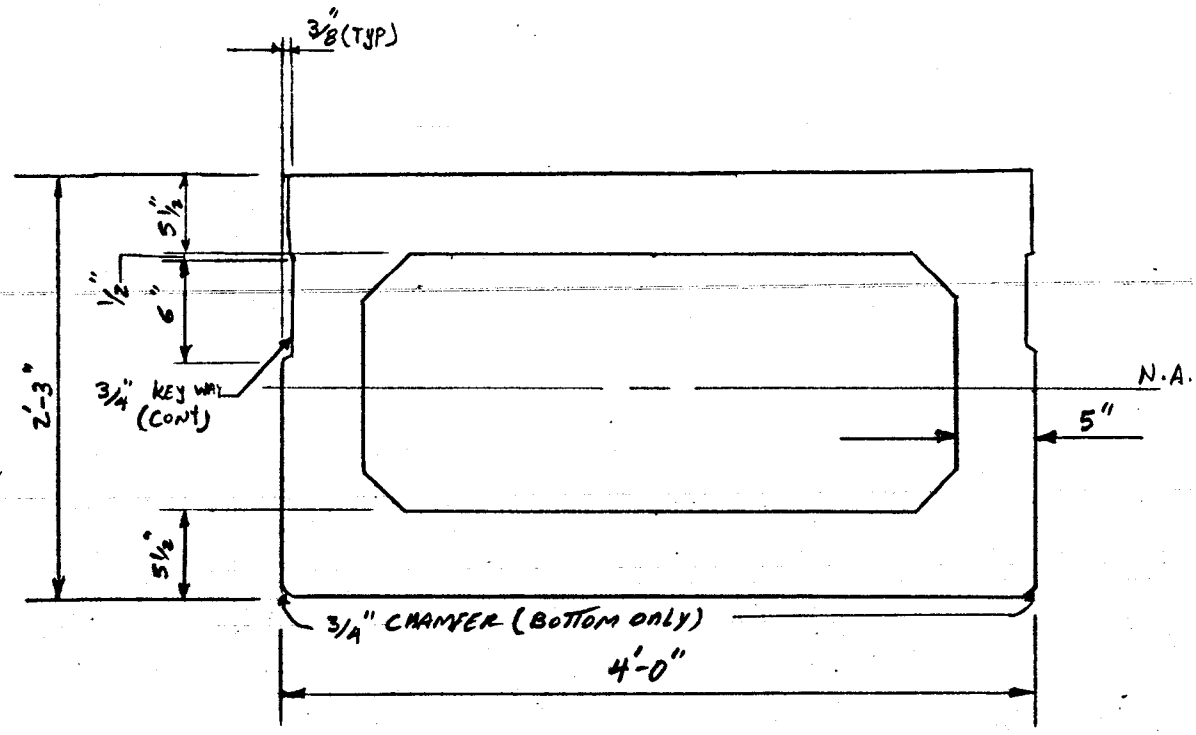
Subject 12th street OVER ACDC

Longitudinal Analysis



NON-Composite Section (BJ-48)

$I = 65{,}941 \text{ in}^4$

$y_b = 13.37 \text{ in}$

$S_b = 4932 \text{ in}^3$

$S_t = 4838 \text{ in}^3$

$y_t = 13.63 \text{ in}$

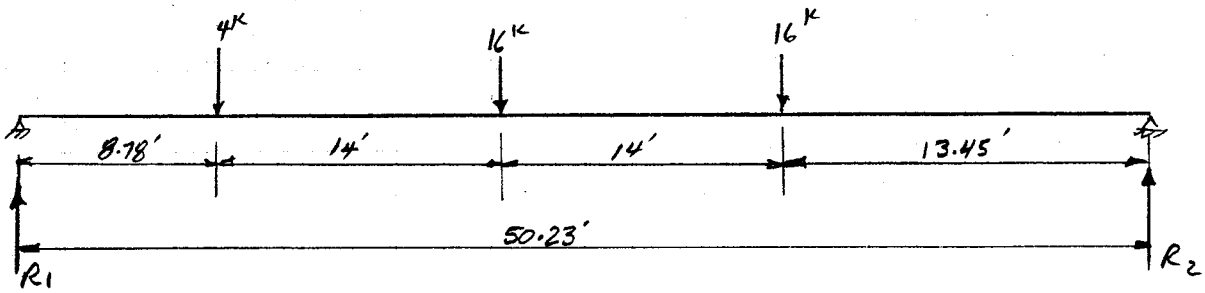
$A = 693 \text{ in}^2$

$f'_c = 5000 \text{ Psi}$

$f'_{ci} = 4000 \text{ Psi}$

Subject 12th street over ACDC
Longitudinal analysis

LIVE Load Moment (1. HS20-44)



$\Sigma M_2^{+} = 0 \quad -R_1(50.23) + 4(28+13.45) + 16(14+13.45+14) = 0 \quad R_1 = 16.504^{k}$

$M_x = 16.504(x - 4(x-8.78)\ ;\ x = 22.78' \quad M = 320^{k'} \times 1.285 \text{ (Impact)} = 412^{k\text{-}ft}$ ←

2. Pedestrain LL (85 PSF) AASTHO (3.14.1) $M = \frac{85(8')(50.23)^2}{8 \times 1000 \times 2 \text{ Girders}} = 107.3^{k\text{-}'}$ ←
Side walk is supported by 2 CELLS

DEAD Load Moments

1. Self weight BI-48 wt. = 722 #/ft $M = \frac{.722(50.28)^2}{8} = 228.2^{k\text{-}'}$ ←

2. Future Wearing surface 25 #/ft² & 2" Asphalt
$w = .025(4') + \frac{2''}{12}(4') \times 0.15 = 0.2^{k}/ft \quad M = \frac{.2(50.28)^2}{8} = 63.2^{k\text{-}'}$ ←

3. Sidewalk, Barrier & Fence (2 CELL LOADED)
sidewalk = 1.1 k/ft, Barrier & Fence = 0.4 k/ft $M = \frac{1.5(50.28)^2}{2 \times 8} = 237.^{k\text{-}'}$ ←

4. Water & Sewage Pipe & Content 4.13^k $M = \frac{2.1(50.23)}{8} = 13.2^{k\text{-}'}$ ←
wt. = $\frac{2.1}{50.23}$ TOTAL (see PREL. DESIGN)

TOTAL MOMENTS

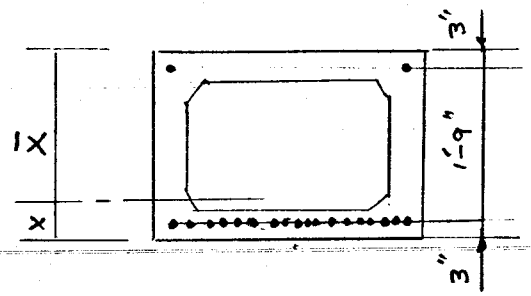
$M_{LL} = 412 \times \frac{0.6}{0.5} = 494.^{k\text{-}'}$ NOTE: increase LL Moment Due to adj Beam DL

A. $M = 228.2 + 63.2 + 494 = \underline{785.4^{k\text{-}'}}$ Due to SW + future Wearing + TRAFIC LL (interior BOX)
$M_D = 228.2^{k\text{-}'} \quad M_{D+50} = 291.4^{k\text{-}'} \quad M_{LL} = 494.^{k\text{-}'}$ ←

B. $M = 228.2 + 237 + 13.2 + 107.3 = \underline{585.7^{k\text{-}'}}$ Due to SW + Sidewalk, Barrier + Pipes + Ped. LL

Subject 12th street over ACDC
Prestressing Calculation & LOSSES

$P_j = 576^k$ $\quad A_s = \frac{570}{.75(270)} = 2.84''$ $\quad$ 20 strands of ½" ϕ



$18\left(\frac{3''}{12}\right) + 2(2') = 20X \quad X = 0.425' \quad \bar{X} = 2.25 - 0.425 = 1.825'$ FROM TOP

$P_0 = 0.95(576) = 547^k \quad \bar{e}_{N.A.} = 13.37'' - 0.43 \times 12 = 8.21''$

$f_{cir} = \frac{547}{693} + \frac{547(8.21)(8.21)}{65941} - \frac{228.2 \times 12(8.21)}{65941} = 1.007$ ksi

$f_{cds} = \frac{632 \times 12(8.21)}{65941} = 0.0944$ ksi

Shrinkage (AASHTO 9.16.1)

$SH = 17000 - 150\,RH = 17000 - 150(40) = 11000$ psi

Elastic Shortening

$ES = \frac{E_s}{E_{ci}} f_{cir} \quad E_s = 28 \times 10^6$ psi $\quad E_{ci} = 33(150)^{1.5}\sqrt{5000} = 4{,}287000$ psi

$ES = \frac{28 \times 10^6}{4287000} \times 1007 = 6577$ psi

Creep

$CR_c = 12 f_{cir} - 7 f_{cds} = 12(1007) - 7(94.4) = 11423$ psi

Relaxation of Prestressing Steel

$CR_s = 5000 - 0.10\,ES - 0.05(SH + CR_c) = 5000 - 0.1(6577) - 0.05(11000 + 11423) = 3221$ psi

TOTAL LOSS $= SH + ES + CR_c + CR_s = 11000 + 6577 + 11423 + 3221 = 32220$ psi

$P_f = 576 - 32.(2.84) = 485^k$

PCI METHOD Pg-3-29 PCI Handbook

$T_L = 19.8 + 16.3 f_{cir} - 5.4 f_{cds}$

$T_L = 19.8 + 16.3(1.007) - 5.4(.0944) = 35.7^{ksi}$

$V/S = \frac{693}{(8+4.5)\times 12} = 4.62 \quad Adj = -7.6 - 3.8 \times .62 = -9.956\%$

$T_L = 35.7 - .09956(35.7) = \underline{32.\ ksi}$

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Subject 12th street over ACDC
Shear Design

Shear strength of concrete AASHTO 9.20.2 CASE A. Typical interior Box

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7\sqrt{f'_c}\, bd \quad ; M_{cr} = \frac{I}{y_t}\left(6\sqrt{f'_c} + f_{pc} - f_d\right)$$

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b'd + V_p \qquad V_p = 0$$

$$2 \times 0.6\frac{\sqrt{5000}}{1000} \times 5'' \times 27'' \times 0.8 = 9.2 \qquad f_{pc} = \frac{P_f}{A} = \frac{491}{693} = 0.708^{ksi}$$

$$V_{ci} = 4.6 + V_d + V_i \frac{M_{cr}}{M_{max}}$$

$$V_{cw} = \left(3.5\frac{\sqrt{5000}}{1000} + 0.3 \times 0.708\right) \times 10'' \times 27'' \times 0.8 = 100^k \longleftarrow$$

$$f_{pc} = \frac{P_f e}{S_b} + f_{pc} \quad \frac{491 \times 8.21''}{4932} + 0.71 = 1.53^{ksi}$$

AT 0.5 L

$M_{LL} = 494 \qquad V_{LL} = 14^k$

$M_{D+SD} = 291.4^{k'} \qquad V_{D+SD} = 0^k$

$M_D = 228.2^{k'}$

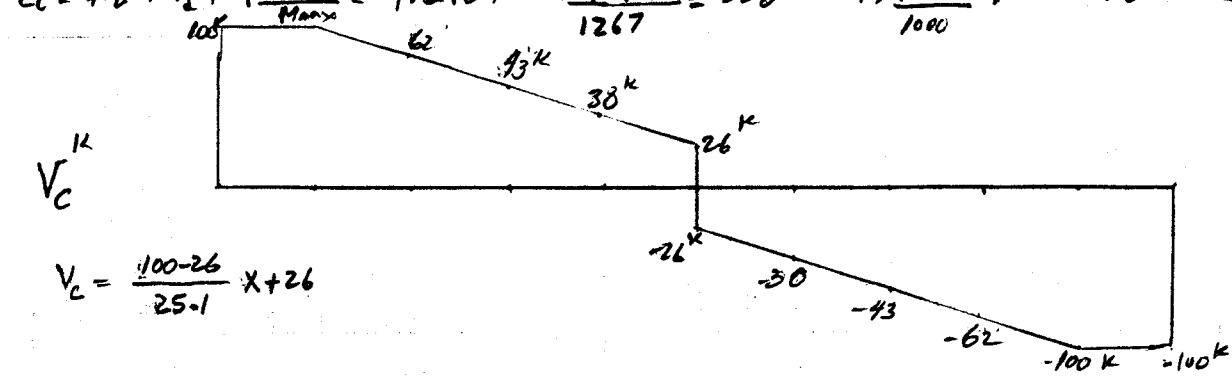
$$M_{max} = 1.3\left(291.4 + \frac{5}{3} \times 494\right) = 1444^{k'}$$

$$V_i = 1.3 \times \frac{5}{3} \times 14 = 30.3^k$$

$$f_d = \frac{M_D}{S_b} = \frac{228.2 \times 12}{4932} = 0.56^{ksi}$$

$$M_{cr} = \left(6\frac{\sqrt{5000}}{1000} + 1.53 - 0.56\right) \times \frac{4932}{12} = 572^{k'}$$

$$V_{ci} = 9.2 + V_d + V_i \frac{M_{cr}}{M_{max}} = 9.2 + 0 + 30.3\frac{(572)}{1267} = 22.8^k \qquad 1.7\frac{\sqrt{5000}}{1000} \times 27'' \times 10'' \times 0.8 = 26^k \longleftarrow$$



$$V_c = \frac{100-26}{25.1}x + 26$$

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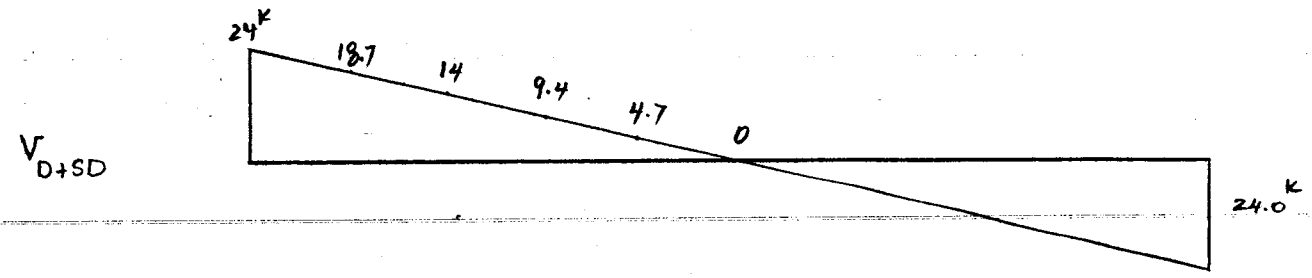
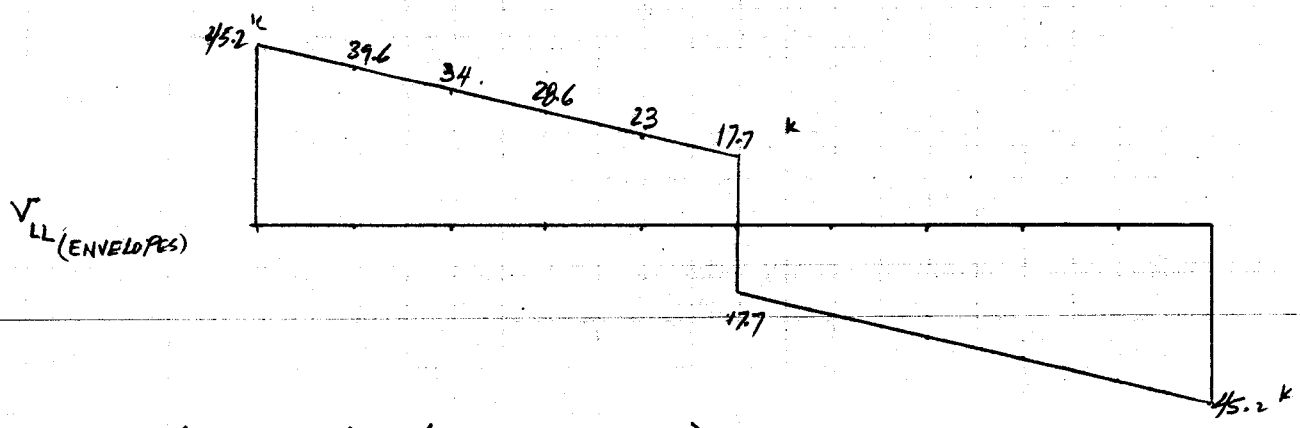
Subject 12th street over ACDC

Shear DIAG. FOR 1 Typical CELL

Shear Design Typical interior Box Case A





$V_u = 1.3\left(V_{D+SD} + \frac{5}{3} V_{LL}\right)$ (AASHTO 9.20.1)

$V_s = \frac{V_u}{.9} + V_c$ $A_s = \frac{V_s}{F_s d} \times 12 = \frac{V_s}{40 \times 21.8} \times 12$ $A_{v\,min} = \frac{50(10")}{40{,}000} \times 12 = 0.15\ in^2/ft$ USE 2#4@18 = .26

d = 0.8h is conservative for the area of the section

	0.L	0.1L	.2L	.3L	.4L	.5L	.6L	.7L	.8L	.9L	1.0L
V_u^k	129	110	92.2	74.2	56.3	38.4	-56.3	-74.2	-92.2	-110	-129
V_c	-100	-100	-62	-43	-30	-26	+30	+43	+62	+100	+100
V_s	29.2	10.2	30	31	26.3	12.4					
A_s in²/ft	.40 2#4@12"	.14 12"	.41 12"	.41 12"	.36 12"	.17 12"	12"	12"	12"	12"	12"

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Subject 12th street OVER ACDC
Longitudinal Design (Stresses)

$P_j = 576^k$

$P_o = 547^k$ (initial)

$P_f = 485^k$ (Final)

$e = 5.16''$ From BOT. $\bar{e} = 13.37 - 5.16 = 8.21''$

Compression (+)

Load	Support @ Release P = Po		Mid span @ Release P = Po		MIDSpan @ service P = Pf		
PSi	fb	ft	fb	ft	fb	ft	
P/A	789	789	789	789	709	709	
Pē/S	911	-928	911	-928	807	-823	
MD/S			-555	+566			
MD+SD/S					-709	723	
MLL/S					-1202	1225	
TOTAL Stress	1700	-139	1145	427	-395	1834	
Allowable Stress	.6 f'ci 2400	6√f'ci -379	0.6 f'ci 2400	0.6 f'ci 2400	6√f'c -844	0.45 f'c 2475	
	OK	OK Tension	OK	OK	OK ✓ Tension	OK	

NOTE: This is a Typical interior Box Design
Per calculation of WVE 10-25-89 The ultimate moment capacity is low there for the number of strands are increased by 2 & the eccent. of strands are reduced by 1" to 2" from bottom of Box
This will Produce Tension in the ends, close to supports; therefore either a compression strand or additional steel @ top close to the support must be provided

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Date: 10-25-89
Checked by: N/A
Date: 10-26-89

Shear Strength of Concrete Vci
Calculated at tenth points

1) Calulate Mmax & Vi at tenth points

(a) Mmax

Loc.	M_{DL}	$5/3$ LL
0.0L,	$(0.0^{'k} + 0.0)(1.3) = 0$	
0.1L,	$(106.75^{'k} + 1.54(128.4^{'k})^{5/3})(1.3) = 567.20^{'k}$	
0.2L,	$(188.50^{'k} + 1.54(220.8^{'k})^{5/3})(1.3) = 981.79^{'k}$	
0.3L,	$(242.00^{'k} + 1.54(277.2^{'k})^{5/3})(1.3) = 1246.024^{'k}$	
0.4L,	$(282.25^{'k} + 1.54(308.8^{'k})^{5/3})(1.3) = 1397.29^{'k}$	
0.5,	$294.0^{'k} + 1.54(310.0^{'k})^{5/3})(1.3) = 1417.^{'k}$	

(b) Vi

Loc.	V_{DL}	$5/3$ LL
0.0L,	$(24.^{k} + (29.3^{k})^{5/3})1.3 = 95^{k}$	
0.1L,	$(19.^{k} + (25.7^{k})^{5/3})1.3 = 80.4^{k}$	
0.2L,	$(14.^{k} + (22.1^{k})^{5/3})1.3 = 66.1^{k}$	
0.3L,	$(9.4^{k} + (18.48^{k})^{5/3})1.3 = 52.3^{k}$	
0.4L,	$(5^{k} + ((14.88^{k} - 4(.12))^{5/3})1.3 = 37.7^{k}$	
0.5L,	$(0 + ((11.52 - .22(4^{k}))^{5/3})1.3 = 23^{k}$	

Subject 12th St. over ACDC

Shear Strength of Concrete V_{ci}
calculated at tenth points

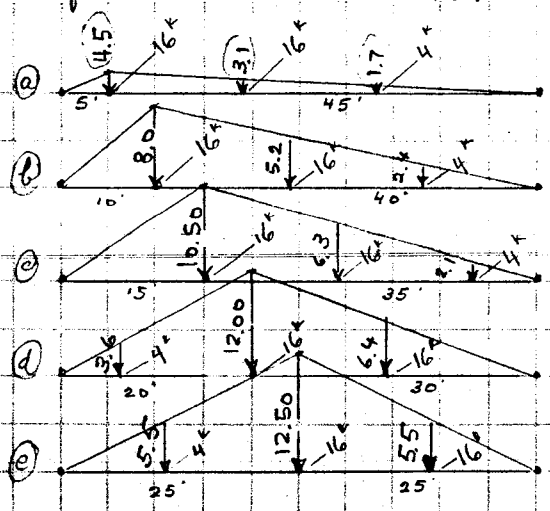
(2) Summarize V_{ci}

Loc.	4.6	+	V_D	+	$V_i \frac{M_{cr}}{M_{max}}$	
0.0L.	9.2^k	+	$24.^k$	+	$(95.^k)\frac{(572)}{572} = 128.2^k$	use $V_{cw} = 100^k$
0.1L.	9.2^k	+	$19.^k$	+	$(80.4^k)\frac{(572)}{567.2} = 109.3^k$	use $V_{cw} = 100^k$
0.2L.	9.2^k	+	$14.^k$	+	$(66.1^k)\frac{(572)}{982.} = 62^k$	
0.3L.	9.2^k	+	9.4^k	+	$(52.3^k)\frac{(572)}{1246.} = 43.^k$	
0.4L.	9.2^k	+	$5.^k$	+	$(37.7^k)\frac{(572)}{1397.} = 30^k$	
0.5L.	9.2^k	+	$0.^k$	+	$(23^k)\frac{(572)}{1417.} = 18.5^k$	use 26^k

Subject 12th Street over ACDC

① Maximum Moment at 5' intervals for LL wheel loads

Influence Lines for maximum moment, no impact included, ½ wheel load only



where $\frac{\Delta}{45} + \frac{\Delta}{5} = 1$, $\Delta = 4.5$, $M_{max} = 16 \times 4.5 + 16 \times 3.$ $+ 4 \times 1.7 = 128.$

where $\frac{\Delta}{40} + \frac{\Delta}{10} = 1$, $\Delta = 8.00$, $M_{max} = 16 \times 8.0 + 16 \times 5.$ $+ 4 \times 2.4 = 220.$

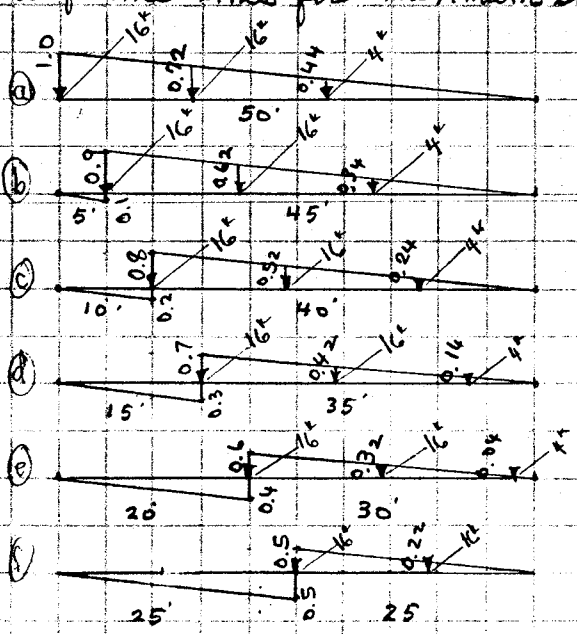
where $\frac{\Delta}{35} + \frac{\Delta}{15} = 1$, $\Delta = 10.50$ $M_{max} = 16 \times 10.50 + 16 \times 6$ $+ 4 \times 2.1 = 277.2'$

where $\frac{\Delta}{30} + \frac{\Delta}{20} = 1$, $\Delta = 12.00$ $M_{max} = 4 \times 3.6 + 16 \times 12.0$ $+ 16 \times 6.4 = 308.8$

where $\frac{\Delta}{25} + \frac{\Delta}{25} = 1$, $\Delta = 12.50$ $M_{max} = 4 \times 5.5 + 16 \times 12.5$ $+ 16 \times 5.5 = 310.^k$

② Maximum Shear at 5' intervals for LL wheel loads

Influence Lines for maximum shear



$V_{max} = 16 \times 1.0 + 16 \times .72 + 4 \times .44 = 29.28^k$

$V_{max} = 16 \times 0.9 + 16 \times .62 + 4 \times .34 = 25.68^k$

$V_{max} = 16 \times 0.8 + 16 \times 0.52 + 4 \times 0.24 = 22.1^k$

$V_{max} = 16 \times 0.7 + 16 \times .42 + 4 \times .14 = 18.48^k$

$V_{max} = 16 \times 0.6 + 16 \times 0.32 + 4 \times .04 = 14.88^k$

$V_{max} = 16 \times 0.5 + 16 \times 0.22 = 11.52^k$

③ Impact Factor $I = \frac{50}{L+125} = \frac{50}{50.23+125} = 0.285$

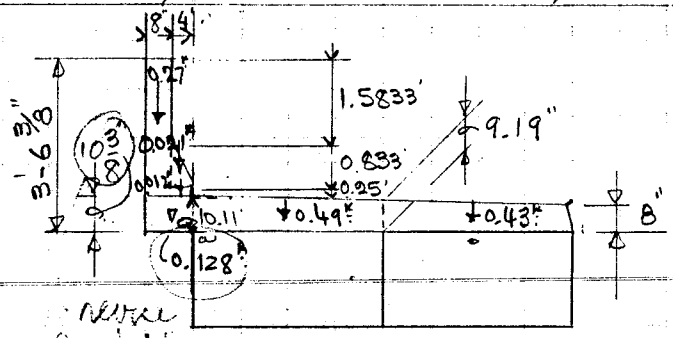
Distribution Factor $4.0/7.0 = 0.6$ for moment & shear AASHTO 3.23

Factor above by $\frac{.6}{.5} \times 1.285 = 1.542^k$

Subject 12th Street over ACDC
Longitudinal Analysis

(1) Loadings - other

(ii) Sidewalk & Barrier DL to exterior box girders



Neglect for 1-0

Sum DL outside girder	Sum DL interior girder
0.5014^k	0.1983^k
0.490^k	0.43^k
0.128^k	0.232^k

For Barrier

$\Sigma M_{D_0} = 0.27^k(6.67') + 0.021^k(6.22') + 0.012^k(6.17') - R(4') = 0,\ R = 0.5014^k$

$\Sigma M_{D_a} = 0.27^k(2.67') + 0.021^k(2.22') + 0.012^k(2.17') + R(4'),\ R = .1983^k$

(iii) Live Load for Sidewalk (3.0')

$(.0850\ \text{LBS}/\text{ft}^2)(4') = .340^k$ per girder

(iii) Intermediate Diaphragm $\left[(16")(38") - 2(3")^2\right]/144\text{in}^2/\text{ft}^2 \times (3.0') \times .150\frac{k}{ft^3} = 1.84^k$ say

(iv) Barrier rail

$\left(\frac{.75"}{12"}\right)(4)\left(\frac{.065"}{12"}\right)(3.0')\left(\frac{12"}{4.5"}\right) = 0.01083^k$

$\left(2\right)\left(\frac{1.5"}{12"}\right)(4)\left(\frac{.083"}{12"}\right)\left(.490\frac{k}{ft^3}\right) = 0.0034^k$

$\left(.25'\right)(4)\left(\frac{.120}{12"}\right)(0.0833')\left(.490\frac{k}{ft^3}\right)\frac{1}{10.0833} = 0.002^k$, Sum 0.0162^k say $.02^k$

(v) 2" Asphaltic Concrete Surface, FWS

$\left(\frac{.167'}{12"}\right)(.150\frac{k}{ft^3})(4') + (.025\frac{k}{ft^2})(4') = 0.108^k$

(vi) 8" water line 72.42^{lbs}, $\pi\left(\frac{8^2}{4(144)}\right)(62.4\frac{lbs}{ft^3}) = 22^{lbs}$

use 0.100^k

(2) Distribution of Loads

AASHTO 3.23, Table 3.23.1 For interior box girders $S/7.0 = \frac{4.0}{7.0} = 0.6$

For exterior box girders $We/7.0 = \frac{4.0}{7.0} = 0.6$

(3) Moments at ℄ Box, exterior girder

(ii) SW of Box, intermed. diaph.

where $w = \frac{692.5}{144} \times .150 \frac{k}{ft^3} = 0.72 \frac{k}{ft}$

intermed. diaph $P = 1.84^k$

$M = \frac{PL}{4} + \frac{wL^2}{8} = \frac{(1.84)(50.23')}{4} + \frac{(0.72)(50.23)^2}{8} = \boxed{250.2^{'k}}$

(iii) Sidewalk Load, SDL

DL For sidewalk & Barrier, other

$w = 1.12 + .02 + .100 = 1.24$

LL For sidewalk (0.340) see footnote h p. 35 AASHTO

Sum $1.24 \frac{k}{ft}$

$M = \frac{wl^2}{8} = \frac{1.24(50.23)^2}{8} = \boxed{391.10^{'k}}$

(iii) LL

$M = (316^{'k})(1.285)\left(\frac{0.6}{0.5}\right) = \boxed{487.27^{'k}}$

(4) Stresses at midspan, exterior girder

(ii) SW of Box, intermed. diaph. $S_b = 4932 \text{ in}^3$, $S_t = 4938. \text{ in}^3$

$f_{xt} = \frac{(250.2^{'k})(12'')}{4838. \text{ in}^3} = \boxed{0.621 \text{ k/in}^2}$

$f_{xb} = \frac{(250.2^{'k})(12'')}{4932. \text{ in}^3} = \boxed{-0.609 \text{ k/in}^2}$

(iii) Sidewalk Load, SDL

$f_{xt} = \frac{391.1 (12'')}{4838. \text{ in}^3} = \boxed{0.97 \text{ k/in}^2}$

$f_{xb} = \frac{(391.1)(12'')}{4932. \text{ in}^3} = \boxed{-0.95 \text{ k/in}^2}$

(iii) LL

$f_{xt} = \frac{(487.3^{'k})(12'')}{4838.0 \text{ in}^3} = \boxed{+1.209 \text{ k/in}^2}$

$f_{xb} = \frac{(487.3^{'k})(12'')}{4932.0 \text{ in}^3} = \boxed{-1.186 \text{ k/in}^2}$

LL using SW in composite action

$f_{xt} = \frac{(487.3^{'k})(12'')}{9889.27 \text{ in}^3} = 0.591 \text{ ksi}$

$f_{xb} = \frac{(487.3^{'k})(12'')}{7955.15 \text{ in}^3} = 0.735 \text{ ksi}$

(iv) Total Stresses midspan

$f_{xt} = 0.621 + 0.97 + 1.209 = 2.800 \text{ k/in}^2$ w/o LL $f_{xt} = 1.591$ ksi

$f_{xb} = -0.609 - 0.95 - 1.186 = -2.745 \text{ k/in}^2$ w/o LL $f_{xb} = -1.56$ ksi

(5) Estimate Post Tensioning Force, center of span, exterior girder

say $e_b = 0.1h = 2.7''$ or $3''$, $e = 13.37'' - 3'' = 10.37''$ reduce to 8"

$\frac{P_x}{A} \pm \frac{P_x e}{I} = -2.745 \frac{k}{in^2}$, $+.424$ bottom fiber $\quad 6\left(\frac{\sqrt{5000}}{1000}\right) = 0.424\ k/in^2$

$$\frac{P_x}{692.50} \pm \frac{P_x(8'')}{4932.}(10.37'') = \begin{matrix}+2.745\\-.424\end{matrix}, \quad P_x\left[\frac{1}{692.50} \pm \frac{8}{10.37}\right] = 2.321, \quad P_x = 654.42^k \ (757^k)$$

check top fiber stress

$$\frac{654.42^k}{692.50} - \frac{654.42^k(10.37'')}{4838.0} + 2.815 = 0.945 - 1.403 + 2.815 = 2.357\ k/in^2$$

$0.4(5000) = 2000\ k/in^2$

For 8" eccentricity $+\left(\frac{P}{692.50}\right) - \left(\frac{P}{4838.0}\right)(8'') + 2.800 = 2.0$, $P = 3818^k$ no good for 8" eccentricity

top fiber controls in compression

$$\frac{P_x}{692.50} - \frac{P_x(10.37'')}{4838.0} + 2.815 = 2.0\ \frac{k}{in^2} \rightarrow P_x\left[\frac{1}{692.50} - \frac{10.37}{4838.0}\right] + 2.815 = 2.0$$

$$P_x[-0.0006994] + 2.815 = 2.0$$

$$P_x = 1165.3^k$$

where $f_{s\,pose} = 35\ k/in^2$, $P = \frac{1165.3^k}{\left(\frac{202.5 - 35}{202.5}\right)} = 1409.^k$ or $\frac{1409.0^k}{31} = 45.5$ strands *

(6) Estimate Post-tensioning Force, ends of box girder, exterior girder

Bot. fiber

$$\frac{1165.3^k}{692.50} + \frac{(1165^k)(10.37'')}{4932.0} = 1.683 + 2.45 = 4.133\ k/in^2 > 2.0\ k/in^2$$

Top fiber

$$\frac{1165.3}{692.50} - \frac{(1165)(10.37'')}{4838.0} = -0.815\ k/in^2 > -.424\ k/in^2$$

* ⇨ Note: The exterior girder cannot be designed for sidewalk DL & .6W Live Load without considering composite action with the sidewalk slab. See further calculations and notes for exterior girder.

Subject 12th Street over ACDC
Longitudinal Analysis

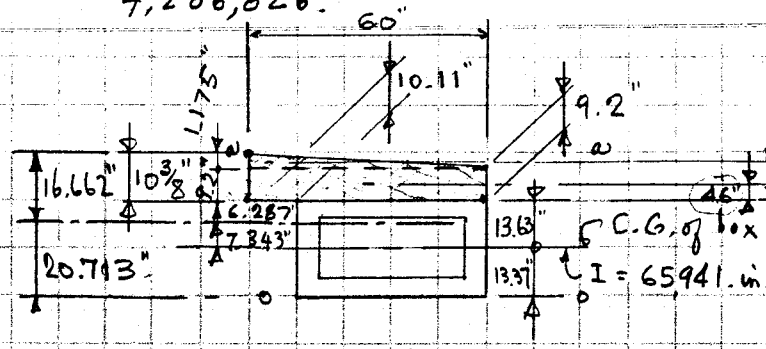
Compute Properties of Concrete Cross Section using Sidewalk for Composite Action

$f'c$ box = 5000 psi, $f'c$ sidewalk = 3000 psi

for box $E_c = 33(150)^{3/2}\sqrt{5000} = 4,286,826.$ psi

for Sidewalk $E_c = 33(150)^{3/2}\sqrt{3000} = 3,320,561.$ psi

$r = \dfrac{3,320,561.}{4,286,826.} = 0.7746$



$A_{c1} = \frac{1}{2}(60")(1.175")(.7746) = 27.305$

$A_{c2} = (9.2")(60")(.7746) = 427.58 in.^2$

$\Sigma M_{a} = (27.305 in.^2)(0.775")$
$(427.58 in.^2)(5.775")$
$(693.0 in.^2)(24.005") = 1147.89\,(y_t)$, $y_t = 16.662"$

$\Sigma M_{o} = (693 in.^2)(13.37")$
$(427.58 in.^2)(31.60")$
$(27.305 in.^2)(36.60) = 1147.89\,(y_b)$, $y_b = 20.713"$

$I = 65,941.0 + (693.0)(7.343")^2$
$60.(9.2)^3/12 + (427.58 in.^2)(10.887")^2$
$(60.)(1.175)^3/36 + (27.305 in.^2)(15.887")^2 = 164775. in.^4$

$I_x = 164,775 in.^4$, $A_x = 1147.89 in.^2$

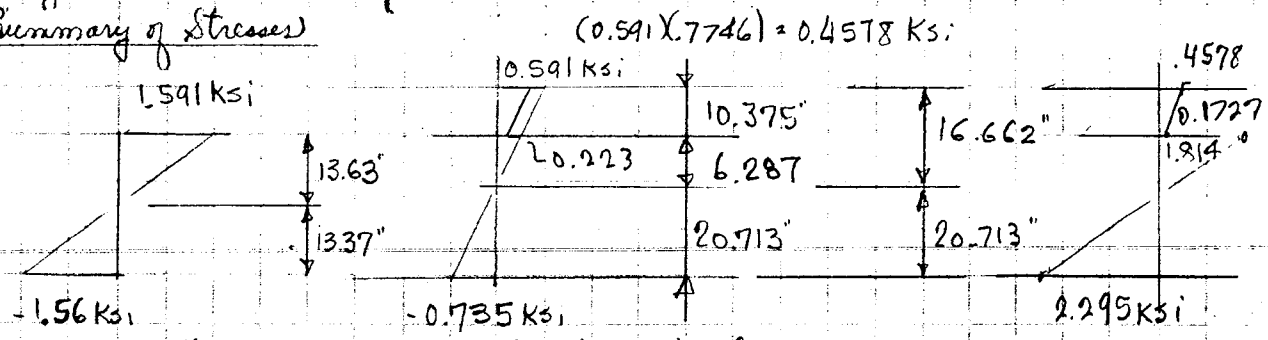
$S_{xt} = 164,775 in.^4 / 16.662" = 9889.27 in.^3$

$S_{xB} = 164,775 in.^4 / 20.713" = 7955.15 in.^3$

Subject 12th Street over ACDC
Longitudinal Analysis

(5r) Estimate Posttensioning Force, center of span, exterior girder using sidewalk for composite action

(a) Summary of Stresses



(b) Approximate C.G.S. where e = 10.37" & Post-tensioning

Bottom fiber stresses

$$F\left(\frac{1}{692.50\,in^2} + \frac{10.37''}{4932}\right) - 2.295 + .424 = 0$$

$F_x = 527.54$ Ksi

Top fiber stress

$$(528)\left(\frac{1}{692.50} - \frac{10.37''}{4838}\right) + 1.814 = 1.445 \text{ Ksi} < 2.0 \text{ ok}$$

Assume losses of 35 Ksi where $\frac{(202.5-35)}{(202.5)} = 0.83$, $\frac{528}{0.83} = 636^k$

$636^k / 31^k_{\text{strand}} = 21$ strands

(c) Check initial conditions

Say initial conditions at 189 Ksi or $\frac{189}{202.5} = 0.93$

Bottom fiber

$$(.93)(636)\left(\frac{1}{692.50} + \frac{10.37''}{4932}\right) - (.91)(.61) = 1.54 \text{ Ksi}$$

Top fiber

$$(.93)(636)\left(\frac{1}{692.50} - \frac{10.37}{4838}\right) + (.91)(.621) = 0.15 \text{ Ksi}$$

Note: Design of exterior girder is contingent upon composit sidewalk slab behaviou The sidewalk slab must be continuous w/o later joints and adequate shear key, etc. provide

(d) Check stresses at end of girder

Bottom fiber $(.93)(636)\left(\frac{1}{692.5} + \frac{10.37}{4932}\right) = 2.10$ Ksi $\quad .6(4.000) = 2.4$ Ksi

Top fiber $(.93)(636)\left(\frac{1}{692.5} - \frac{10.37}{4838}\right) = -0.414$ Ksi $\quad 7.5\sqrt{\frac{4000}{1000}} = 0.47^k/in^2$ provide bonded reinforcement

(7) Estimate Post-tensioning Force, center of span, interior girder, typ.
Revise to not include sidewalk load
where $w_{SDL} = 0.108^k + .100^k = .208^k$
$M = (0.208^k)(50.23^2)/8 = 65.6^{'k}$
$f_{xt} = 0.621 + (0.97)(\frac{66}{391}) + 1.21 = 1.995\ ^k/in^2$
$f_{xb} = -0.609 - (0.95)(\frac{66}{391}) - 1.186 = -1.955\ ^k/in^2$
Check PT Force req'd. for bottom fiber
$\frac{P_x}{692.50} \pm \frac{P_x(10.37")}{4932.0} = +1.955,\ -.424$, $P_x\left[\frac{1}{692.50} + \frac{10.37}{4932}\right] = 1.531$, $P_x = 432$
Check top fiber stress
$\frac{432^k}{692.50} - \frac{432^k(10.37")}{4838.0} + 1.995 = .624 - .926 + 1.995 = 1.693 < 2.0\ ^k/in^2$
where f_s loss $= 35\ ^k/in^2$, $P = \frac{432}{\frac{(202.5-35)}{202.5}} = 522^k$, $\frac{522^k}{31} = 17$ strands

To avoid 2 designs or to create similiar strand patterns, use 21 strands $P_J = 636^k$
or $P_x = 528^k$, for bottom fiber $f_{xb} = 528^k\left[\frac{1}{692.50} + \frac{10.37}{4932}\right] \mp 1.955 = -0.0824\ ^k/in^2$
$< .424\ ^k/in^2$

top fiber $f_{xt} = 528^k\left[\frac{1}{692.50} - \frac{10.37}{4838}\right] + 1.995 = 1.626\ ^k/in^2$
$< 2.0\ ^k/in^2$

Note: Stresses need not be checked again for initial conditions under self wt. at midspan since it is the same for the exterior girder. Similiarly, the stresses need not be checked at the girder ends.

(8) Check losses in pretensioned box girder See p. 94, Sect. 9.16.2 AASHTO
$\Delta f_s = SH + ES + CR_c + CR_s$, where $SH = 17000. - 150\,RH = 17000. - 150(40) = \boxed{11000.\ psi}$ SH
$f_{cir} = \frac{-250.2^k(12")(10.37")}{65941.} + \frac{(.93)(636^k)(10.37")^2}{65941} + \frac{(.93)(636^k)}{693.\ in^2} = -0.472 + 0.965 + 0.85 = 1.343\ ^k/in^2$
$E_S = (1.0)(\frac{29000000}{4286826.0})(1.343) = 9.085\ ^k/in^2$ or $\boxed{9085.\ psi}$ ES
$f_{cds} = (\frac{391.1^k \times 12" \times 10.37"}{65941.}) = 0.738$ ksi or 738.0 psi; $CR_C = 1000[12(1.343) - 7(.738)] = \boxed{10950.0\ ^k/in^2}$ CR_c
$CR_{S1} = 20000. - 0.4(9085) - 0.2(11000 + 10950) = \boxed{11976.0\ psi}$
$CR_{S2} = 5000. - 0.10(9085) - 0.05(11000 + 10950) = \boxed{2994.0\ psi}$
Note: for interior girders the f_{cds} must be revised where $f_{cds} = \frac{66}{391} \times .74 = 0.125\ ^k/in^2$ or 125 psi
$CR_C = 1000[12(1.343) - 7(.125)] = \boxed{15241.0}$ $CR_{S2} = 5000 - 0.10(9085) - 0.05(11000 + 15241) = 2779.\ psi$
For ext. girder using CR_{S2}: $\Delta f_s = 11000. + 9085. + 10950. + 2994. = \boxed{34029.\ psi}$ 35 ksi assumed OK
For int. girder using CR_{S2}: $\Delta f_s = 11000. + 9085. + 15241 + 2779. = \boxed{38105.\ psi}$ some overdesign exist for interior girders OK

Subject 12th Street over ACDC
Longitudinal Analysis

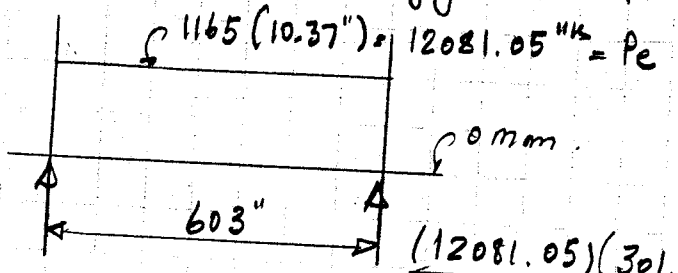
Check Deflections, exterior girder

① Deflections due DL of Box and sidewalk loading
where $I = (65941\ in^4)$, $w = .72 + 1.24 = 1.96^k$, $P = 1.84^k$, $l = 50.23'(12'') = 603''$
where $\Delta_w = \frac{5wP^4}{384EI}$, $\Delta_p = \frac{Pl^3}{48EI}$

$E = 33(150)^{3/2}\sqrt{5000} = 4{,}286{,}826.$ psi or 4286.83 ksi

$$\Delta_w + \Delta_p = \frac{5(2)^k(603)^4}{(384)(4287)(65941)(12'')} + \frac{1.84(603)^3}{48(4287)(65941)} = 1.015'' + .03 = 1.04''$$

② Deflections due to PT use Conjugate Beam method $\frac{Pe}{EI}$ diag. use factor $\frac{528^k}{1165^k} = 0.45$



$Pe = 1165(10.37'')$.

$$\frac{(12081.05)(301.5'')}{(4287)(65941)} = .01288 \text{ rad.}$$

$$M = \frac{12081.05(301.5)(301.5)}{(4287)(65941)} - \frac{12081.05}{(4287)(65941)}(301.5)\left(\frac{301.5}{2}\right) = \Delta_{PT} = 1.94''$$

Factor by $\left(\frac{528^k}{1165^k}\right) \times 1.94'' = 0.88''$

④ Initial deflection under sidewalk slab & Barrier loading
$\Delta_I = +0.88 - 1.04'' = -0.16''$

⑤ Further creep downward will be stiffened by the sidewalk slab acting compositely with the girder.

$$\Delta_f = -0.16 + \frac{4.0(-0.16)(65941.0)}{164775\ in^4} = -.16'' - .26'' = -0.42''$$

⑥ LL deflection - refer to HP41-C p.24, Simple beam, $L = 50.23'(12'') = 603''$, exterior girder
$a_1 = 8.78'(12'') = 105.4$, 40001bs, $I = 164775\ in^4$, $E = 3{,}320{,}561.$
$a_2 = (8.78' + 14')(12'') = 273.36$, 16000lbs, $L/2 = 301.4$, $y = -.246''$, $\theta = -.000033$, $M_x = 3{,}690{,}000$ "LBS
$a_3 = (8.78' + 28')(12'') = 441.4$, 16000lbs, $V = 3665$ LBS
$-.42'' - .25\frac{(.6)(1.3)}{.5} = -.81''$, $\frac{1}{1000} \times 50.13 \times 12 = .942''$ OK

Check Deflections, interior girder

① Deflections due to DL of Box & SDL ~ Refer to similar checks for ext. girder

$$\Delta_w + \Delta_p = \frac{5(.72+.21)(603)^4}{(384)(4287)(65941)(12'')} + 0.03 = 0.502''$$

② Deflections due to PT, $\Delta_{PT} = 0.88''$

③ Initial deflection under sidewalk slab & Barrier loading

$$\Delta_I = +0.88 - 0.502'' = 0.378''$$

④ Further creep upward will occur.
For conservatism in checking final deflections, assume no additional upward deflection due to PT & DL. $\Delta_f = 0.378''$ ↑

⑤ LL deflection

$$y = \frac{-.246\,(164775.0\text{ in.}^4)}{(65941.\text{ in.}^4)} = -0.615'' \downarrow$$

$$\Delta_r = .378'' - 0.615'' = -0.24'' < .942'' \text{ OK}$$

Comment: Differential deflection will exist between the exterior box girder & the interior girders where $\Delta_{f_{ext}} = -0.42''$ $\Delta_{f_{int}} = 0.378''$ ↑
the intermediate diaphragm will distribute this differential to some extent though the resulting deflection cannot be determined without a suitable 3 dimensional (grillage model). With some distribution due to the intermediate diaphragm and friction between the boxes, the remainder of the differential deflection will be hidden by the flexibility of the asphalt concrete and its ability to shift without cracking, etc. This is mentioned as a characteristic of precast box girder construction where a topping is provided in floor construction. See p. 372 "Modern Prestressed Concrete" by Libby. Use of 2 intermediate diaphragms will distribute the differential deflections more evenly, but 1 was used per Seetha Ramaiah & Dan Church in the original design layout.

Ultimate Moment Check

Refer to AASHTO 9.17

ⓐ Ultimate Flexural Capacity non-composite moment capacity

For bonded members where $\rho^* = \frac{(22)(0.153 in^2)}{(48")(24")} = 0.003 in^2$

$f^*_{su} = 270\left(1 - \frac{(0.5)(0.003 in^2)(270)}{5.0}\right) = 248.13\ ^k/in^2$

$\frac{1.4 d \rho^* f^*_{su}}{f'_c} = \frac{1.4(24")(.003)(248.13)}{5.0} = 5.0"$ Rectangular Section behavior controls $5.0" < 5.5"$

$\phi M_n = 1.0\left[22(.153 in^2)(248.13 \frac{k}{in^2})(24")\left(1 - \frac{0.6(.003)(248.13)}{5.0}\right)\right]$

$\phi M_n = 18{,}254.4\ ''^k$ or $1521.2\ '^k$

ⓑ Ultimate Moment Required

$M_u = 1.3(250.2^k + 391.1^k + \frac{5}{3}(487.3^k)) = 1890^k$ for exterior girder $> 1521.2^k$

$M_u = 1.3(250.2^k + 66^k + \frac{5}{3}(487.3^k)) = 1467.^k < 1521.2^k$

ⓒ Ultimate Flexural Capacity - composite section

where $b = 48$, $d = 24 + (10.11 + 9.2)(.5) = 34"$, $f'_c = 3000.$ psi

$\rho^* = \frac{(22)(.153 in^2)}{(48)(34")} = 0.0021$

$f^*_{su} = 270\left(1 - \frac{0.5(.0021)(270)}{3.0}\right) = 244.5 \frac{k}{in^2}$

$\frac{1.4(34")(.0021)(244.5)}{3.0} = 8.15$ flange, Rectangular Section behavior governs $8.15 < 15.1$

$\phi M_n = 1.0\left[(22)(.153 in^2)(34")(245)\left(1 - \frac{0.6(.0021)(245)}{3.0}\right)\right]\frac{1}{12} = 2096^k > 1890'$

ⓓ $\rho^* f^*_{su}/f'_c = (.003)(248)/5.0 = 0.149 < 0.3$ $.0021(245)/3.0 = 0.17 < 0.3$

ⓔ where $f_r = 7.5\sqrt{5000} = 530.33$ $F_r = \frac{M_{cr}}{I}$; $P_x = 22(.153 in^2)(168) = 565.^k$

$f_{pe} = 565[.00355] = 2.006$, $2.006 + .530 = \frac{M_{cr}(12")}{4932}$, $M_{cr} = 1042^k$

$1042^k \times 1.2 = 1250.^k$

Subject 12th street over ACDC
FRICTION Between the boxes

CHECKING stresses in THE RODS of The Box

GALV. TIE ROD $1\frac{1}{4}''\phi \times 7'-7\frac{1}{2}''$

$A = \pi R^2 = 1.2272 \text{ in}^2$

$I = \frac{\pi R^4}{4} = 0.1198 \text{ in}^4$

Tension in ROD = 30,000 Lbs

$$\sigma_N = \frac{P}{A} = \frac{30^k}{1.2272} = 24.45 \text{ ksi}$$

$$\tau = \frac{VQ}{It} \quad \text{where } Q = A \cdot \bar{y} \qquad \bar{y} \text{ for half of circle} = \frac{4R}{3\pi}$$

$$\tau = \frac{V\left(\frac{\pi R^2}{2}\right)\left(\frac{4R}{3\pi}\right)}{\frac{\pi R^4}{4} \times 2R} = \frac{4V}{3\pi R^2} = 0$$

*NOTE:

No shear force will be transfered through the RODS and The Concrete friction between the Boxes will Transfer the shear force.

$$\delta = \frac{PL}{EA} = \frac{30^k \times 91.5''}{1.2272 \times 30 \times 10^3} = 0.0746'' \quad \text{Elongation of TIE ROD}$$

Assuming all the friction is transfered through the joint by comp. of the boxes together

FORCE Due to Friction $= \mu N = 0.60 \times 30^k \times 2 = 36^k$ per Box joint

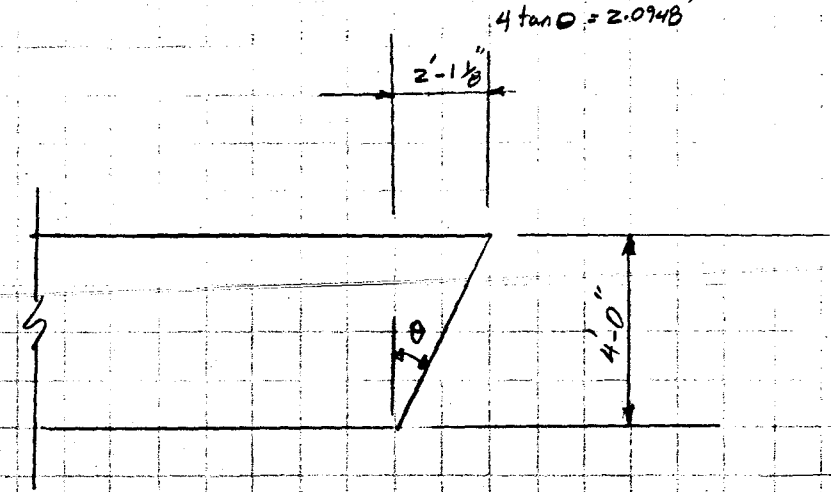
MAX Longitudinal Shear Due to LL @ (0.1L) $= 33^k$ per Box (see shear Design)

Parsons Brinckerhoff **Computation Sheet**

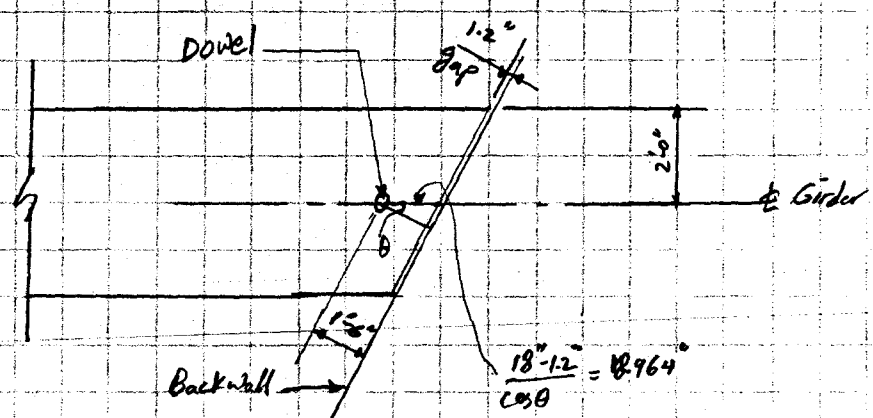
Subject 12th Street OVER ACDC
END DIAPH GEOMETRY

Page ____ of ____ 27
Made by Ali
Date 10-2-89
Checked by
Date

$4 \tan\theta = 2.0948'$

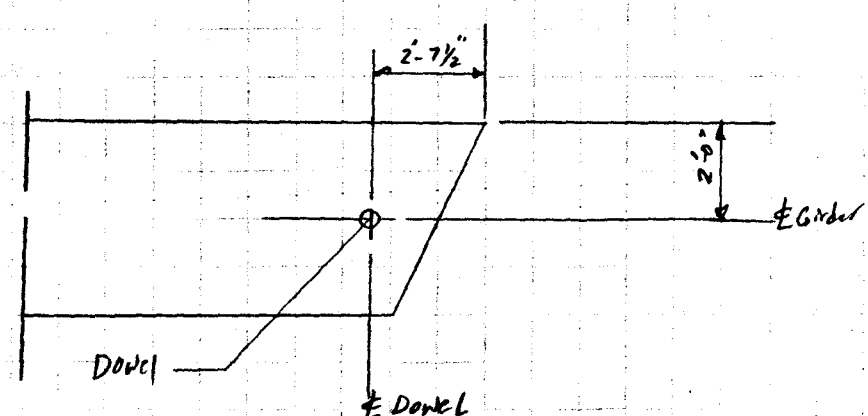


θ = skew Angle = 27°38'26"



$\frac{18" - 1.2"}{\cos\theta} = 18.964"$

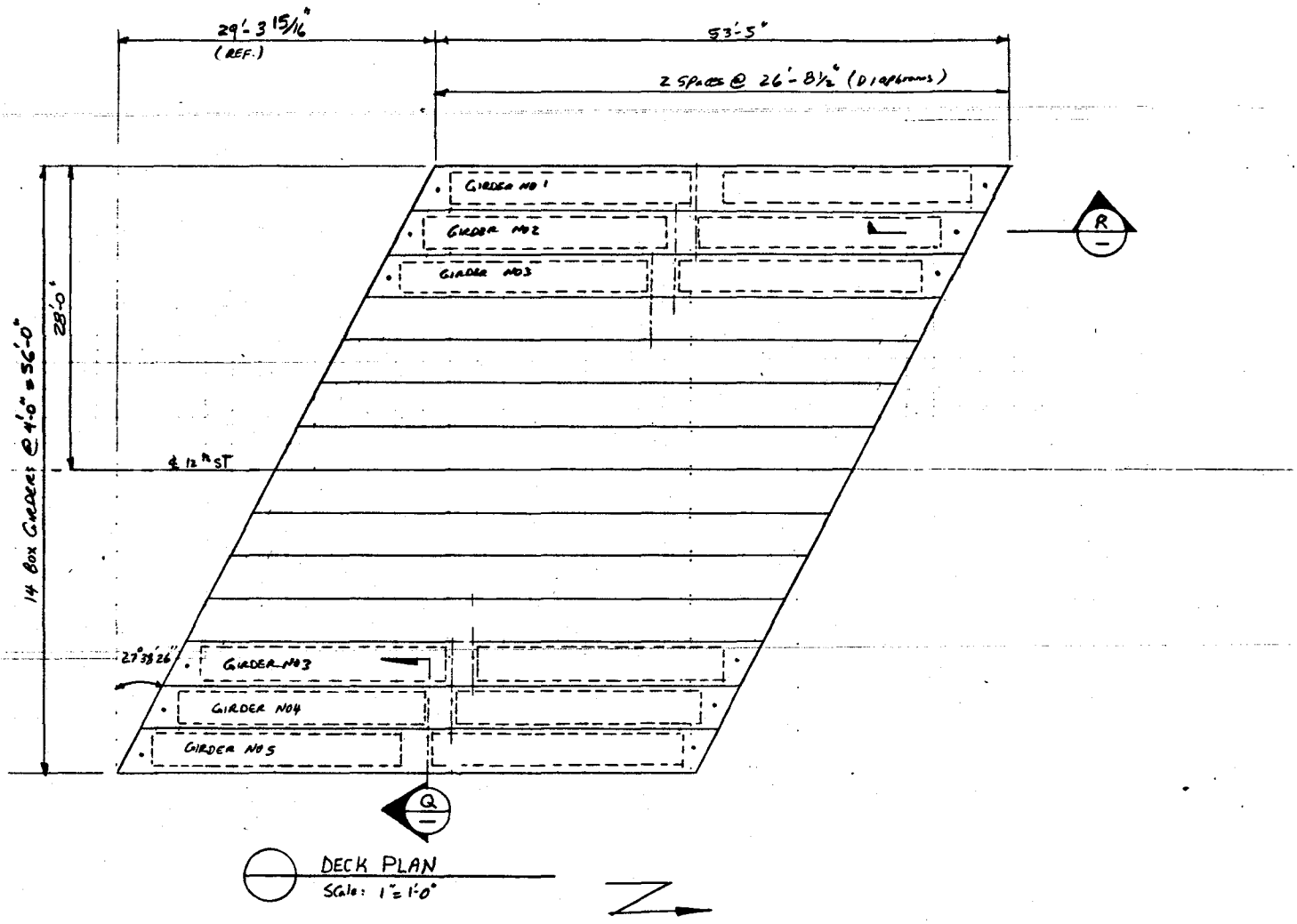
$\frac{2.0948'}{2} + \frac{18.964}{12} = 2.6278' = 2'-7\frac{1}{2}"$



Parsons Brinckerhoff Computation Sheet

Subject: 12th street OVER ACDC
DECK PLAN

Page ____ of ____
Made by: Ali
Date: 10-5-89
Checked by: ____
Date: ____

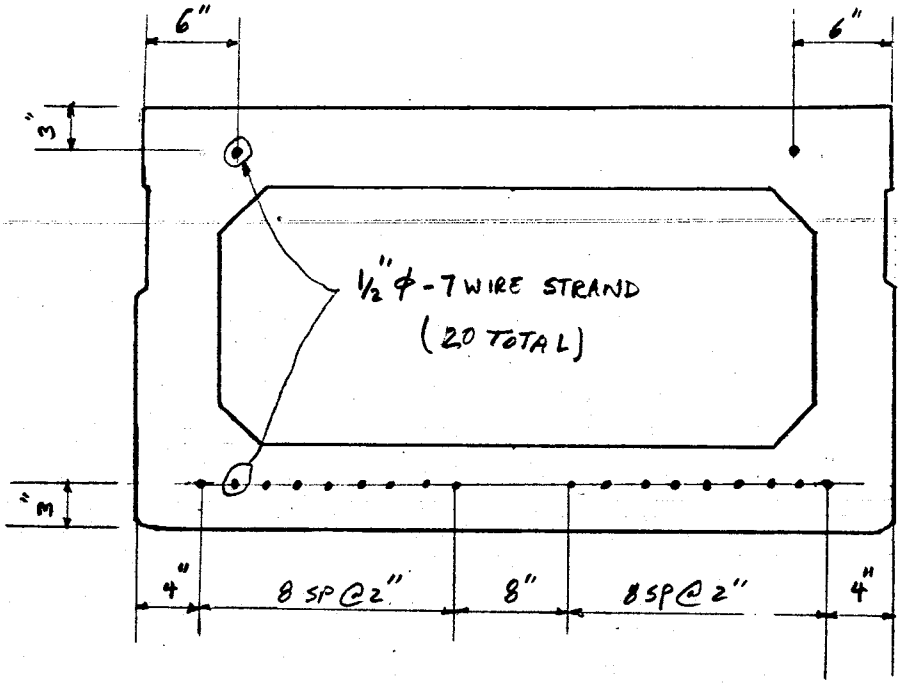


DECK PLAN
Scale: 1" = 1'-0"

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
PRESTRESSING DETAIL

Page ____ of ____ 30
Made by ALI
Date 9-14-89
Checked by
Date



U/— TYPICAL SECTION
SCALE: 1" = 1'-0"

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC

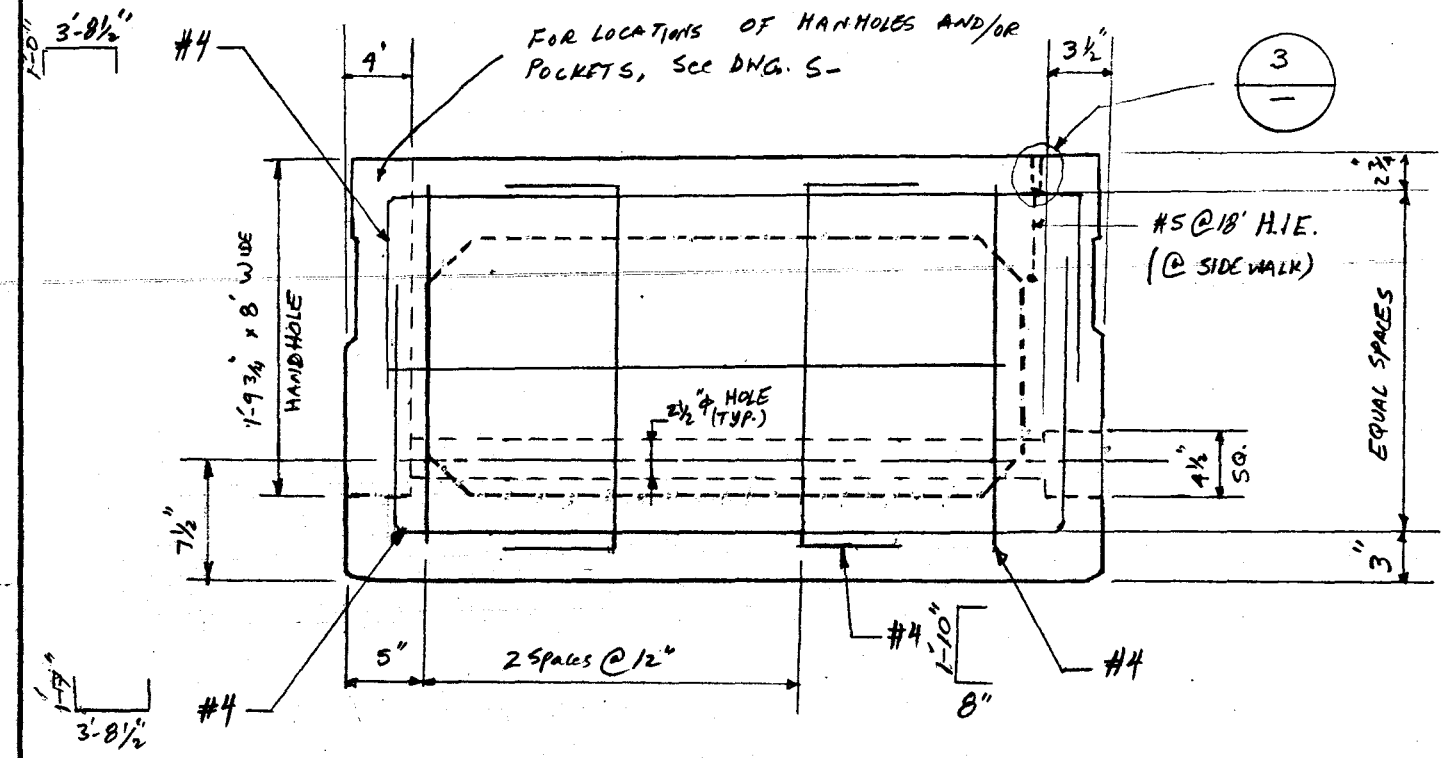
Page ____ of ____ 31

Made by ALI

Date 9-18-89

Checked by

Date



1 — SECTION

SCALE: 1" = 1'-0"

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC

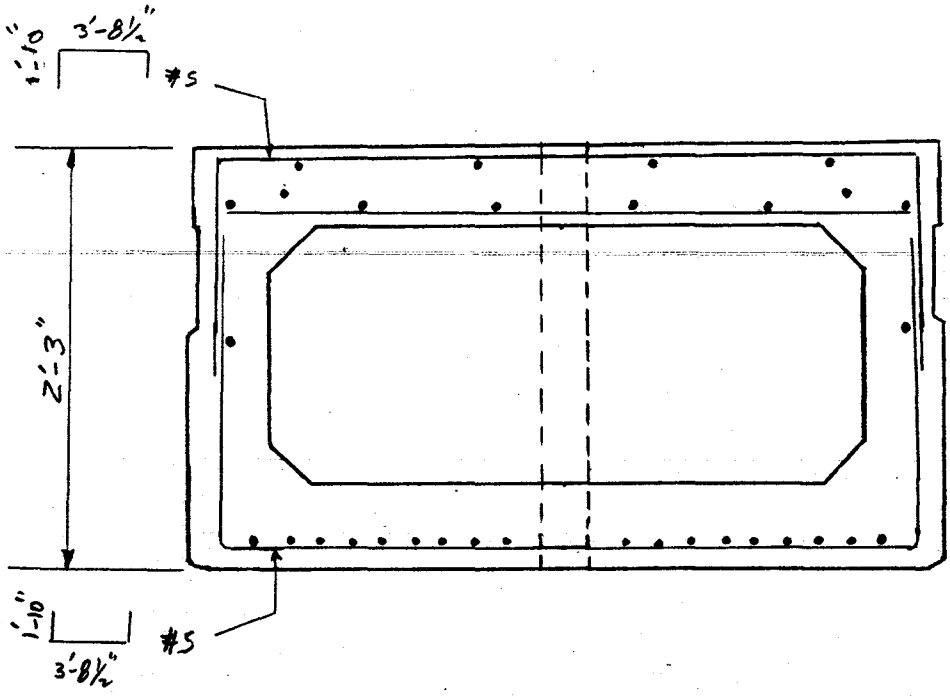
Page ____ of ____ 32

Made by ALI

Date 9-18-89

Checked by

Date



W

END SECTION

SCALE: 1" = 1'-0"

Digital Equipment Corporation - VAX/VMS Version V5.0-2

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NOTE: This Run is used for backchecking the Hand Calculation

12th street over ACDC
Longitudinal Analysis
TRIAL NO. 22 Page 10
(Post-Tension prog.)

Digital Equipment Corporation - VAX/VMS Version V5.0-2

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Digital Equipment Corporation - VAX/VMS Version V5.0-2

35

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00          040 2250550 55  0000050000000                 1  7311
   02000                          105      SBL 2*125 4/SE      7316
50  050  05 05                             1G20-44 TBUCK       7320
                                                               7321
101  5010 189 189 189            270  L0000 44     Y00031      7315
101  5010 182 182 182            270  L0000 44  570Y00031      7315
101  5010 182 182 182            270  L0000 40     Y00031      7315
```

N is USED
Kalkocking
/ Calculation

```
V   V   999     I    4  4    SSS   TTTTT  RRRR
V   V  9   9   II    4  4   S        T    R   R
V   V  9   9    I    4  4   S        T    R   R
V   V   9999    I    44444   SSS     T    RRRR
V   V      9    I       4       S    T    R R
 V V       9    I       4       S    T    R  R
  V     999    III      4   SSSS     T    R   R

  AAAAAA      CCCCCCCC  DDDDDDDD      CCCCCCCC
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    OOOOOO    UU      UU  TTTTTTTTTT
    OOOOOO    UU      UU  TTTTTTTTTT
  OO      OO  UU      UU      TT
  OO      OO  UU      UU      TT
  OO      OO  UU      UU      TT
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....  OOOOOO    UUUUUUUUUU      TT
....  OOOOOO    UUUUUUUUUU      TT

::::        II      SSSSSSSSSS
::::        II      SSSSSSSSSS
::::      IIII      SS
::::      IIII      SS
::::        II      SSSSSS
            II      SSSSSS
::::        II          SS
::::        II          SS
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::::        II            SS
::::        II      SS    SS
  ::        II      SS    SS
  ::      IIIIII      SSSSSS
::        IIIIII      SSSSSS
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Job ACDC (527) queued to SYS$PRINT on 14-SEP-1989 11:30 by user V914STR, UIC [STRUCTURAL,V914STR], under account 2245V914 at priority 100, started on printer _TXA0: on 14-SEP-1989 11:31 from queue SYS$PRINT.

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BEGINNING OF JOB

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BDL035 - BRIDGE ANALYSIS AND DESIGN CMS VERSION 1.0 REL. 1.20 ---- JAN. 7, 1986

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 1

FRAME DESCRIPTION

LINE NO.	MEM NO	JOINT NO. LT	JOINT NO. RT	END COND LT	END COND RT	DIR	SPAN	I	SUPPORT OR HINGE	E	DEAD LOAD UNIFORM	DEAD LOAD SEC	SKEW LT	SKEW RT	MEMBER SLOPE DEGREE	RECALL MEM
	1	1	2	R	P	G	50.2	0.00	0.0	4287.	0.000	150.	0.00	0.00	0.0	

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 2

SECTION PROPERTIES

LINE	MEM	LOC	RECALL	X	Y	SUPERSTRUCTURE WIDTH	SUPERSTRUCTURE DEPTH	SLAB THICKNESS TOP	SLAB THICKNESS BOTTOM	INT. GIRDER NO.	INT. GIRDER WEB	STORE
	1	0.0		0.0	0.0	4.0	2.25	5.50	5.50	0	0.	1

LT. EXT. GIRDER TYPE	LT. EXT. GIRDER WEB	LT. EXT. GIRDER FACTOR	RT. EXT. GIRDER TYPE	RT. EXT. GIRDER WEB	RT. EXT. GIRDER FACTOR	LT. OVERHANG LENGTH	LT. OVERHANG EXT.	LT. OVERHANG INT.	RT. OVERHANG LENGTH	RT. OVERHANG EXT.	RT. OVERHANG INT.
0	5.	0.00	0	5.	0.00	0.0	0.	0.	0.0	0.	0.

AREA	CENTROID LOCATION X	CENTROID LOCATION Y	MOMENT OF INERTIA ABOUT CENTROID X-X	MOMENT OF INERTIA ABOUT CENTROID Y-Y
4.89	2.00	1.14	3.21	6.71

MEMBER 1 PROPERTIES

LENGTH	MIN INERTIA	STIFFNESS LT	STIFFNESS RT	CARRY OVER LT	CARRY OVER RT
50.2	3.21	4.000	4.000	0.500	0.500

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 3

ANALYSIS DIAGNOSTICS

NO ERRORS FOUND

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN	I	SUPPORT OR HINGE	E	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT	F'C
1	1	2	R	P	G	50.2	3.21	0.0	4287.	0.000	0.000	0.000	0.000	5.00

STRUCTURE TYPE = P

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 4

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT	FIXED END MOMENTS RT	MEM NO	FIXED END MOMENTS LT	FIXED END MOMENTS RT	MEM NO	FIXED END MOMENTS LT	FIXED END MOMENTS RT
1	0.	0.						

****

**** CAUTION - RESULTS ARE UNFACTORED, UNLESS INDICATED OTHERWISE. ****

****

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 5

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	83.	148.	194.	222.	231.	222.	194.	148.	83.	0.

HORIZONTAL MEMBER STRESSES TRIAL 0 BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	-205.	-364.	-470.	-547.	-569.	-547.	-470.	-364.	-205.	0.

HORIZONTAL MEMBER STRESSES TRIAL 0 TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	206.	356.	468.	534.	557.	534.	468.	356.	206.	0.

HORIZONTAL MEMBER SHEARS TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	18.4	14.7	11.0	7.4	3.7	0.0	-3.7	-7.4	-11.0	-14.7	-18.4

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 6

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	0.001953	-0.001953						

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/10 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 0.038 0.073 0.100 0.117 0.123
0.117 0.100 0.073 0.038 0.000

*** MEMBER DEFLECTIONS BASED ON THE MODULUS OF ELASTICITY OF CONCRETE GIVEN BY USER. ******

*** MEMBER DEFLECTIONS HAVE BEEN MULTIPLIED BY A CONCRETE CREEP FACTOR OF 4.0 . ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 7

LOAD DATA TRIAL 1

LINE	MEM	W ORP	LOAD CODE	A	B	FIXED END MOMENTS LEFT	RIGHT	DELT	ANGLE	COMMENTS
1		0.200	U	0.0	0.0	0.	0.	10	0.0	SBL 2"125 #/SF
				ASSUMED DATA	50.2					

FIXED END MOMENTS TRIAL 1

MEM NO	FIXED END MOMENTS LT	RT	MEM NO	FIXED END MOMENTS LT	RT	MEM NO	FIXED END MOMENTS LT	RT
1	0.	0.						

IDENT ACDC PAGE 8

SIDESWAY DIAGNOSTICS

ERROR - FRAME MEMBER ARE ALL HORIZONT. OR ALL VERTICAL.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 9

*** SIDESWAY NOT CONSIDERED. SIDESWAY DIAGNOSTICS PRESENT. ***

HORIZONTAL MEMBER MOMENTS TRIAL 1

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	23.	40.	53.	60.	63.	60.	53.	40.	23.	0.

HORIZONTAL MEMBER STRESSES TRIAL 1 BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	-56.	-99.	-130.	-149.	-155.	-149.	-130.	-99.	-56.	0.

HORIZONTAL MEMBER STRESSES TRIAL 1 TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT

39

HORIZONTAL MEMBER SHEARS TRIAL 1

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	5.0	4.0	3.0	2.0	1.0	0.0	-1.0	-2.0	-3.0	-4.0	-5.0

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 10

TRIAL 1

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	0.000533	-0.000533						

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/10 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 0.010 0.020 0.027 0.032 0.033
0.032 0.027 0.020 0.010 0.000

*** MEMBER DEFLECTIONS BASED ON THE MODULUS OF ELASTICITY OF CONCRETE GIVEN BY USER. ******

*** MEMBER DEFLECTIONS HAVE BEEN MULTIPLIED BY A CONCRETE CREEP FACTOR OF 4.0 . ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 11

LIVE LOAD DIAGNOSTICS

NO ERRORS FOUND

SUPERSTRUCTURE LIVE LOAD

LINE NO.	MEM NO.	NUMBER OF LIVE LOAD LANES SUPERSTRUCTURE LT.END	RT.END	SUBSTRUCTURE LT.END	RT.END	RESISTING MOMENT OF UNIT STEEL POSITIVE	NEGATIVE	PLOT M & S ENV.	PLOT SCALE	DEFLECTION LINES	P13
1		0.500	0.500	0.5	0.5	0.	0.	0	0	NO	NO

LINE NO.	LIVE LOAD NO.	TRUCK P1	D1	P2	D2	P3	LANE UNIFORM	MOM. RIDER	SHEAR RIDER	IMPACT	NO. LL LNS.	LIVE LOAD SIDESWAY	COMMENTS
1.		8.0	14.0	32.0	14.0	32.0	0.640	18.0	26.0	YES	0.00	NO	

COMPUTED IMPACT FACTORS

MEMBER NO.	IMPACT FACTOR
1	20.5 %

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 12

LL NO. 1. NEGATIVE LIVE LOAD MOMENT ENVELOPE AND ASSOCIATED SHEARS

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SHEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

HORIZONTAL MEMBER STRESSES LL MAX NEG BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

HORIZONTAL MEMBER STRESSES LL MAX NEG TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 13

LL NO. 1. DEAD LOAD PLUS NEGATIVE LIVE LOAD MOMENT ENVELOPE

40

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	83.	148.	194.	222.	233.	222.	194.	148.	83.	0.

HORIZONTAL MEMBER STRESSES FOR DL+LL MAX NEG BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	-205.	-364.	-478.	-547.	-569.	-547.	-478.	-364.	-205.	0.

HORIZONTAL MEMBER STRESSES FOR DL+LL MAX NEG TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	200.	356.	468.	534.	557.	534.	468.	356.	200.	0.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 14

LL NO. 1. **POSITIVE LIVE LOAD MOMENT ENVELOPE AND ASSOCIATED SHEARS**

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	166.	285.	348.	399.	401.	399.	348.	285.	166.	0.
SHEAR	0.0	33.0	28.4	23.8	18.3	-18.7	-18.3	-23.8	-28.4	-33.0	0.0

HORIZONTAL MEMBER STRESSES LL MAX POS BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	-409.	-703.	-883.	-984.	-988.	-984.	-883.	-703.	-409.	0.

HORIZONTAL MEMBER STRESSES LL MAX POS TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	400.	688.	863.	962.	966.	962.	863.	688.	400.	0.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 15

LL NO. 1. DEAD LOAD PLUS POSITIVE LIVE LOAD MOMENT ENVELOPE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	249.	433.	552.	621.	632.	621.	552.	433.	249.	0.

HORIZONTAL MEMBER STRESSES FOR DL+LL MAX POS BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	-614.	-1067.	-1361.	-1530.	-1557.	-1530.	-1361.	-1067.	-614.	0.

HORIZONTAL MEMBER STRESSES FOR DL+LL MAX POS TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	600.	1044.	1331.	1496.	1523.	1496.	1331.	1044.	600.	0.

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LL NO. 1. **LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS**

MEMBER 1	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
POS. V	37.7	33.0	28.4	23.8	19.2	14.8	10.7	6.6	4.1	2.1	0.0
MOMENT	0.0	165.9	285.3	358.3	384.8	372.2	322.8	232.1	165.2	92.9	0.0
NEG. V	0.0	-2.1	-4.1	-6.6	-10.7	-14.8	-19.2	-23.8	-28.4	-33.0	-37.7
MOMENT	0.0	92.9	165.2	232.1	322.8	372.2	384.8	358.3	285.3	165.9	0.0
RANGE	37.7	35.1	32.5	30.4	29.9	29.7	29.9	30.4	32.5	35.1	37.7

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LL NO. 1. DEAD LOAD PLUS LIVE LOAD SHEAR ENVELOPE

MEMBER 1	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
POS. V	56.1	47.8	39.5	31.2	22.8	14.8	7.0	-0.8	-6.9	-12.7	-18.4
NEG. V	18.4	12.7	6.9	0.8	-7.0	-14.8	-22.8	-31.2	-39.5	-47.8	-56.1

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 18

LL NO. 1. LIVE LOAD SUPPORT RESULTS -- GRADE INCLUDED

	MAX. AXIAL LOAD			MAX. LONGITUDINAL MOMENT		
	AXIAL LOAD	MOMENT TOP	MOMENT BOT.	AXIAL LOAD	MOMENT TOP	MOMENT BOT.
SUPPORT JT. 1						
POSITIVE	37.7	0.	0.	0.0	0.	0.
NEGATIVE	0.0	0.	0.	0.0	0.	0.
SUPPORT JT. 2						
POSITIVE	37.7	0.	0.	0.0	0.	0.
NEGATIVE	0.0	0.	0.	0.0	0.	0.

THE RATIO OF SUBSTRUCTURE / SUPERSTRUCTURE LOADING IS 1.000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 19

INPUT PRESTRESSED DATA

TRIAL 20 FRAME 1 PATH 1

LINE NO.	MEM NO.	LLT	LLP	LRT	YLT	YLP	YRT	H	K
1		0.00	0.50	0.10	1.89	1.89	1.89	0.25	0.00020

XLT(FT) = 0.0 XRT(FT) = 0.0 STEEL STRESS(KSI) = 270. JACKING % = .75 JACKING ENDS = 1.

ANCHOR SET(IN); LEFT = 0.000 RIGHT = 0.000 CONC. STRENGTH(PSI) = 5000. ALLOW. TENSION(PSI) = -187.

P-JACK(KIPS) = 0. SHORTENING PERCENT= 0 TOTAL LOSSES(KSI) = 30

LO-LAX = YES RELATIVE HUMIDITY = 40 %

****
****** THE ANSWERS ARE UNCHECKED AND THE USER IS RESPONSIBLE FOR CHECKING THEM. ******
****

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 20

CABLE PATH OFFSETS

TRIAL 20 FRAME 1 PATH 1

MEMBER	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 21

CABLE PATH ECCENTRICITIES

TRIAL 20 FRAME 1 PATH 1

MEMBER	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.778	0.778	0.778	0.778	0.778	0.778	0.778	0.778	0.778	0.778	0.778

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FORCE COEFFICIENTS AFTER ANCHOR SET AND LONG TERM LOSSES

TRIAL 20 FRAME 1 PATH 1

MEMBER	LEFT	.1PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.847	0.846	0.845	0.844	0.843	0.842	0.841	0.840	0.839	0.838	0.837

THE POINT OF NO MOVEMENT IS IN SPAN 1, 50.20 FEET FROM THE LEFT END OF THE SPAN

THE LEFT ANCHOR SET LENGTH IS 0.0

THE RIGHT ANCHOR SET LENGTH IS 0.0

THE FORCE COEF. AT THE LEFT END IS 0.847

THE FORCE COEF. AT THE RIGHT END IS 0.837

*** THE FORCE COEFFICIENT AT THE POINT OF NO MOVEMENT BEFORE ANCHOR SET IS 0.990 *****

*** THE TOTAL LENGTH OF THIS CABLE PATH INCLUDING 4 FEET FOR JACKING IS 54.20 FEET. *****

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 23

SECONDARY MOMENTS

TRIAL 20 FRAME 1 PATH 1

FEM'S DUE TO SECONDARY EFFECTS BEFORE BALANCING

MEMBER	LEFT END	RIGHT END
1	0.000	0.000

FEM'S DUE TO SECONDARY EFFECTS

MEMBER	LEFT END	RIGHT END
1	0.000	0.000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 24

P/S MOMENT COEF.

TRIAL 20 FRAME 1 PATH 1

*** SIDESWAY NOT CONSIDERED. DEAD LOAD WAS NOT SWAYED. ***
ADJUSTED FOR LOSSES & SECONDARY MOMENTS BUT NO SHORTENING

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-0.6586	-0.6578	-0.6570	-0.6562	-0.6554	-0.6547	-0.6539	-0.6531	-0.6523	-0.6516	-0.6508

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TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1535.	1533.	1531.	1529.	1527.	1526.	1524.	1522.	1520.	1518.	1517.

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-209.	-208.	-208.	-208.	-208.	-207.	-207.	-207.	-207.	-206.	-206.

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TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1535.	1328.	1167.	1051.	981.	956.	977.	1044.	1156.	1313.	1517.

HORIZONTAL MEMBER STRESSES DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-209.	-8.	148.	260.	327.	349.	327.	261.	150.	-6.	-206.

*** THE ALLOWABLE TENSION HAS BEEN EXCEEDED AT TIME OF INITIAL STRESSING ***

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TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1535.	1272.	1067.	921.	832.	801.	828.	913.	1056.	1258.	1517.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-209.	47.	245.	387.	473.	501.	473.	388.	247.	49.	-206.

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TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX POS LL + I + P/S BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1535.	863.	364.	38.	-152.	-187.	-155.	30.	353.	849.	1517.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX POS LL + I + P/S TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-209.	447.	933.	1251.	1435.	1467.	1435.	1252.	934.	448.	-206.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 29

TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1535.	1272.	1067.	921.	832.	801.	828.	913.	1056.	1258.	1517.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-209.	47.	245.	387.	473.	501.	473.	388.	247.	49.	-206.

**** MIN PJACK = 543. KIPS CONC STRENGTH @ 28 DAYS = 3837. PSI @ STRESSING = 2790. PSI ****

***** THE ALLOWABLE TENSION HAS BEEN EXCEEDED IN THE FINAL STRESSES. *****

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TRIAL 20 FRAME 1 PATH 1

HORIZONTAL MEMBER MOMENTS DUE TO P/S

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-358.	-357.	-357.	-356.	-356.	-356.	-355.	-355.	-354.	-354.	-353.

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TRIAL 20 FRAME 1 PATH 1

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	-0.004519	0.004500						

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 -0.170 -0.226 -0.170 0.000

*** MEMBER DEFLECTIONS BASED ON THE MODULUS OF ELASTICITY OF CONCRETE GIVEN BY USER. ******

*** MEMBER DEFLECTIONS HAVE BEEN MULTIPLIED BY A CONCRETE CREEP FACTOR OF 4.0 . ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 32

INPUT PRESTRESSED DATA

TRIAL 21 FRAME 1 PATH 1

LINE NO.	MEM NO.	LLT	LLP	LRT	YLT	YLP	YRT	U	K
1		0.00	0.50	0.10	1.82	1.82	1.82	0.25	0.00020

XLT(FT) = 0.0 XRT(FT) = 0.0 STEEL STRESS(KSI) = 270. JACKING % = .75 JACKING ENDS = L

ANCHOR SET(IN): LEFT = 0.000 RIGHT = 0.000 CONC. STRENGTH(PSI) = 5000. ALLOW. TENSION(PSI) = -187.

P-JACK(KIPS) = 570. SHORTENING PERCENT= 0 TOTAL LOSSES(KSI) = 33

LO-LAX = YES RELATIVE HUMIDITY = 40 %

****

****** THE ANSWERS ARE UNCHECKED AND THE USER IS RESPONSIBLE FOR CHECKING THEM. ******

****

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CABLE PATH OFFSETS

TRIAL 21 FRAME 1 PATH 1

MEMBER	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82

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CABLE PATH ECCENTRICITIES

TRIAL 21 FRAME 1 PATH 1

MEMBER	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.708	0.708	0.708	0.708	0.708	0.708	0.708	0.708	0.708	0.708	0.708

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FORCE COEFFICIENTS AFTER ANCHOR SET AND LONG TERM LOSSES

TRIAL 21 FRAME 1 PATH 1

MEMBER	LEFT	.1PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.847	0.846	0.845	0.844	0.843	0.842	0.841	0.840	0.839	0.838	0.837

THE POINT OF NO MOVEMENT IS IN SPAN 1, 50.20 FEET FROM THE LEFT END OF THE SPAN

THE LEFT ANCHOR SET LENGTH IS 0.0

THE RIGHT ANCHOR SET LENGTH IS 0.0

THE FORCE COEF. AT THE LEFT END IS 0.847

THE FORCE COEF. AT THE RIGHT END IS 0.837

*** THE FORCE COEFFICIENT AT THE POINT OF NO MOVEMENT BEFORE ANCHOR SET IS 0.990 *****

*** THE TOTAL LENGTH OF THIS CABLE PATH INCLUDING 4 FEET FOR JACKING IS 54.20 FEET. *****

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 36

SECONDARY MOMENTS

TRIAL 21 FRAME 1 PATH 1

FEM'S DUE TO SECONDARY EFFECTS BEFORE BALANCING

MEMBER	LEFT END	RIGHT END
1	0.000	0.000

DEM'S DUE TO SECONDARY EFFECTS

MEMBER	LEFT END	RIGHT END
1	0.000	0.000

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P/S MOMENT COEF.

TRIAL 21 FRAME 1 PATH 1

*** SIDESWAY NOT CONSIDERED. DEAD LOAD WAS NOT SWAYED. ***
ADJUSTED FOR LOSSES & SECONDARY MOMENTS BUT NO SHORTENING

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-0.5993	-0.5986	-0.5979	-0.5972	-0.5964	-0.5957	-0.5950	-0.5943	-0.5936	-0.5929	-0.5922

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TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1528.	1526.	1524.	1522.	1520.	1519.	1517.	1515.	1513.	1511.	1510.

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-137.	-137.	-137.	-137.	-137.	-137.	-136.	-136.	-136.	-136.	-136.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 39

TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1528.	1321.	1160.	1044.	974.	949.	970.	1037.	1149.	1306.	1510.

HORIZONTAL MEMBER STRESSES DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-137.	63.	219.	331.	398.	420.	398.	331.	220.	64.	-136.

*** THE ALLOWABLE TENSION HAS BEEN EXCEEDED AT TIME OF INITIAL STRESSING ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 40

TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1528.	1265.	1060.	914.	825.	794.	821.	906.	1049.	1251.	1510.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-137.	118.	316.	458.	543.	572.	544.	459.	317.	119.	-136.

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TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX DEC LL + P/S BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1528.	856.	357.	31.	-159.	-194.	-163.	[illegible]	[illegible]	842.	1510.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX POS LL + I + P/S TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-137.	518.	1004.	1322.	1505.	1538.	1506.	1322.	1005.	519.	-136.

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TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1528.	1265.	1060.	914.	825.	794.	821.	906.	1049.	1251.	1510.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-137.	118.	316.	458.	543.	572.	544.	459.	317.	119.	-136.

**** MIN PJACK = 570. KIPS CONC STRENGTH @ 28 DAYS = 3844. PSI @ STRESSING = 2777. PSI ****

***** THE ALLOWABLE TENSION HAS BEEN EXCEEDED IN THE FINAL STRESSES. *****

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TRIAL 21 FRAME 1 PATH 1

HORIZONTAL MEMBER MOMENTS DUE TO P/S

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-342.	-341.	-341.	-340.	-340.	-340.	-339.	-339.	-338.	-338.	-338.

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TRIAL 21 FRAME 1 PATH 1

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	-0.004316	0.004299						

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4207. 0.000 -0.162 -0.216 -0.162 0.000

*** MEMBER DEFLECTIONS BASED ON THE MODULUS OF ELASTICITY OF CONCRETE GIVEN BY USER. ******

*** MEMBER DEFLECTIONS HAVE BEEN MULTIPLIED BY A CONCRETE CREEP FACTOR OF 4.0 . ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 45

INPUT PRESTRESSED DATA

TRIAL 22 FRAME 1 PATH 1

LINE NO.	MEM NO.	LLT	LLF	LRT	YLT	YLF	YRT	U	K
1		0.00	0.50	0.10	1.82	1.82	1.82	0.25	0.00020

XLT(FT) = 0.0 XRT(FT) = 0.0 STEEL STRESS(KSI) = 270. JACKING % = .75 JACKING END = L

ANCHOR SET(IN); LEFT = 0.000 RIGHT = 0.000 CONC. STRENGTH(PSI) = 5000. ALLOW. TENSION(PSI) = -170.

P-JACK(KIPS) = 0. SHORTENING PERCENT= 0 TOTAL LOSSES(KSI) = 30

[illegible]

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*****************************************************************************
****** THE ANSWERS ARE UNCHECKED AND THE USER IS RESPONSIBLE FOR CHECKING THEM. ******
*****************************************************************************
IDENT  ACDC                    BRIDGE ANALYSIS AND DESIGN                    PAGE  46
CABLE PATH OFFSETS
TRIAL 22   FRAME 1   PATH 1
MEMBER   LEFT    .1 PT   .2 PT   .3 PT   .4 PT   .5 PT   .6 PT   .7 PT   .8 PT   .9 PT   RIGHT
 1       1.82    1.82    1.82    1.82    1.82    1.82    1.82    1.82    1.82    1.82    1.82
IDENT  ACDC                    BRIDGE ANALYSIS AND DESIGN                    PAGE  47
CABLE PATH ECCENTRICITIES
TRIAL 22   FRAME 1   PATH 1
MEMBER    LEFT    .1 PT   .2 PT   .3 PT   .4 PT   .5 PT   .6 PT   .7 PT   .8 PT   .9 PT   RIGHT
  1      0.708   0.708   0.708   0.708   0.708   0.708   0.708   0.708   0.708   0.708   0.708
IDENT  ACDC                    BRIDGE ANALYSIS AND DESIGN                    PAGE  48
FORCE COEFFICIENTS AFTER ANCHOR SET AND LONG TERM LOSSES
TRIAL 22   FRAME 1   PATH 1
MEMBER   LEFT    .1PT    .2 PT   .3 PT   .4 PT   .5 PT   .6 PT   .7 PT   .8 PT   .9 PT   RIGHT
  1      0.847   0.846   0.845   0.844   0.843   0.842   0.841   0.840   0.839   0.838   0.837
THE POINT OF NO MOVEMENT IS IN SPAN  1,  50.20 FEET FROM THE LEFT END OF THE SPAN
THE LEFT ANCHOR SET LENGTH IS      0.0
THE RIGHT ANCHOR SET LENGTH IS     0.0
THE FORCE COEF. AT THE LEFT END IS   0.847
THE FORCE COEF. AT THE RIGHT END IS  0.837

*** THE FORCE COEFFICIENT AT THE POINT OF NO MOVEMENT BEFORE ANCHOR SET IS  0.990      *****

*** THE TOTAL LENGTH OF THIS CABLE PATH INCLUDING 4 FEET FOR JACKING IS    54.20 FEET. *****
IDENT  ACDC                    BRIDGE ANALYSIS AND DESIGN                    PAGE  49
SECONDARY MOMENTS
TRIAL 22   FRAME 1   PATH 1
                               FEM'S DUE TO SECONDARY EFFECTS BEFORE BALANCING
                              MEMBER          LEFT END       RIGHT END
                                1                0.000          0.000
                               DEM'S DUE TO SECONDARY EFFECTS
                              MEMBER          LEFT END       RIGHT END
                                1                0.000          0.000
IDENT  ACDC                    BRIDGE ANALYSIS AND DESIGN                    PAGE  50
P/S MOMENT COEF.
TRIAL  22 FRAME 1  PATH 1
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*** SIDESWAY NOT CONSIDERED. DEAD LOAD WAS NOT SWAYED. ***
ADJUSTED FOR LOSSES & SECONDARY MOMENTS BUT NO SHORTENING

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-0.5993	-0.5986	-0.5979	-0.5972	-0.5964	-0.5957	-0.5950	-0.5943	-0.5936	-0.5929	-0.5922

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TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1552.	1550.	1548.	1546.	1544.	1543.	1541.	1539.	1537.	1535.	1533.

HORIZONTAL MEMBER STRESSES PRESTRESS ONLY TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-140.	-139.	-139.	-139.	-139.	-139.	-139.	-138.	-138.	-138.	-138.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 52

TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1552.	1345.	1184.	1060.	999.	973.	994.	1061.	1173.	1330.	1533.

HORIZONTAL MEMBER STRESSES DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-140.	61.	217.	328.	395.	418.	396.	329.	218.	62.	-138.

*** THE ALLOWABLE TENSION HAS BEEN EXCEEDED AT TIME OF INITIAL STRESSING ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 53

TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1552.	1289.	1084.	938.	849.	818.	845.	930.	1073.	1274.	1533.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + P/S FOR TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-140.	116.	314.	456.	541.	570.	542.	457.	315.	117.	-138.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 54

TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX POS LL + I + P/S BOTTOM FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1552.	880.	381.	55.	-135.	-170.	-139.	47.	370.	866.	1533.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX POS LL + I + P/S TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-140.	515.	1002.	1319.	1503.	1536.	1503.	1320.	1003.	517.	-138.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 55

TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S BOTTOM FIBRE

MEM NO LEFT .1 PT .2 PT .3 PT .4 PT .5 PT .6 PT .7 PT .8 PT .9 PT RIGHT

49

1	1552.	1209.	1084.	938.	849.	818.	845.	930.	1073.	1274.	1533.

HORIZONTAL MEMBER STRESSES DL + ADDED DL + MAX NEG LL + I + P/S DUE TOP FIBRE

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-140.	116.	314.	456.	541.	570.	542.	457.	315.	117.	-138.

****** MIN PJACK = 579. KIPS CONC STRENGTH @ 28 DAYS = 3879. PSI @ STRESSING = 2821. PSI ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 56

TRIAL 22 FRAME 1 PATH 1

HORIZONTAL MEMBER MOMENTS DUE TO P/S

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	-347.	-347.	-346.	-346.	-345.	-345.	-345.	-344.	-344.	-343.	-343.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 57

TRIAL 22 FRAME 1 PATH 1

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	-0.004384	0.004367						

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 -0.165 -0.220 -0.165 0.000

*** MEMBER DEFLECTIONS BASED ON THE MODULUS OF ELASTICITY OF CONCRETE GIVEN BY USER. ******

*** MEMBER DEFLECTIONS HAVE BEEN MULTIPLIED BY A CONCRETE CREEP FACTOR OF 4.0 . ******

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 58

AASHTO LONG TERM PRESTRESS LOSSES

TOTAL LOSS = CKS + CKC + ES + SH.

TRIAL NO. 20

MEM NO	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
1	34.60	32.34	30.55	29.28	28.50	28.23	28.46	29.19	30.43	32.17	34.43

WARNING : THE LONG TERM PRESTRESS LOSSES SHOULD BE EXAMINED CAREFULLY. IF THE CALCULATED LOSSES DOES NOT AGREE WITH THE DEFAULT/INPUT LOSSES MORE THAN 15 PERCENT AT THE CRITICAL POINT. REDESIGN SHOULD BE CONSIDERED.

THE MEAN VALUE OF THE LONG TERM CALCULATED PRESTRESS LOSSES IS 30.74 KSI.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 59

SUMMARY OF HORIZONTAL MEMBER DEFLECTIONS FOR PRESTRESSED GIRDER

TRIAL NO. 20

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 -0.170 -0.220 -0.170 0.000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 60

ULTIMATE RESISTING MOMENT OF PRECTRESSED MEMBER

	MOM. TYPE	FACTORED APL. MOM (K-FT)	ULTIMATE MON. (K-FT)	FSU @ COR MIN. (KSI)	MAX.	NEUTRAL AXIS (IN)	MILD STEEL (SQ.IN)	COMBINED STEEL INDEX
TRIAL NO. 20 MEMBER NO. : 1								
LOCATION :								
0.0 PT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000
0.1 PT.	POS	497.7	1165.3	261.92	261.92	4.30	0.00	0.129
0.2 PT.	POS	864.0	1165.3	261.91	261.91	4.30	0.00	0.129
0.3 PT.	POS	1090.8	1165.3	261.91	261.91	4.30	0.00	0.129
0.4 PT.	POS	1233.5	1238.0	261.52	261.52	4.56	0.71	0.136
0.5 PT.	POS	1252.3	1257.8	261.39	261.39	4.63	0.91	0.138
0.6 PT.	POS	1233.5	1238.0	261.51	261.51	4.56	0.72	0.136
0.7 PT.	POS	1090.8	1165.2	261.90	261.90	4.30	0.00	0.129
0.8 PT.	POS	864.0	1165.2	261.89	261.89	4.30	0.00	0.129
0.9 PT.	POS	497.7	1165.2	261.89	261.89	4.30	0.00	0.129
1.0 PT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000

*** THE ULTIMATE MOMENT IS BASED ON THE 5.00 KSI CONCRETE STRENGTH. ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 61

SHEAR MODIFICATION FACTOR -- GIRDER / BENT CAP

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 62

AASHTO STIRRUP DESIGN FOR SHEAR -- (GIRDER / BENT CAP)

MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
APPLIED SHEAR FORCE VN :	62.	53.	44.	35.	26.	18.	26.	35.	44.	53.	62.
CONC. SH RESISTANCE VC :	50.	50.	42.	28.	19.	14.	19.	28.	42.	50.	50.
WIDTH OF GIRD WEB (IN) :	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.
STIRRUP DESIGN SHR. VS :	12.	3.	3.	8.	7.	4.	7.	8.	3.	3.	12.
AREA OF STEEL REQ'D AV : (TOTAL STEEL GRADE 40)	0.16	0.04	0.03	0.10	0.10	0.06	0.10	0.10	0.04	0.04	0.16

NO.	DESCRIPTION	SUGGESTED SPACING @ 10TH POINT FOR DIFFERENT STIRRUP SIZE

51

#4	SPACING	INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#5	SPACING	INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#6	SPACING	INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.

*** NO GIRDER FLANGE REQUIRED FOR BOTH ENDS OF THIS MEMBER. ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 63

AASHTO LONG TERM PRESTRESS LOSSES

TOTAL LOSS = CRS + CRC + ES + SH.

TRIAL NO. 21

MEM NO	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
1	33.82	31.76	30.13	28.97	28.26	28.01	28.22	28.89	30.01	31.50	33.65

WARNING : THE LONG TERM PRESTRESS LOSSES SHOULD BE EXAMINED CAREFULLY. IF THE CALCULATED LOSSES DOES NOT AGREE WITH THE DEFAULT/INPUT LOSSES MORE THAN 15 PERCENT AT THE CRITICAL POINT, REDESIGN SHOULD BE CONSIDERED.

THE MEAN VALUE OF THE LONG TERM CALCULATED PRESTRESS LOSSES IS 30.30 KSI.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 64

SUMMARY OF HORIZONTAL MEMBER DEFLECTIONS FOR PRESTRESSED GIRDER

TRIAL NO. 21

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 -0.162 -0.216 -0.162 0.000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 65

ULTIMATE RESISTING MOMENT OF PRESTRESSED MEMBER

	MOM. TYPE	FACTORED APL. MOM (K-FT)	ULTIMATE MOM. (K-FT)	ESU & CGS MIN. (KSI)	ESU & CGS MAX. (KSI)	NEUTRAL AXIS (IN)	MILD STEEL (SQ.IN)	COMPUTED STEEL INDEX
TRIAL NO. 21 MEMBER NO. : 1								
LOCATION :								
0.0 PT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000
0.1 PT.	POS	497.7	1166.4	261.21	261.21	4.51	0.00	0.140
0.2 PT.	POS	864.0	1166.3	261.21	261.21	4.51	0.00	0.140
0.3 PT.	POS	1092.0	1166.3	261.20	261.20	4.51	0.00	0.140

0.4	FT.	POS	1233.5	1237.5	260.80	260.80	4.76	1.06	0.147
0.5	FT.	POS	1252.3	1257.3	260.66	260.66	4.83	1.36	0.149
0.6	FT.	POS	1233.5	1237.5	260.79	260.79	4.76	1.06	0.147
0.7	FT.	POS	1098.8	1166.3	261.18	261.18	4.50	0.00	0.140
0.8	FT.	POS	864.0	1166.2	261.18	261.18	4.50	0.00	0.140
0.9	FT.	POS	497.7	1166.2	261.17	261.17	4.50	0.00	0.140
1.0	FT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000

*** THE ULTIMATE MOMENT IS BASED ON THE 5.00 KSI CONCRETE STRENGTH. ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 66

SHEAR MODIFICATION FACTOR -- GIRDER / BENT CAP

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 67

AASHTO STIRRUP DESIGN FOR SHEAR -- (GIRDER / BENT CAP)

MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
APPLIED SHEAR FORCE VN :	62.	53.	44.	35.	26.	18.	26.	35.	44.	53.	62.
CONC. SH RESISTANCE VC :	49.	49.	42.	27.	19.	13.	19.	27.	41.	49.	49.
WIDTH OF GIRD WEB (IN) :	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.
STIRRUP DESIGN SHR. VS :	13.	4.	3.	8.	8.	5.	8.	8.	3.	4.	13.
AREA OF STEEL REQ'D AV :	0.18	0.05	0.04	0.11	0.11	0.06	0.11	0.11	0.04	0.06	0.18

(TOTAL STEEL GRADE 40)

NO.	DESCRIPTION	SUGGESTED SPACING @ 10TH POINT FOR DIFFERENT STIRRUP SIZE										
#4	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#5	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#6	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.

*** NO GIRDER FLARE REQUIRED FOR BOTH ENDS OF THIS MEMBER. ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 68

AASHTO LONG TERM PRESTRESS LOSSES

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TOTAL LOSS = CRS + CRC + ES + SH.

TRIAL NO. 22

MEM NO	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
1	34.14	32.08	30.46	29.29	28.59	28.34	28.54	29.20	30.33	31.91	33.97

WARNING : THE LONG TERM PRESTRESS LOSSES SHOULD BE EXAMINED CAREFULLY. IF THE CALCULATED LOSSES DOES NOT AGREE WITH THE DEFAULT/INPUT LOSSES MORE THAN 15 PERCENT AT THE CRITICAL POINT, REDESIGN SHOULD BE CONSIDERED.

THE MEAN VALUE OF THE LONG TERM CALCULATED PRESTRESS LOSSES IS 30.62 KSI.

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 69

SUMMARY OF HORIZONTAL MEMBER DEFLECTIONS FOR PRESTRESSED GIRDER

TRIAL NO. 22

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1 E= 4287. 0.000 -0.165 -0.220 -0.165 0.000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 70

ULTIMATE RESISTING MOMENT OF PRESTRESSED MEMBER

	MOM. TYPE	FACTORED APL. MOM (K-FT)	ULTIMATE MOM. (K-FT)	FSU @ CGS MIN. (KSI)	FSU @ CGS MAX. (KSI)	NEUTRAL AXIS (IN)	MILD STEEL (SQ.IN)	COMBINED STEEL INDEX
TRIAL NO. 22 MEMBER NO. : 1								
LOCATION :								
0.0 FT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000
0.1 FT.	POS	497.7	1182.5	261.07	261.07	4.57	0.00	0.142
0.2 FT.	POS	864.0	1182.5	261.06	261.06	4.57	0.00	0.142
0.3 FT.	POS	1098.8	1182.5	261.06	261.06	4.57	0.00	0.142
0.4 FT.	POS	1233.5	1236.7	260.77	260.77	4.77	0.80	0.148
0.5 FT.	POS	1252.3	1256.5	260.64	260.64	4.84	1.10	0.150
0.6 FT.	POS	1233.5	1236.7	260.76	260.76	4.77	0.80	0.148
0.7 FT.	POS	1098.8	1182.4	261.04	261.04	4.57	0.00	0.142
0.8 FT.	POS	864.0	1182.4	261.04	261.04	4.57	0.00	0.142
0.9 FT.	POS	497.7	1182.4	261.03	261.03	4.57	0.00	0.142
1.0 FT.		0.0	0.0	0.00	0.00	0.00	0.00	0.000

*** THE ULTIMATE MOMENT IS BASED ON THE 5.00 KSI CONCRETE STRENGTH. ***

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 71

SHEAR MODIFICATION FACTOR -- GIRDER / BENT CAP

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

IDENT ACDC BRIDGE ANALYSIS AND DESIGN PAGE 72

AASHTO STIRRUP DESIGN FOR SHEAR -- (GIRDER / BENT CAP)

MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3 PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
APPLIED SHEAR FORCE VN :	62.	53.	44.	35.	26.	18.	26.	35.	44.	53.	62.
CONC. SH RESISTANCE VC :	50. 100	50. 100	42. 84	28. 56	19. 38	13. 26	19.	28.	42.	50.	50.
WIDTH OF GIRD WEB (IN) :	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.	5.
STIRRUP DESIGN SHR. VS :	13.	4.	2.	8.	7.	4.	8.	8.	3.	4.	13.
AREA OF STEEL REQ'D AV : (TOTAL STEEL GRADE 40)	0.17	0.05	0.03	0.11	0.10	0.05	0.10	0.11	0.04	0.05	0.18

NO.	DESCRIPTION	SUGGESTED SPACING @ 10TH POINT FOR DIFFERENT STIRRUP SIZE										
#4	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#5	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.
#6	SPACING INCH	12.	12.	18.	18.	18.	18.	18.	18.	18.	12.	12.

*** NO GIRDER FLARE REQUIRED FOR BOTH ENDS OF THIS MEMBER. ***

```
*****************************************
**  SUMMARY OF INPUT CARDS PROCESSED.  **
*****************************************
              NO. OF A  CARDS  :  1
              NO. OF B  CARDS  :  1
              NO. OF D  CARDS  :  1
              NO. OF G  CARDS  :  3
      ----------------------------------
          TOTAL NO. OF CARDS  =   6
```

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```
V   V   999     1     4  4    SSSS  TTTTT  RRRR
V   V  9   9   11     4  4   S        T    R   R
V   V  9   9    1     4  4   S        T    R   R
V   V   9999    1     44444   SSS     T    RRRR
V   V       9   1        4       S    T    R R
 V V        9   1        4       S    T    R  R
  V      999   111       4    SSSS    T    R   R
```

```
  AAAAAA      CCCCCCCC  DDDDDDDD      CCCCCCCC
  AAAAAA      CCCCCCCC  DDDDDDDD      CCCCCCCC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AAAAAAAAAA  CC         DD      DD  CC
 AAAAAAAAAA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA  CC         DD      DD  CC
 AA      AA    CCCCCCCC  DDDDDDDD      CCCCCCCC
 AA      AA    CCCCCCCC  DDDDDDDD      CCCCCCCC
```

```
PPPPPPPP      SSSSSSSS
PPPPPPPP      SSSSSSSS
PP      PP  SS
PP      PP  SS
PP      PP  SS
PP      PP  SS
PPPPPPPP      SSSSSS
PPPPPPPP      SSSSSS
PP                  SS
PP                  SS
PP                  SS
PP                  SS
PP          SSSSSSSS
PP          SSSSSSSS

        11        333333
        11        333333
      1111      33      33
      1111      33      33
        11              33
        11              33
        11            33
        11            33
        11              33
        11              33
        11      33      33
        11      33      33
      111111      333333
      111111      333333
```

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MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
FSHRH LL FACTOR SHEAR:	82.	72.	62.	52.	42.	-32.	-42.	-52.	-62.	-72.	-82.
FSHRP LL FACTOR SHEAR:	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
FDLS DL FACTOR SHEAR:	24.	19.	14.	10.	5.	0.	-5.	-10.	-14.	-19.	-24.
DLS1 SDL UNFACT SHEAR:	5.	4.	3.	2.	1.	0.	-1.	-2.	-3.	-4.	-5.
VM SECONDARY SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
VP P/S CABLE SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
FSHRH LL FACTOR SHEAR:	82.	72.	62.	52.	42.	-32.	-42.	-52.	-62.	-72.	-82.
FSHRP LL FACTOR SHEAR:	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
FDLS DL FACTOR SHEAR:	24.	19.	14.	10.	5.	0.	-5.	-10.	-14.	-19.	-24.
DLS1 SDL UNFACT SHEAR:	5.	4.	3.	2.	1.	0.	-1.	-2.	-3.	-4.	-5.
VM SECONDARY SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
VP P/S CABLE SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

MEMBER NO. : 1	LEFT	.1 PT.	.2 PT.	.3PT.	.4 PT.	.5 PT.	.6 PT.	.7 PT.	.8 PT.	.9 PT.	RIGHT
FSHRH LL FACTOR SHEAR:	82.	72.	62.	52.	42.	-32.	-42.	-52.	-62.	-72.	-82.
FSHRP LL FACTOR SHEAR:	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
FDLS DL FACTOR SHEAR:	24.	19.	14.	10.	5.	0.	-5.	-10.	-14.	-19.	-24.
DLS1 SDL UNFACT SHEAR:	5.	4.	3.	2.	1.	0.	-1.	-2.	-3.	-4.	-5.
VM SECONDARY SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
VP P/S CABLE SHEAR :	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

Abutment Design

Parsons Brinckerhoff **Computation Sheet**

Page ____ of ____ 58

Made by ALI

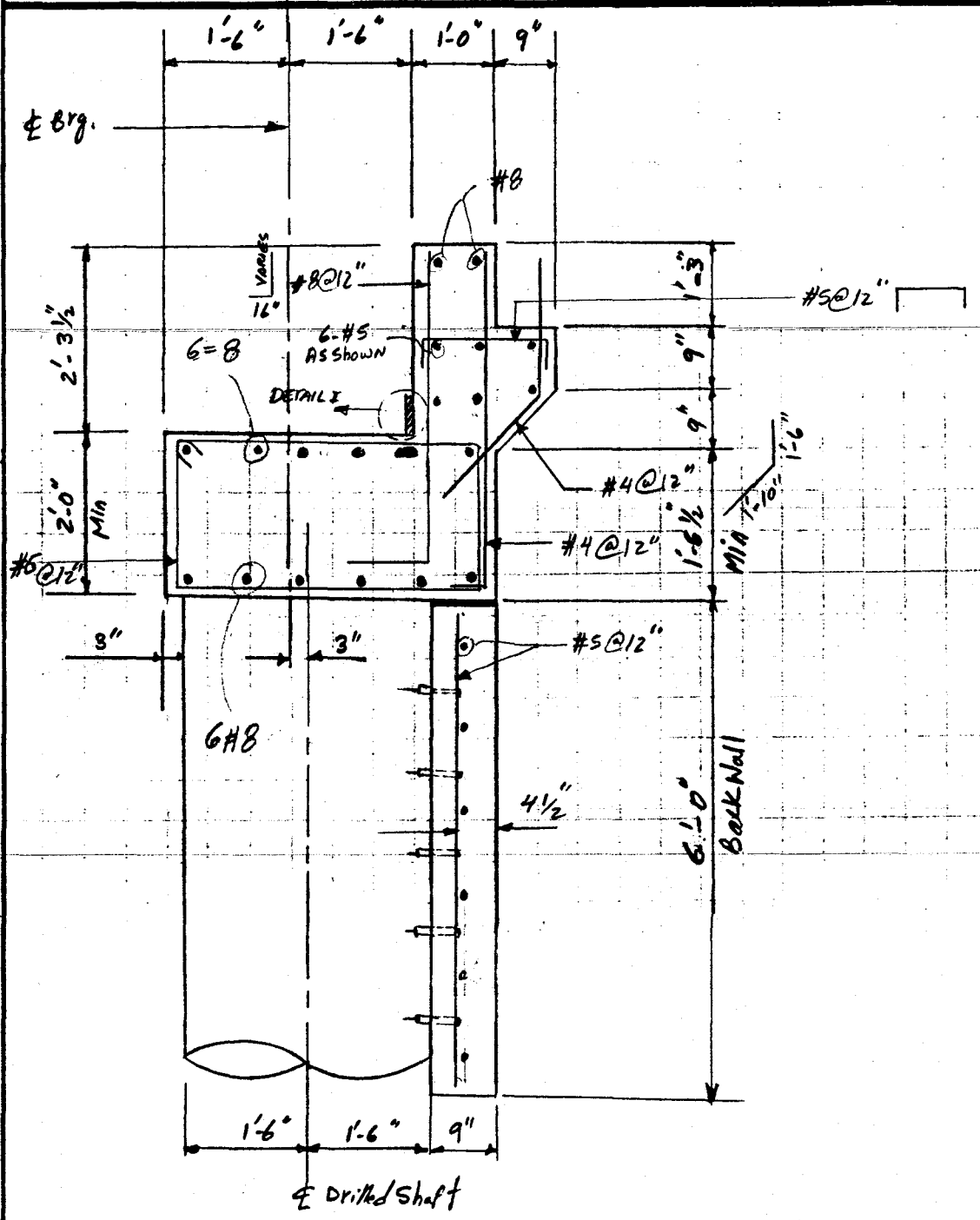
Date 9-26-89

Checked by

Date

Subject 12th street over ACDC

ABUTMENT SECTION



SECTION

Scale: 20" = 1'-0"

eet

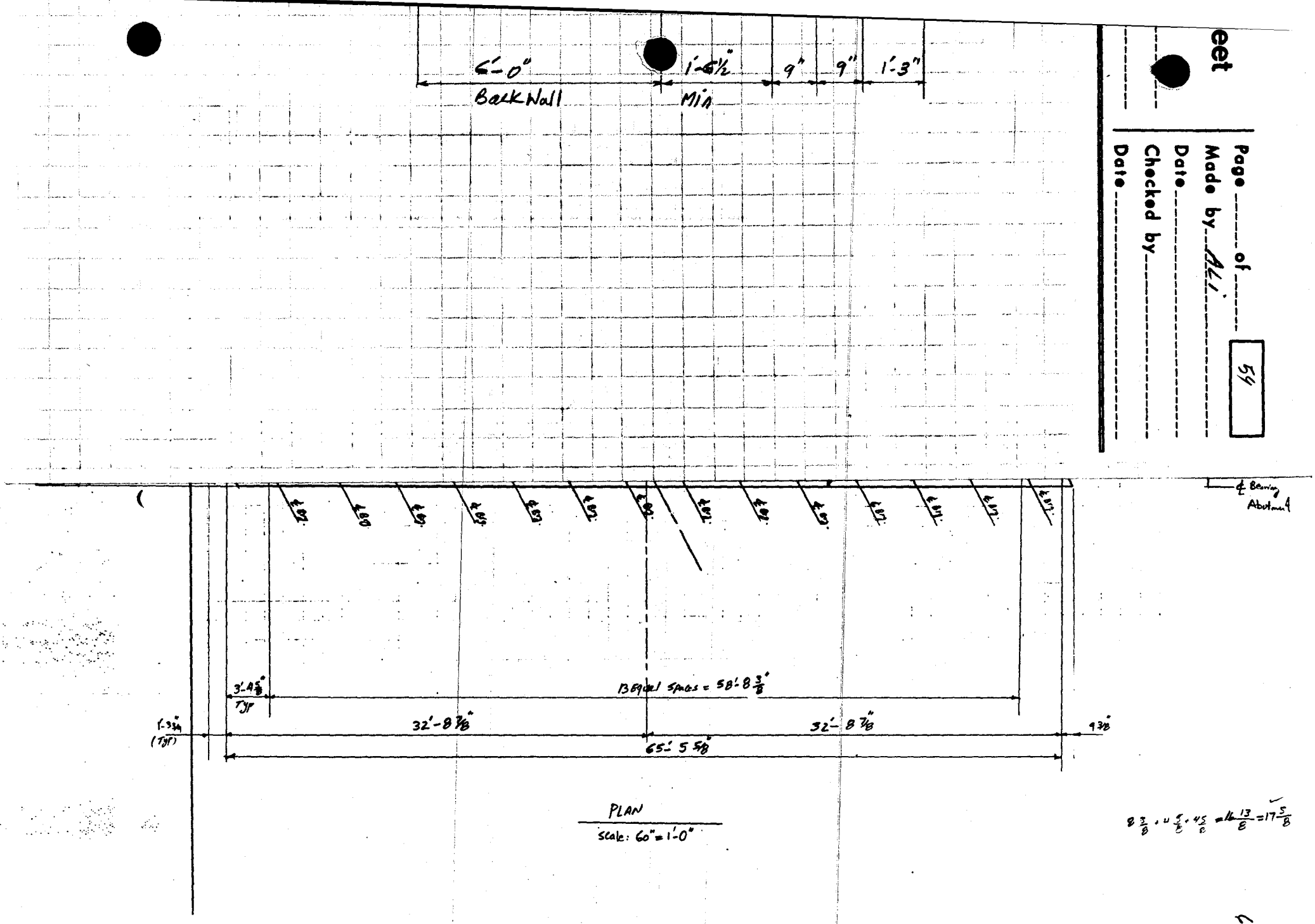
Page ____ of ____ 59

Made by AKI

Date

Checked by

Date



PLAN

Scale: 60" = 1'-0"

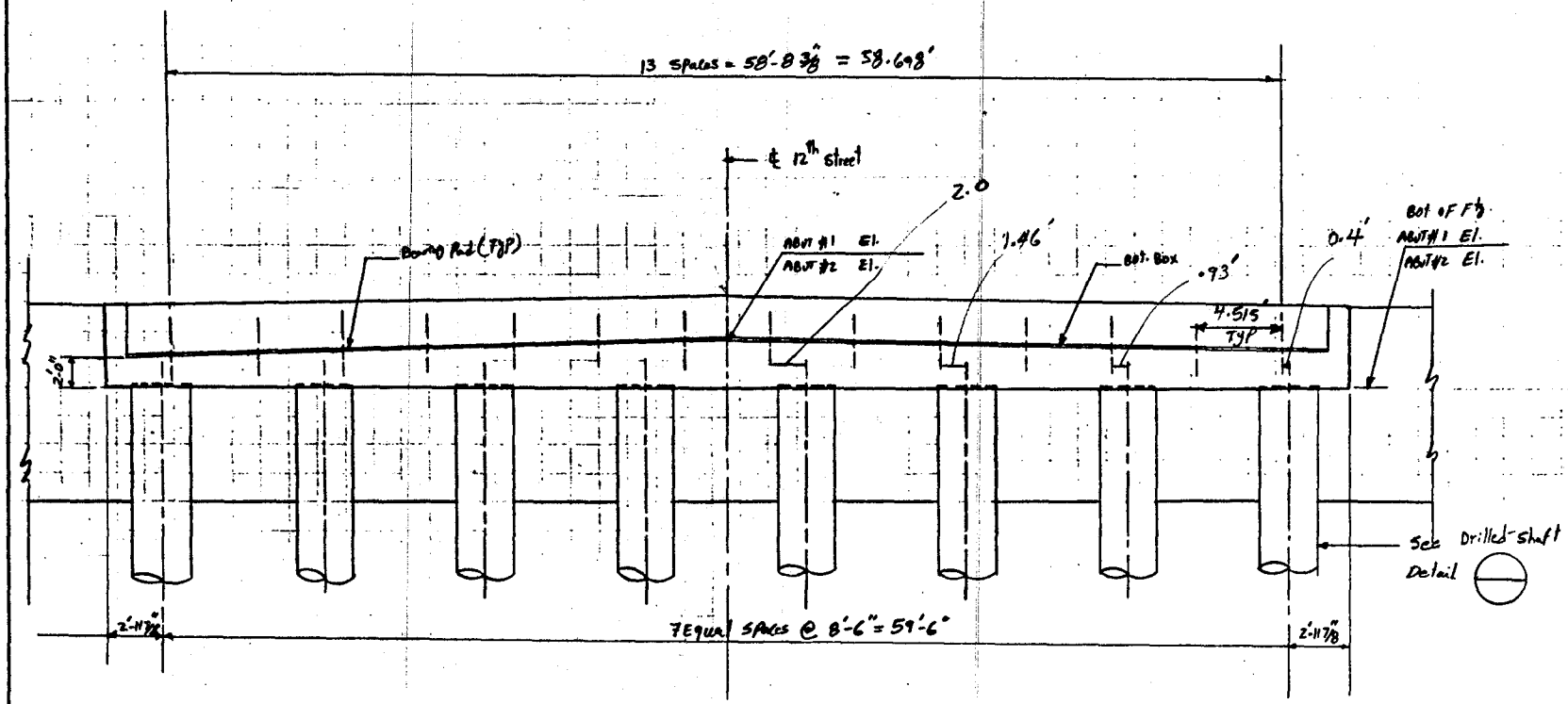
$8\frac{3}{8} + 4\frac{5}{8} + 4\frac{5}{8} = 16\frac{13}{8} = 17\frac{5}{8}$

60

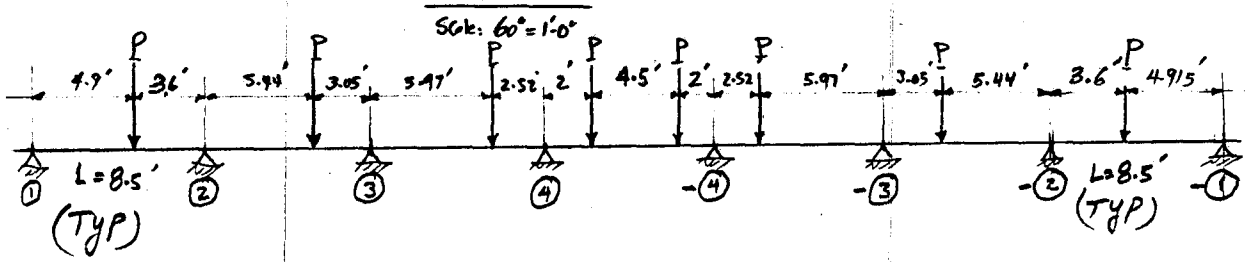
Parsons Brinckerhoff Computation Sheet

Subject: 12th street over ACDC
ABUTMENT ELEVATION DRWG

Page ____ of ____
Made by: Ali
Date: 9-21-89
Checked by: ____
Date: ____



ELEVATION
Scale: 60" = 1'-0"



Made by ALi

Date 9-25-89

Checked by W V ELLIOTT

Date 10-26-89

Subject 12th street OVER ACDC

Abutment Design

Three moment EQS. (slope Deflection) method

② $\cancel{M_1 \frac{L}{6EI}}^{0} + M_2\left(\frac{L}{3EI}+\frac{L}{3EI}\right) + M_3\left(\frac{L}{6EI}\right) = -\Sigma\tau_2 = \frac{-P(8.5)^2}{EI}\left[\frac{3.6\times4.9}{8.5^2}\left(1+\frac{4.9}{8.5}\right)\frac{1}{6} + \frac{5.44\times3.05}{8.5^2}\left(1+\frac{5.44}{8.5}\right)\frac{1}{6}\right]$

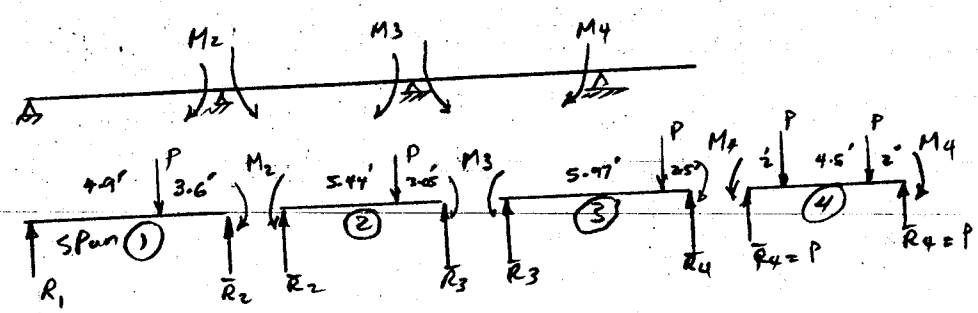
③ $M_2\frac{L}{6EI} + M_3\left(\frac{L}{3EI}+\frac{L}{3EI}\right) + M_4\frac{L}{6EI} = -\Sigma\tau_3 = \frac{-P(8.5)^2}{EI}\left[\frac{3.05\times5.44}{8.5^2}\left(1+\frac{5.44}{8.5}\right)\frac{1}{6} + \frac{5.97\times2.52}{8.5^2}\left(1+\frac{2.52}{8.5}\right)\frac{1}{6}\right]$

④ $M_3\frac{L}{6EI} + M_4\left(\frac{L}{3EI}+\frac{L}{3EI}\right) - M_4\frac{L}{6EI} = -\Sigma\tau_4 = \frac{-P(8.5)^2}{EI}\left[\frac{5.97\times2.52}{8.5^2}\left(1+\frac{5.97}{8.5}\right)\frac{1}{6} + \frac{2.5\times6.5}{8.5^2}\left(1+\frac{6.5}{8.5}\right)\frac{1}{6} + \frac{6.5\times2}{8.5^2}\left(1+\frac{2}{8.5}\right)\frac{1}{6}\right]$

$M_2\left(\frac{2}{3}L\right) + M_3\left(\frac{L}{6}\right) = -P(9.17)$ $\qquad M_2 = -12.34/L = -1.45\,P^{k\text{-}'}$ ✓

$M_2\left(\frac{L}{6}\right) + M_3\left(\frac{2}{3}L\right) + M_4\left(\frac{L}{6}\right) = -P(7.786)$ $\qquad M_3 = -5.64/L = -.663\,P^{k\text{-}'}$ ✓

$M_3\left(\frac{L}{6}\right) + M_4\left(\frac{5}{6}L\right) = -P(10.768)$ $\qquad M_4 = -11.79/L = -1.387\,P^{k\text{-}'}$ ✓



$R_1 = \frac{P\,3.6 - 1.45P}{8.5} = \boxed{0.253P}$ $\qquad \bar{R}_2 = (1-.253)P = \boxed{.747P}$ Span ①

$\bar{R}_2 = \frac{(1.45-.663)P + P(3.05)}{8.5} = \boxed{0.451P}$ $\qquad \bar{R}_3 = P - 0.451P = \boxed{0.549P}$ Span ②

$\bar{R}_3 = \frac{(1.663-1.387)P + P(2.52)}{8.5} = \boxed{0.2113P}$ $\qquad \bar{R}_4 = (1-.2113)P = .7887P$ ③

$\bar{R}_4 = P$ Span ④

$R_1 = 0.253P$

$R_2 = (0.747+0.451)P = 1.2P$

$R_3 = (0.549+0.2113)P = 0.76P$

$R_4 = (.7887+1)P = 1.789P$

$\overset{+}{M}_{@\,span\,①} = 0.253P(4.9') = 1.24P^{k\text{-}'}$

$\overset{+}{M}_{@\,span\,②} = 0.451P(5.44') - 1.45P = 1.00P^{k\text{-}'}$

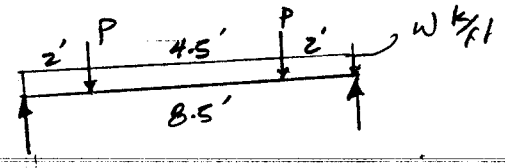
$\overset{+}{M}_{@\,span\,③} = 0.2113P(5.97') - .663P = 0.6P^{k\text{-}'}$

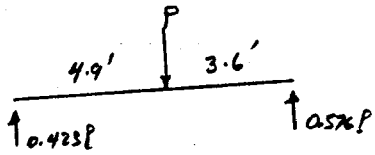
$\overset{+}{M}_{@\,span\,④} = P(2') - 1.387P = 0.613P^{k\text{-}'}$

Subject 12th street OVER ACDC

ABUTMENT DESIGN

Positive MOMENT





DUE TO Concentrated Load $M = P\left(\frac{8.5}{2}\right) - P(2.25) = 2P$ k-' @ span (4)

$M = 0.423P(4.9') = 2.07P$ k-' @ span (1)

DUE TO UNIFORM Load $M = \frac{WL^2}{8} = \frac{W(8.5)^2}{8} = 9.03W$ k-' all spans

DUE TO SETTEL. $M = 12.9$ k-' @ span (1) see Page 3 ABUT-ACDC

NEGATIVE MOMENT

DUE TO Concentrated Load $M = 1.45P$ k-' @ SUPPORT #2

$M = 0.663P$ @ #3

DUE TO UNIFORM Load $M = 7.63W$ k-'/ft @ SUPPORT #2

$M = 5.597W$ @ #3

DUE TO SETTELMENT $M = 56.48 \times 0.5 = 28.4$ k-' @ #3

DUE TO SETTELMENT $M = 16.3 \times 0.5'' = 8.2$ k-' @ #2

AISC 2-127

#2 DL =	$-.100WL^2$	Case 36
#2 LL =	$-.1205WL^2$	Case 37

LOADS

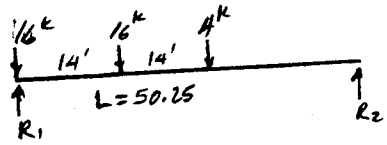
Weight of ABUT $= \left[1(2.5) + .75^2 + \frac{.75^2}{2} + 2(4)\right] .15 = 1.71$ k/ft $= W$

* FOR Positive Moment assume 2 Truck Adj. to each other, P is equal to half TRUCK

$P = \frac{72}{2} + \frac{422}{14} = 66.1^k$ (442k DL see PRELIM. DESIGN)

* FOR Negative Moment DL = 422k/ABUT, see PRELIM. DESIGN By Ali & LOAD the first TWO spans of the Beam.

$W_{DL} = \frac{422^k}{14 \times \frac{4'}{\cos 15.4°}} + 1.71 = \frac{422}{14 \times \frac{4}{.966}} + 1.71 = 8.38$ k/ft



$R_{LL} = 16 + 16\left(\frac{50.25-14}{50.25}\right) + 4\left(\frac{50.25-28}{50.25}\right) = 29.31^k$ USE 0.6 Truck Wheel

$W_{LL} = 29.31 \times \frac{0.6}{.5} \times \frac{1}{4'} = 8.8$ k/ft

Subject 12th street over ACDC Abutment Design

BENDING

$\bar{M}_{SOIL} = 18.2^{k'}$ from prev. page

first interior support

$\bar{M}_{DL} = .1WL^2 = .1(8.38)(8.5)^2 = 60.58$

$\bar{M}_{LL} = .1205WL^2 = .1205(8.8)(8.5)^2 = 76.6^{k'}$

$\bar{M}_{TOT} = 8.2 + 60.6 + 76.6 = 145.3^{k'}$

* AISC 2-127 (36) (37)

Max positive moment $= 2.07(66.1) + 9.03(1.71) + 12.9 = 165.2^{k'} = \overset{+}{M}$

$d = 24'' - 3.5 = 20.5''$

$A_s = \dfrac{165.2 \times 12}{.87(20)(20.5)} = 5.55 \text{ in}^2$ or 4.63 in^2 (60 grade) USE $6\#8 = 4.71 \text{ in}^2$ cont. (BOT)

$\bar{A}_s = \dfrac{145.3 \times 12}{.87(24)(20.5)} = 4.07 \text{ in}^2$; $M_n > 1.2 M_{cr}$ see below USE $6\#8$ @ top cont.

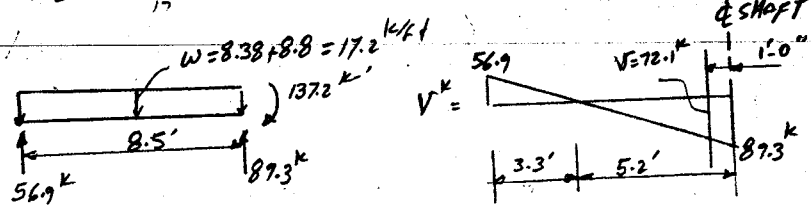
NOTE: The reinforcement @ face of support will be even less.

$M_{cr} = 7.5\dfrac{\sqrt{3000}}{1000}\dfrac{I_g}{y_c} = \dfrac{7.5\sqrt{3000}}{12000}\dfrac{48(24)^3}{12}\cdot\dfrac{1}{12} = 157.7^{k'}$

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{4.71 \times 60}{.85(3)(48)} = 2.31 \text{ in}$

$M_n = .9 A_s f_y \left(d - \dfrac{a}{2}\right) = .9 \times 4.71 \times 60\left(20.5 - \dfrac{2.31}{2}\right) = 710^{k'} > 1.2 M_{cr}$

SHEAR



$V_{@1'} = 72.1^k$

$V_c = 0.95\sqrt{f'_c}\,bd = 0.95\dfrac{\sqrt{3000}}{1000}(48)(24)(0.8) = 48^k$

$V_s = 72.1 - 48^k = 24.1^k$

$A_v = \dfrac{24.1}{2d(24)(0.8)} \times 12 = 0.75 \text{ in}^2/\text{ft}$ $\quad 2\#6 @ 12'' = 0.88 \text{ in}^2/\text{ft}$

$A_{v\,min} = \dfrac{50(48)(12)}{40{,}000} = 0.72 \text{ in}^2 < 0.88 \text{ in}^2$ ok

Parsons Brinckerhoff **Computation Sheet**

Page ____ of ____ 65

Made by A.Li

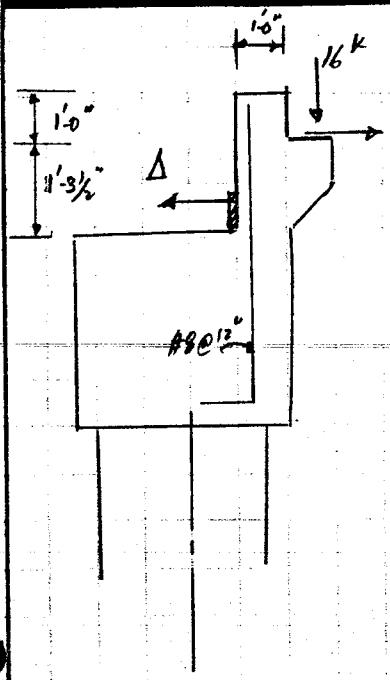
Date 10-25-89

Checked by

Date

Subject 12th street OVER ACDC

Back wall OF ABUTMENT DESIGN



NOTE: There is no Anchor bar between the Backwall & Approach slab

$MN = 0.5 \times 16 = 8^k$

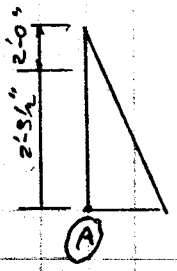
$2\Delta = \epsilon L \Delta T = 0.0006 \times 50.25' \times \frac{40°}{100} \times 12 = 0.145''$ TOTAL Temp Exp & Cont.

$\Delta = \frac{0.145}{2} = 0.072''$ / ABUTMET

FRICTION Force on The Approach slab resisting contraction is $MN = 8^k$

$M = 1.25(8) = 10^{k'}$

$A_s = \frac{10 \times 12}{.87(24)(9.5)} = 0.61 \text{ in}^2/\text{ft}$ USE #8@12"



BENDING DUE TO SOIL Pressure

$M_A = \frac{(4.25)^3}{6} \times 0.031 = 0.4^{k'/ft}$

$A_s = \frac{0.4 \times 12}{.87(20)(9.5)} = 0.029 \text{ in}^2/\text{ft}$ USE #4@12"

Parsons Brinckerhoff Computation Sheet

Page ____ of ____ 66

Subject: 12th over ACDC

Out PUT: ABUT-ACDC

Made by: W VELLIOTT

Date: 10-16-89

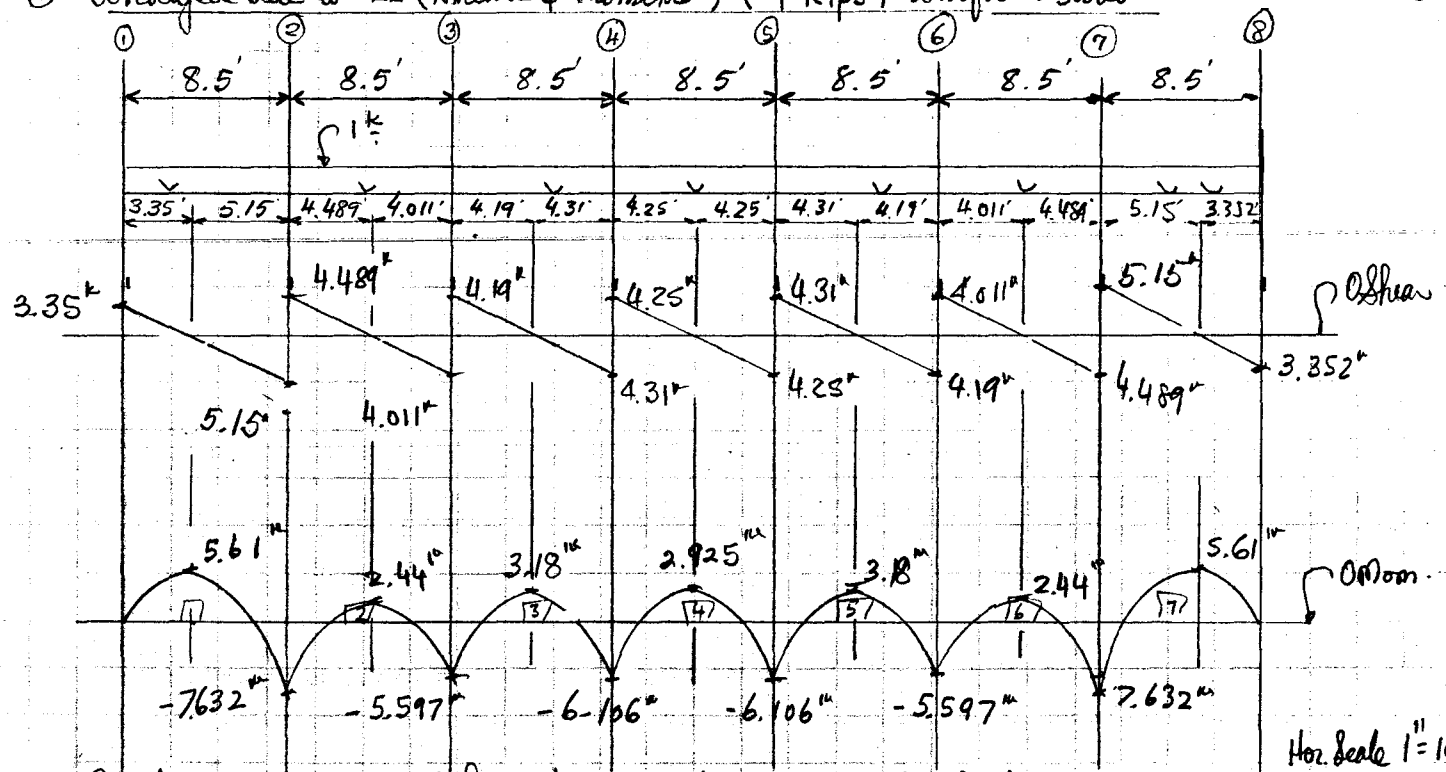
Checked by: N/A

Date: 10-26-89

Hor. Scale 1" = 10'
Vert. Scale 1 unit = 5^k
5^k

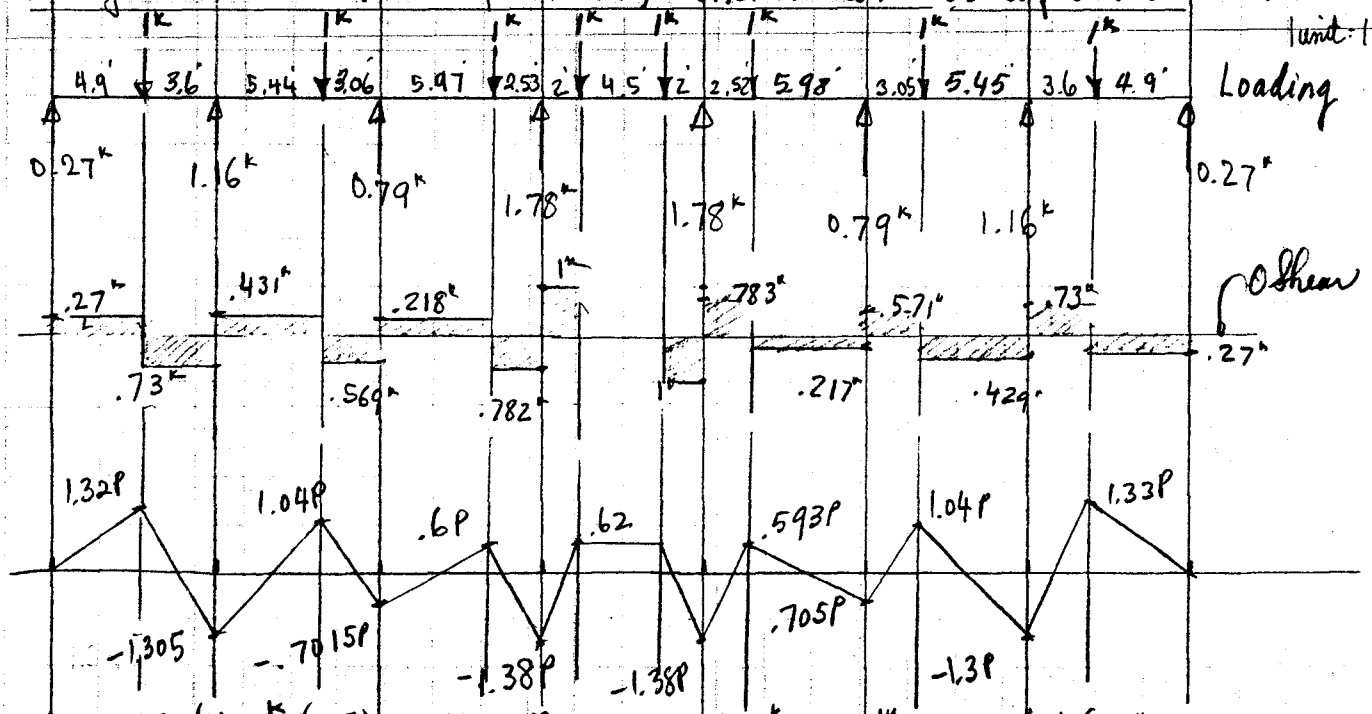
Abutment Cap Moments & Shears

① Analysis due to LL (shears & moments) (1 kips) uniform load



Hor. Scale 1" = 10
Vert. Scale 1 unit = 1 unit = 1"

② Analysis due to LL (shears & moments) concentrated loads loc. per Ali



Say $P = (16.2^k)(4.5) = 73 = P$, $M_{pos.} = 1.33(73^k) = 97.1^{k}$, $M_{neg} = 1.4(73^k) = 102.2^{k}$

Parsons Brinckerhoff **Computation Sheet**

Page ____ of ____ 68

Made by W VELLIOTT

Date 10-17-89

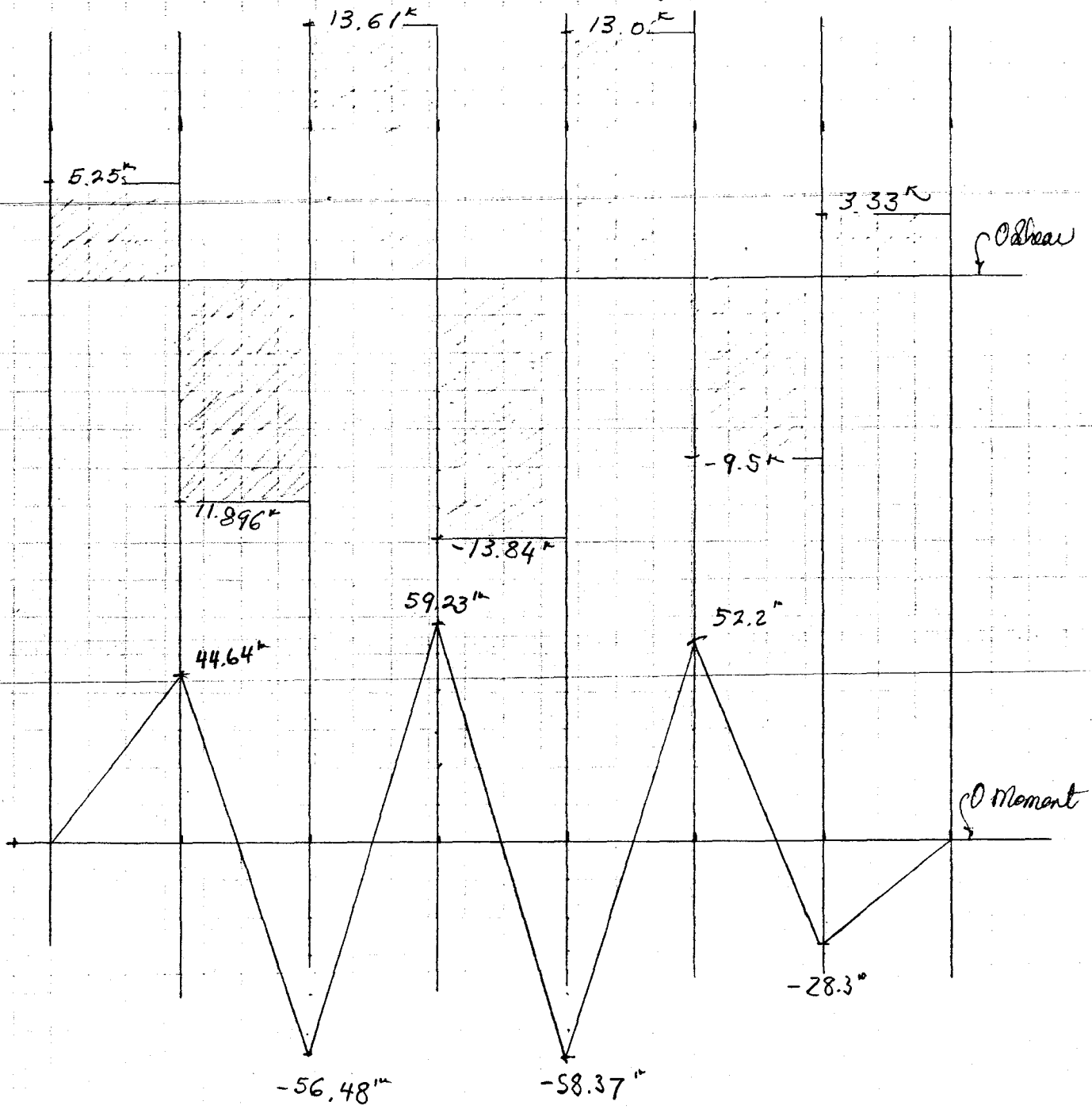
Subject 12th Street over ACDC

OUTPUT ABUT-ACDC

Checked by N/A

Date 10-26-89

Analysis of Pile Cap due to 1" Settlement, Loading C



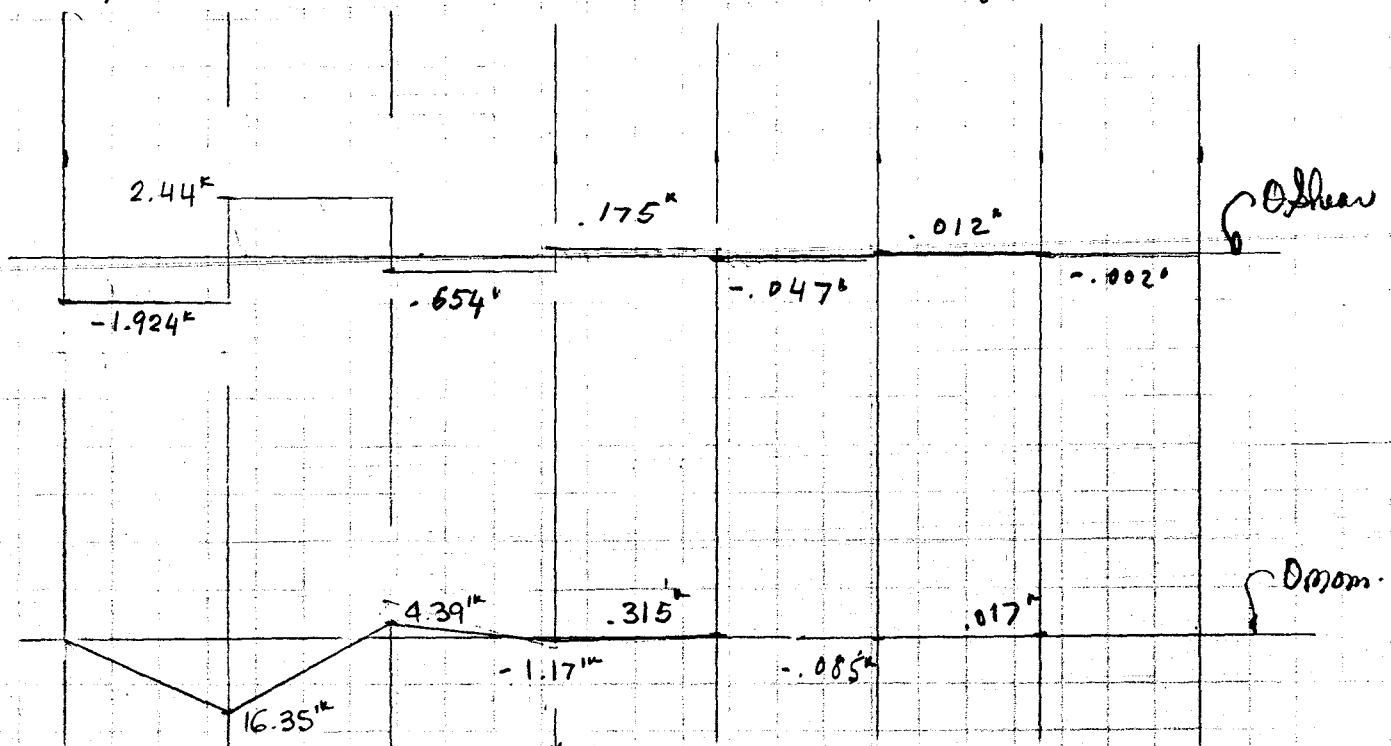
Scale: Hor. 1" = 10'
Vert. 1" = 40'k moment
1 unit = 2k shear

Parsons Brinckerhoff **Computation Sheet**

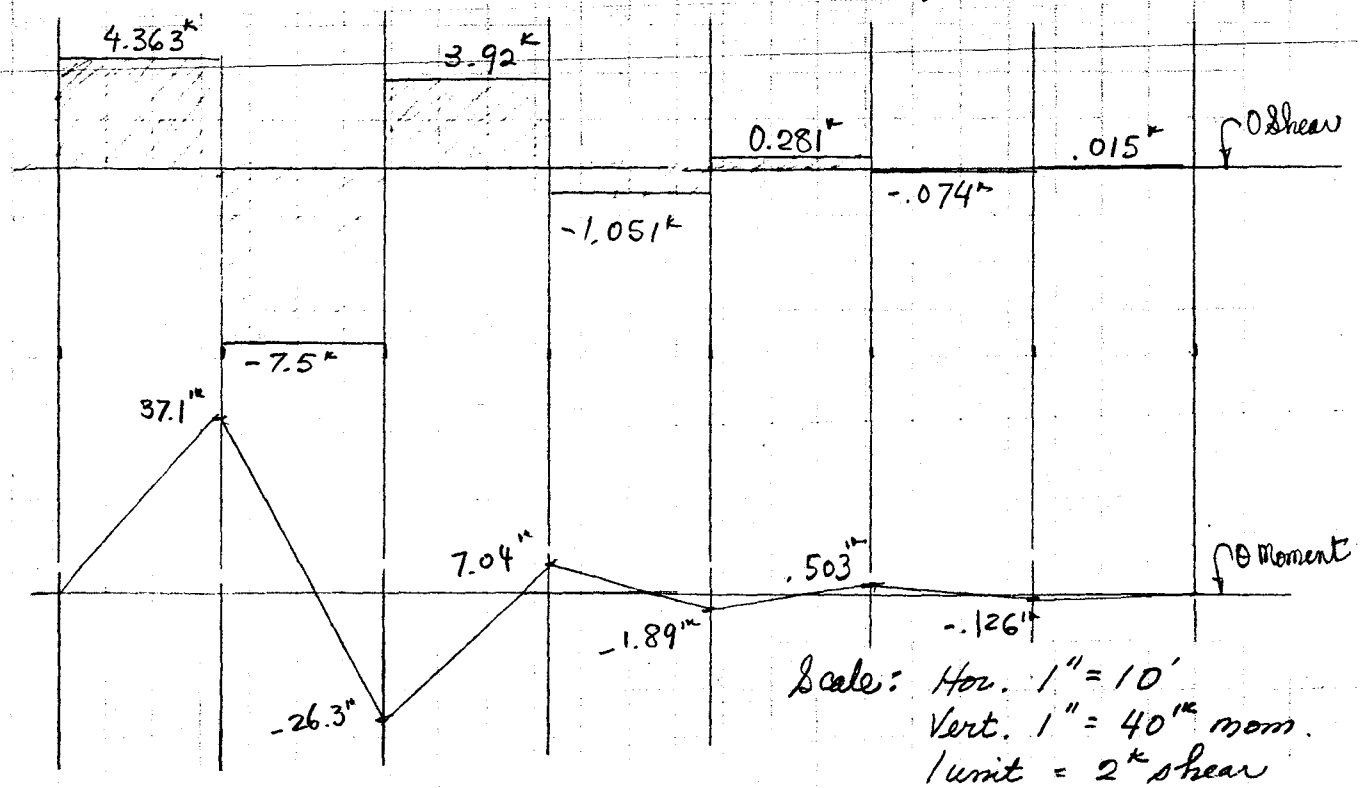
Subject 12th Street over ACDC
OUTPUT ABUT-ACDC

Page ____ of ____ 67
Made by WVELLIOTT
Date 10-16-89
Checked by N/A
Date 10-26-89

Analysis of Pile Cap due to 1" Settlement, Loading A



Analysis of Pile Cap due to 1" Settlement Loading B



Scale: Hor. 1" = 10'
Vert. 1" = 40'k mom.
1 unit = 2k shear

Note: This prog. was used to Recheck The final Design -

Job : ABUT-ACDC - ABUT. REACTIONS

Analysis DUE TO LL (negative moment) (1 KIPS)

System Parameters

Number of Nodes 8
Number of Elements 7
Number of Property Sets 1
Number of Load Cases 2

Node Coordinates

Node Number	X Coordinate(FT)	Y Coordinate(FT)
1	0.0000	0.0000
2	8.5000	0.0000
3	17.0000	0.0000
4	25.5000	0.0000
5	34.0000	0.0000
6	42.5000	0.0000
7	51.0000	0.0000
8	59.5000	0.0000

Node Constraints

Node Number	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	Free	0.0000	Free
2	Free	0.0000	Free
3	Free	0.0000	Free
4	Free	0.0000	Free
5	Free	0.0000	Free
6	Free	0.0000	Free
7	Free	0.0000	Free
8	0.0000	0.0000	Free

Property Set Data

Set Number	Area (FT^2)	Elastic Modulus (K/FT^2)	Inertia (FT^4)
1	9.0000	3320.0000	2.6670

* Note: Values of A, I, E .. Doesnot effect the results For LL & DL. - Ali -

Element Connectivity

Element	Origin Node	Terminal Node	Property Set
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	1
6	6	7	1
7	7	8	1

Uniform Member Loads

Element Number	Load Case	X Uniform Load (K/FT)	Y Uniform Load (K/FT)
1	2	0.0000	-1.0000
2	2	0.0000	-1.0000
3	2	0.0000	-1.0000
4	2	0.0000	-1.0000
5	2	0.0000	-1.0000
6	2	0.0000	-1.0000
7	2	0.0000	-1.0000

Concentrated Member Loads

Element Number	Load Case	Load Location (FT)	X Load (K)	Y Load (K)
1	1	4.9000	0.0000	-1.0000
2	1	5.4400	0.0000	-1.0000
3	1	5.9700	0.0000	-1.0000
4	1	2.0000	0.0000	-1.0000
4	1	6.5000	0.0000	-1.0000
5	1	2.5200	0.0000	-1.0000
6	1	3.0500	0.0000	-1.0000
7	1	3.6000	0.0000	-1.0000

Nodal Displacements

Node Number	Load Case	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	1	0.0000	0.0000	-0.0003
	2	0.0000	0.0000	-0.0017
2	1	0.0000	0.0000	0.0001
	2	0.0000	0.0000	0.0004
3	1	0.0000	0.0000	0.0001
	2	0.0000	0.0000	-0.0001
4	1	0.0000	0.0000	-0.0001
	2	0.0000	0.0000	0.0000
5	1	0.0000	0.0000	0.0001
	2	0.0000	0.0000	-0.0000
6	1	0.0000	0.0000	-0.0001
	2	0.0000	0.0000	0.0001
7	1	0.0000	0.0000	-0.0001
	2	0.0000	0.0000	-0.0004
8	1	0.0000	0.0000	0.0003
	2	0.0000	0.0000	0.0017

Nodal Reactions

Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
1	1	0.0000	0.2698	0.0000
	2	0.0000	3.3521	0.0000
2	1	0.0000	1.1608	0.0000
	2	0.0000	9.6373	0.0000
3	1	0.0000	0.7878	0.0000
	2	0.0000	8.2007	0.0000
4	1	0.0000	1.7818	0.0000
	2	0.0000	8.5599	0.0000

Nodal Reactions

Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
5	1	0.0000	1.7828	0.0000
	2	0.0000	8.5599	0.0000
6	1	0.0000	0.7876	0.0000
	2	0.0000	8.2007	0.0000
7	1	0.0000	1.1596	0.0000
	2	0.0000	9.6373	0.0000
8	1	0.0000	0.2699	0.0000
	2	0.0000	3.3521	0.0000

Element Forces

Element Number	Load Case	Node Number	Axial Force (K)	Shear Force (K)	Moment (FT- K)
1	1	1	0.000	0.270	-0.000
		2	0.000	0.730	-1.307
	2	1	0.000	3.352	0.000
		2	0.000	5.148	-7.632
2	1	2	0.000	0.431	1.307
		3	0.000	0.569	-0.707
	2	2	0.000	4.489	7.632
		3	0.000	4.011	-5.597
3	1	3	0.000	0.218	0.707
		4	0.000	0.782	-1.381
	2	3	0.000	4.190	5.597
		4	0.000	4.310	-6.106
4	1	4	0.000	1.000	1.381
		5	0.000	1.000	-1.380
	2	4	0.000	4.250	6.106
		5	0.000	4.250	-6.106
5	1	5	0.000	0.783	1.380
		6	0.000	0.217	-0.705
	2	5	0.000	4.310	6.106
		6	0.000	4.190	-5.597

Element Forces

Element Number	Load Case	Node Number	Axial Force (K)	Shear Force (K)	Moment (FT- K)
6	1	6	0.000	0.571	0.705
		7	0.000	0.429	-1.306
	2	6	0.000	4.011	5.597
		7	0.000	4.489	-7.632
7	1	7	0.000	0.730	1.306
		8	0.000	0.270	0.000
	2	7	0.000	5.148	7.632
		8	0.000	3.352	-0.000

ANALYSIS DUE TO 1" SETTELMENT

System Parameters

Number of Nodes	8
Number of Elements	7
Number of Property Sets	1
Number of Load Cases	1

Node Coordinates

Node Number	X Coordinate(FT)	Y Coordinate(FT)
1	0.0000	0.0000
2	8.5000	0.0000
3	17.0000	0.0000
4	25.5000	0.0000
5	34.0000	0.0000
6	42.5000	0.0000
7	51.0000	0.0000
8	59.5000	0.0000

Node Constraints

Node Number	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	Free	-0.0830	Free
2	Free	0.0000	Free
3	Free	0.0000	Free
4	Free	0.0000	Free
5	0.0000	0.0000	Free
6	Free	0.0000	Free
7	Free	0.0000	Free
8	0.0000	0.0000	Free

Property Set Data

Set Number	Area (FT^2)	Elastic Modulus (K/FT^2)	Inertia (FT^4)
1	9.0000	3320.0000	2.6670

Element Connectivity

Element	Origin Node	Terminal Node	Property Set
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	1
6	6	7	1
7	7	8	1

Nodal Displacements

Node Number	Load Case	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	1	0.0000	-0.0830	0.0124
2	1	0.0000	0.0000	0.0045
3	1	0.0000	0.0000	-0.0012
4	1	0.0000	0.0000	0.0003
5	1	0.0000	0.0000	-0.0001
6	1	0.0000	0.0000	0.0000
7	1	0.0000	0.0000	-0.0000
8	1	0.0000	0.0000	0.0000

Nodal Reactions

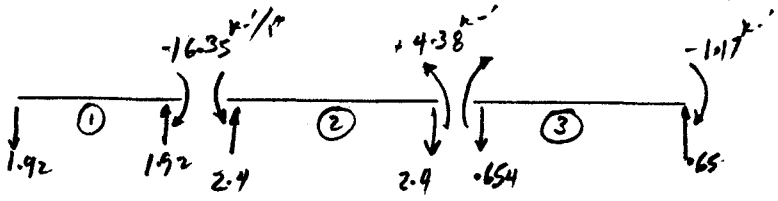
Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
1	1	0.0000	-1.9239	0.0000
2	1	0.0000	4.3633	0.0000
3	1	0.0000	-3.0931	0.0000
4	1	0.0000	0.8288	0.0000

Nodal Reactions

Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
5	1	0.0000	-0.2220	0.0000
6	1	0.0000	0.0592	0.0000
7	1	0.0000	-0.0148	0.0000
8	1	0.0000	0.0025	0.0000

Element Forces

Element Number	Load Case	Node Number	Axial Force (K)	Shear Force (K)	Moment (FT- K)
1	1	1	0.000	-1.924	0.000
		2	0.000	1.924	-16.353
2	1	2	0.000	2.439	16.353
		3	0.000	-2.439	4.382
3	1	3	0.000	-0.654	-4.382
		4	0.000	0.654	-1.174
4	1	4	0.000	0.175	1.174
		5	0.000	-0.175	0.314
5	1	5	0.000	-0.047	-0.314
		6	0.000	0.047	-0.084
6	1	6	0.000	0.012	0.084
		7	0.000	-0.012	0.021
7	1	7	0.000	-0.002	-0.021
		8	0.000	0.002	-0.000



$\bar{M}_{@2} = -16.353 \times 0.5' = 8.2^{k-'}$

System Parameters

Number of Nodes _____ 8
Number of Elements _____ 7
Number of Property Sets _____ 1
Number of Load Cases _____ 1

Node Coordinates

Node Number	X Coordinate(FT)	Y Coordinate(FT)
1	0.0000	0.0000
2	8.5000	0.0000
3	17.0000	0.0000
4	25.5000	0.0000
5	34.0000	0.0000
6	42.5000	0.0000
7	51.0000	0.0000
8	59.5000	0.0000

Node Constraints

Node Number	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	Free	0.0000	Free
2	Free	-0.0830	Free
3	Free	0.0000	Free
4	Free	0.0000	Free
5	Free	0.0000	Free
6	Free	0.0000	Free
7	Free	0.0000	Free
8	0.0000	0.0000	Free

Property Set Data

Set Number	Area (FT^2)	Elastic Modulus (K/FT^2)	Inertia (FT^4)
1	9.0000	3320.0000	2.6670

Element Connectivity

Element	Origin Node	Terminal Node	Property Set
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	1
6	6	7	1
7	7	8	1

Nodal Displacements

Node Number	Load Case	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	1	0.0000	0.0000	-0.0157
2	1	0.0000	-0.0830	0.0021
3	1	0.0000	0.0000	0.0073
4	1	0.0000	0.0000	-0.0020
5	1	0.0000	0.0000	0.0005
6	1	0.0000	0.0000	-0.0001
7	1	0.0000	0.0000	0.0000
8	1	0.0000	0.0000	-0.0000

Nodal Reactions

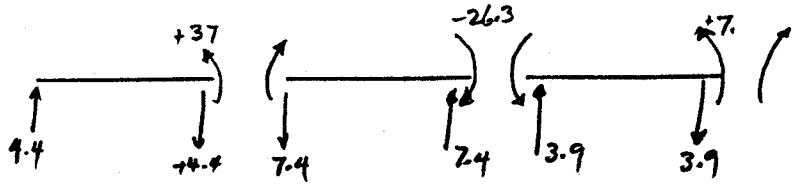
Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
1	1	0.0000	4.3633	0.0000
2	1	0.0000	-11.8197	0.0000
3	1	0.0000	11.3782	0.0000
4	1	0.0000	-4.9726	0.0000

Nodal Reactions

Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
5	1	0.0000	1.3319	0.0000
6	1	0.0000	-0.3552	0.0000
7	1	0.0000	0.0888	0.0000
8	1	0.0000	-0.0148	0.0000

Element Forces

Element Number	Load Case	Node Number	Axial Force (K)	Shear Force (K)	Moment (FT- K)
1	1	1	0.000	4.363	-0.000
		2	0.000	-4.363	37.088
2	1	2	0.000	-7.456	-37.088
		3	0.000	7.456	-26.291
3	1	3	0.000	3.922	26.291
		4	0.000	-3.922	7.044
4	1	4	0.000	-1.051	-7.044
		5	0.000	1.051	-1.887
5	1	5	0.000	0.281	1.887
		6	0.000	-0.281	0.503
6	1	6	0.000	-0.074	-0.503
		7	0.000	0.074	-0.126
7	1	7	0.000	0.015	0.126
		8	0.000	-0.015	-0.000



System Parameters

Number of Nodes	8
Number of Elements	7
Number of Property Sets	1
Number of Load Cases	1

Node Coordinates

Node Number	X Coordinate(FT)	Y Coordinate(FT)
1	0.0000	0.0000
2	8.5000	0.0000
3	17.0000	0.0000
4	25.5000	0.0000
5	34.0000	0.0000
6	42.5000	0.0000
7	51.0000	0.0000
8	59.5000	0.0000

Node Constraints

Node Number	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	Free	0.0000	Free
2	Free	-0.0830	Free
3	Free	0.0000	Free
4	Free	-0.0830	Free
5	Free	0.0000	Free
6	Free	-0.0830	Free
7	Free	0.0000	Free
8	0.0000	0.0000	Free

Property Set Data

Set Number	Area (FT^2)	Elastic Modulus (K/FT^2)	Inertia (FT^4)
1	9.0000	3320.0000	2.6670

Element Connectivity

Element	Origin Node	Terminal Node	Property Set
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	1
6	6	7	1
7	7	8	1

Nodal Displacements

Node Number	Load Case	X Translation (FT)	Y Translation (FT)	Rotation (Rad)
1	1	0.0000	0.0000	-0.0169
2	1	0.0000	-0.0830	0.0045
3	1	0.0000	0.0000	-0.0012
4	1	0.0000	-0.0830	0.0002
5	1	0.0000	0.0000	0.0006
6	1	0.0000	-0.0830	-0.0024
7	1	0.0000	0.0000	0.0091
8	1	0.0000	0.0000	-0.0045

Nodal Reactions

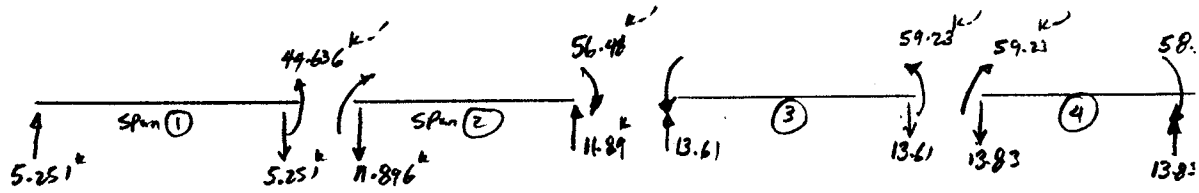
Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
1	1	0.0000	5.2513	0.0000
2	1	0.0000	-17.1475	0.0000
3	1	0.0000	25.5091	0.0000
4	1	0.0000	-27.4479	0.0000

Nodal Reactions

Node Number	Load Case	X Reaction (K)	Y Reaction (K)	Moment (FT- K)
5	1	0.0000	26.8411	0.0000
6	1	0.0000	-22.4753	0.0000
7	1	0.0000	12.7990	0.0000
8	1	0.0000	-3.3299	0.0000

Element Forces

Element Number	Load Case	Node Number	Axial Force (K)	Shear Force (K)	Moment (FT- K)
1	1	1	0.000	5.251	-0.000
		2	0.000	-5.251	44.636
2	1	2	0.000	-11.896	-44.636
		3	0.000	11.896	-56.482
3	1	3	0.000	13.613	56.482
		4	0.000	-13.613	59.228
4	1	4	0.000	-13.835	-59.228
		5	0.000	13.835	-58.369
5	1	5	0.000	13.006	58.369
		6	0.000	-13.006	52.184
6	1	6	0.000	-9.469	-52.184
		7	0.000	9.469	-28.304
7	1	7	0.000	3.330	28.304
		8	0.000	-3.330	0.000



$\overset{+}{M}_{@\,span\,1} = 5.251 \times 4.9' = 25.8\ k' \times 0.5' = 12.9\ k'$

DRILLED SHAFT &
WALL DESIGN

Parsons Brinckerhoff **Computation Sheet**

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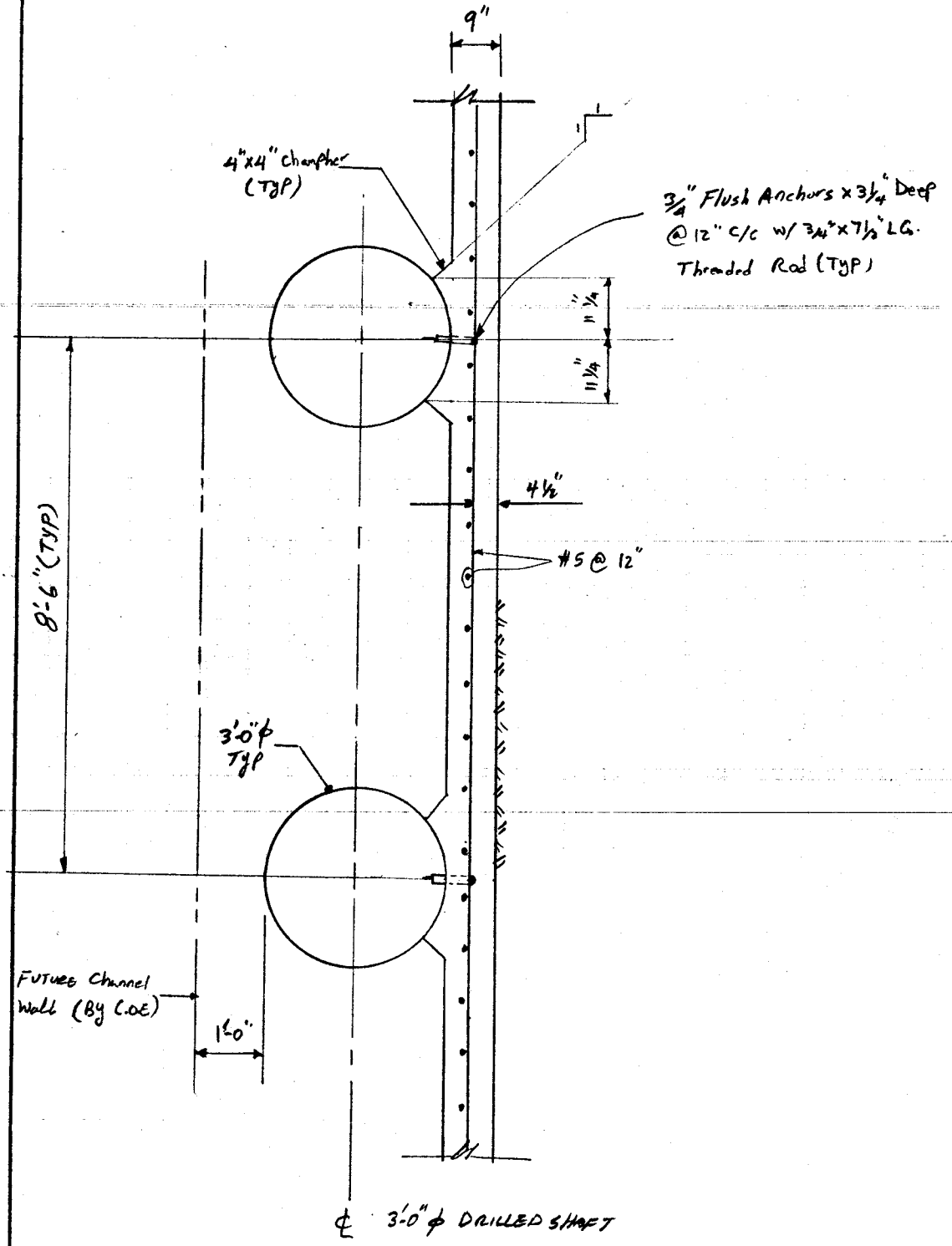
Made by Ali

Date 9-27-89

Checked by _____

Date _____

Subject 12th street OVER ACDC
WALL DETAIL



BACKWALL PLAN
Scale: 1" = 30"

USE 3/8" scale

Parsons Brinckerhoff **Computation Sheet**

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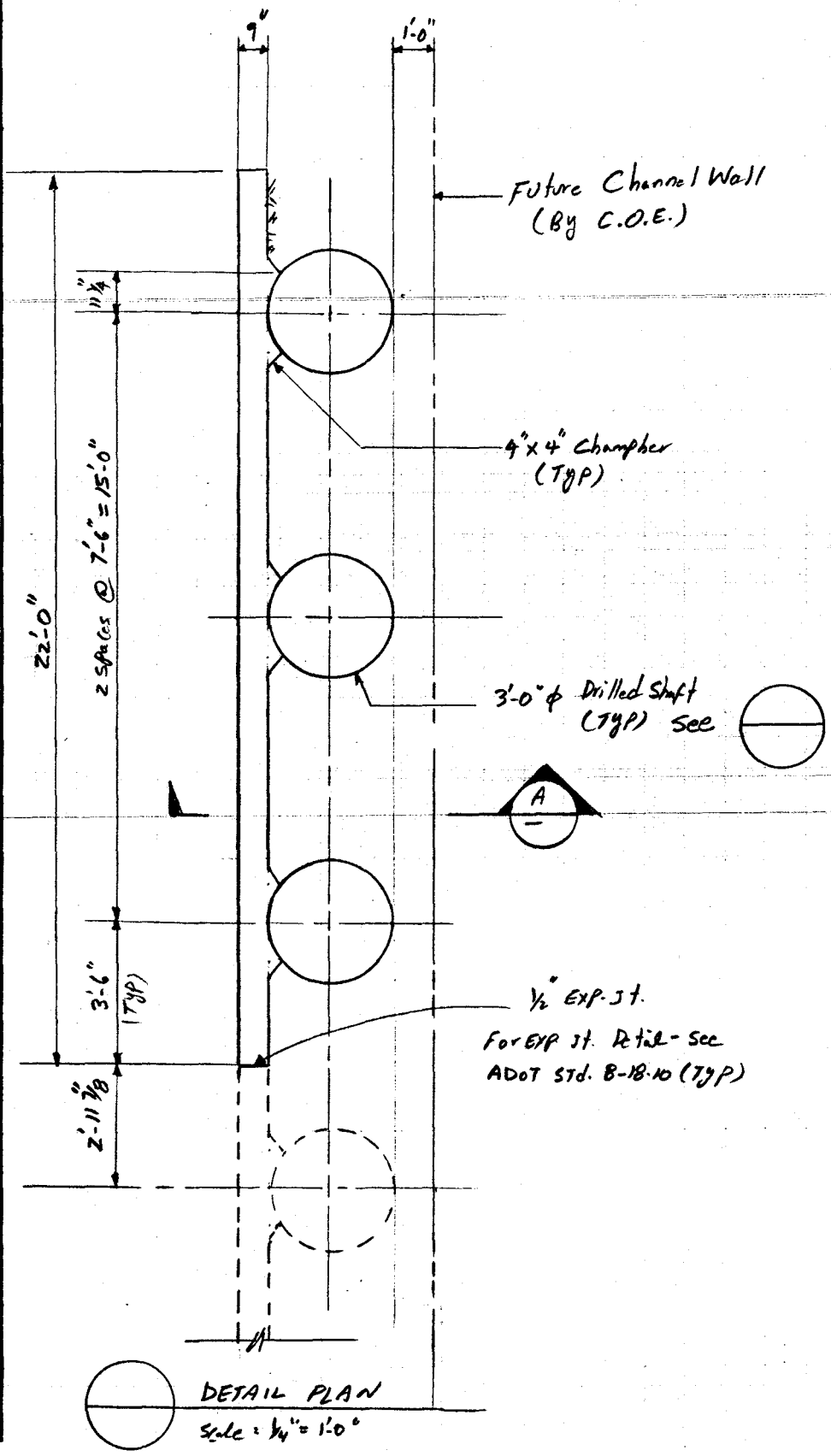
Made by Ali

Date 9-28-89

Checked by _____

Date _____

Subject 12th street over ACOC
Retaining Wall Detail



Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
Retaing Wall Section

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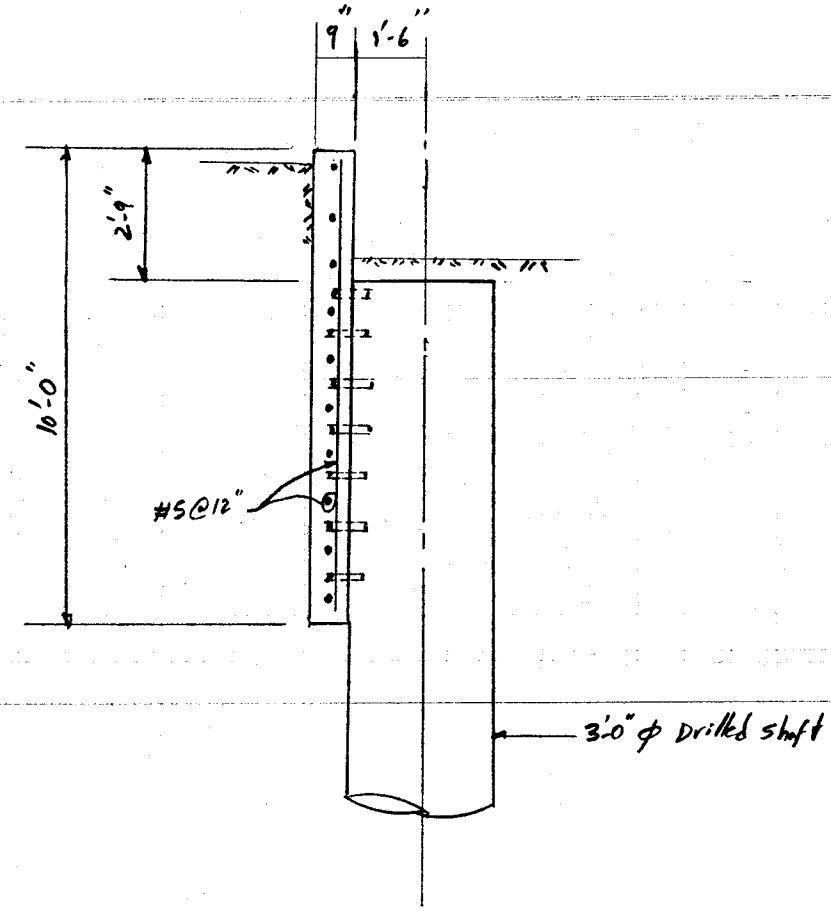
Made by Ali
Date 9-28-89
Checked by WUELLIOTT
Date 10-26-89



A
—
SECTION
Scale: 1/4" = 1'-0"

Parsons Brinckerhoff **Computation Sheet**

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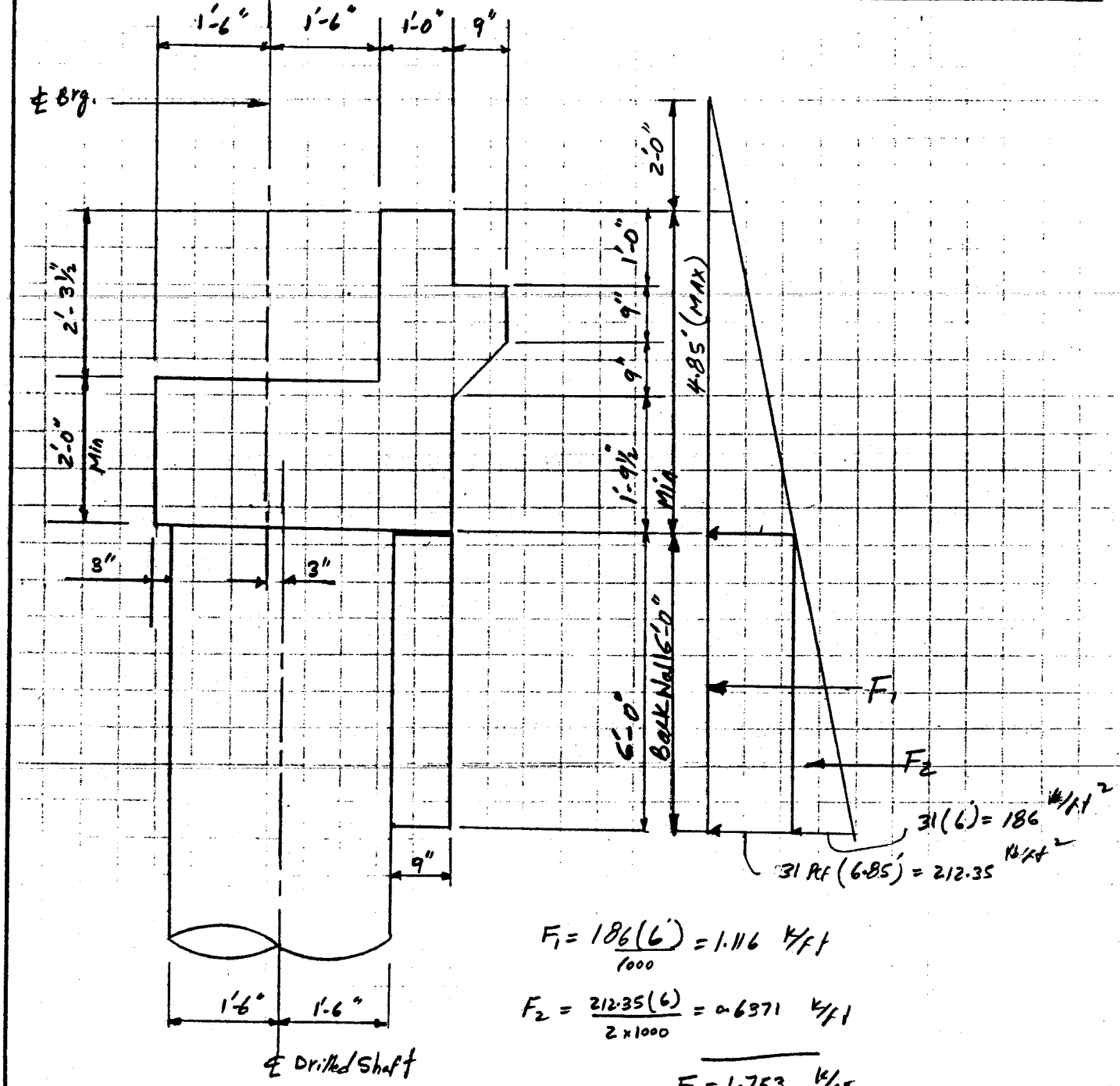
Date _____

Subject 12th street over ACDC

ABUTMENT SECTION (WALL DESIGN)



$$F_1 = \frac{186(6')}{1000} = 1.116 \ k/ft$$

$$F_2 = \frac{212.35(6)}{2 \times 1000} = 0.6371 \ k/ft$$

$$F = 1.753 \ k/ft$$

SECTION

Scale: 20" = 1'-0"

Subject 12th street over ACDC
WALL DESIGN (Back of Abut)

Made by Ali
Date 9-27-89
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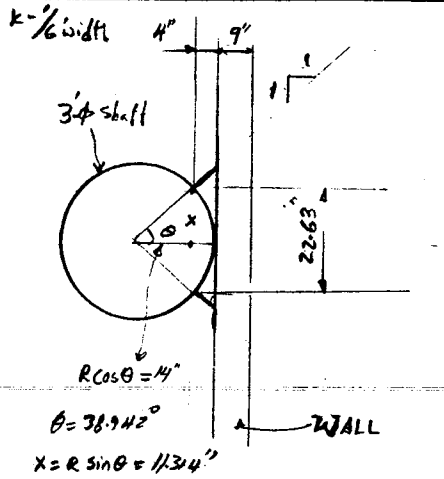
Date

$\bar{M}_{@\,6''\text{ from support span ②}} = -7.6323w - w\frac{(.94)^2}{2} + 4.489w(.94) = -3.85\,w = -3.85(1.7535) = -6.76$ k-'/6' width

$d = 4.5''$ FOR $\overset{+}{M}$; $d = 4.5'' + 4'' = 8.5''$ FOR $\bar{M}$



$\bar{A}_s = \frac{6.76 \times 12}{.87(20)(8.5)} = 0.55$ in²/6' = 0.09 in²/ft

$\overset{+}{A}_s = \frac{9.144 \times 12}{.87(20)(4.5)} = 1.4$ in²/6' = 0.23 in²/ft

$M_{cr} = 7.5\frac{\sqrt{3000}}{1000}\frac{(6\times12)(9)^3}{12\times12}\frac{1}{4.5} = 33.3$ k-'

$a = \frac{A_s f_y}{.85 f_c' b} = \frac{1.40(40)}{.85(3)(6\times12)} = 0.305$

$M_n = A_s f_y\left(d - \frac{a}{2}\right) = \frac{1.40(40)}{12}\left(4.5 - \frac{.305}{2}\right) = 20.3$ k-' $< 1.2\,M_{cr}$ USE 1/3 more steel

$A_s = 0.23$ in²/ft $\times 1.333 = 0.31$ in²/ft USE #5 @ 12" (TOTAL of 6 BAR)

Shear

$V = \bar{R}_2 = 5.148w = 5.148(1.7535) = 9.03$ k

$V_c = 0.95\frac{\sqrt{3000}}{1000}\,6\times12 \times 9 \times 0.8 = 27$ k $\quad V - V_c < 0$ USE #5 @ 12"

For combined plate action & shear

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street over ACDC
WALL DESIGN (Back of Abut.)

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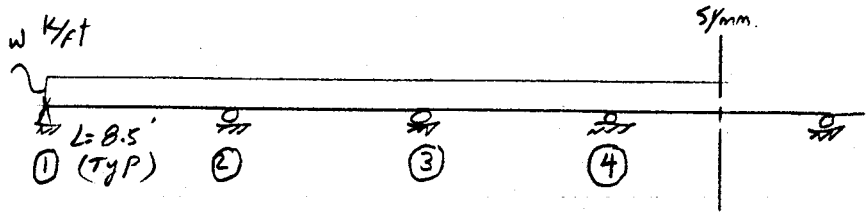
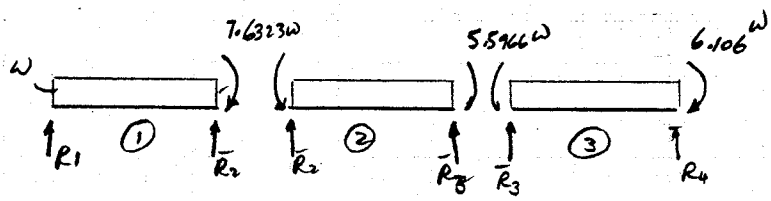


(2) $M_2\left(\frac{2}{3}L\right) + M_3\left(\frac{L}{6}\right) = -\frac{wL^3}{12} = -6.021\,w$ $\qquad M_2 = -7.6323\,w$ k-'

(3) $M_2\left(\frac{L}{6}\right) + M_3\left(\frac{2}{3}L\right) + M_4\left(\frac{L}{6}\right) = -\frac{wL^3}{12} = -6.021\,w$ $\qquad M_3 = -5.5966\,w$ k-'

(4) $+ M_3\left(\frac{L}{6}\right) + M_4\left(\frac{5L}{6}\right) = -\frac{wL^3}{12} = -6.021\,w$ $\qquad M_4 = -6.1059\,w$ k-'



$R_1 = \left(\frac{w(8.5)^2}{2} - 7.6323w\right)/8.5 = 3.352w$ $\qquad \bar{R}_2 = 8.5w - 3.352w = 5.148w$ $\qquad$ Span (1)

$\bar{R}_2 = \frac{(7.6323 - 5.5966)w + w\,8.5^2/2}{8.5} = 4.489w$ $\qquad \bar{R}_3 = 4.01\,w$ $\qquad$ Span (2)

$\bar{R}_3 = \frac{(5.5966 - 6.106)w + w\,8.5^2/2}{8.5} = 4.19\,w$ $\qquad \bar{R}_4 = 4.31\,w$ $\qquad$ Span (3)

Span (1) $\quad \overset{+}{M} = 3.352\,w\left(\frac{8.5}{2}\right) - w\frac{8.5^2}{8} = 5.215\,w$

Span (2) $\quad \overset{+}{M} = 4.489w\left(\frac{8.5}{2}\right) - 7.6323w - w\frac{8.5^2}{8} = 2.415\,w$

Span (3) $\quad \overset{+}{M} = 4.19w\left(\frac{8.5}{2}\right) - 5.5966w - w\frac{8.5^2}{8} = 3.18\,w$

Max Negative $\bar{M} = 7.6323(1.7535) = 13.38$ k-' $\qquad$ @ SUPPORT

Max Positive $\overset{+}{M} = 5.215(1.7535) = 9.144$ k-'/6' width $\qquad$ @ MID span

NOTE: USE MAX OF THE TWO MOMENTS for DESIGN & Calculate The negative Moment @ face of SUPPORT (@ The Fillet)

Parsons Brinckerhoff **Computation Sheet**

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Made by Ali

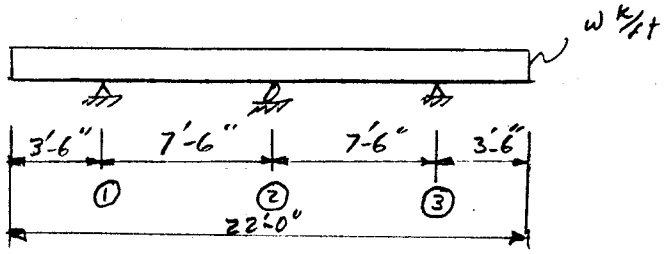
Date 9-27-89

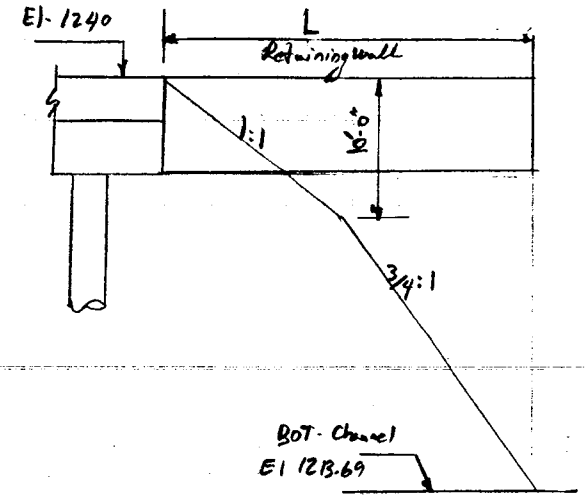
Checked by WV ELLIOTT

Date 10-26-89

Subject 12th street OVER ACDC

Retaining wall Design





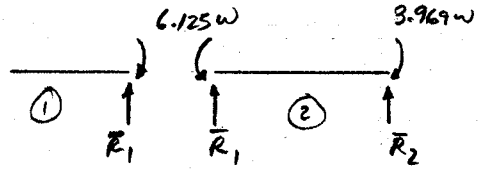
$h = El\ (1240 - 1213.69) = 26.3'$

$L = (26.3 - 10)\frac{3}{4} + 10' \approx 22'$

② $M_1 \frac{L}{6} + M_2\left(\frac{2}{3}L\right) + M_3 \frac{L}{6} = -\frac{wL^{3/2}}{12} = -4.6875\,w$

$\bar{M}_3 = \bar{M}_1 = \frac{w(3.5)^2}{2} = 6.125\,w$

$\bar{M}_2 = \frac{(-4.6875 + 1.021 + 1.021)\,w}{2/3} = -3.969\,w$

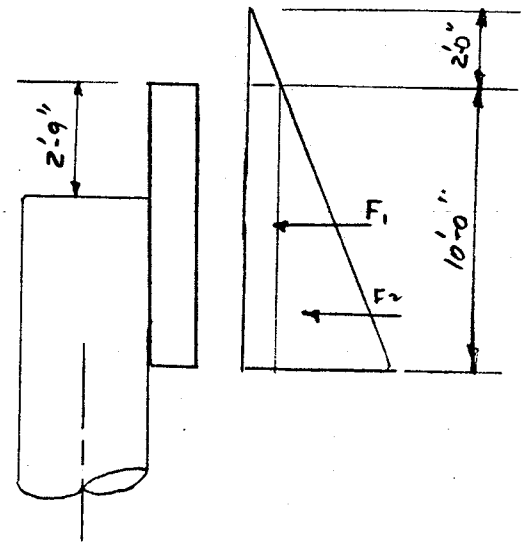


$\bar{R}_1 = 3.5\,w$ Span ①

$\bar{R}_1 = \frac{(6.125 - 3.969)\,w + w\frac{(7.5)^2}{2}}{7.5} = 4.04\,w$ Span ②

$\bar{R}_2 = (7.5 - 4.04)\,w = 3.46\,w$ Span ②

$\overset{+}{M}_{@\,mid\,span} = -6.125\,w + 4.04\,w\left(\frac{7.5}{2}\right) - w\frac{\left(\frac{7.5}{2}\right)^2}{2} = 1.99\,w$



$F_1 = 0.031\,kcf\,(2)(10') = 0.62$ k/ft

$F_2 = 0.031\frac{(10)^2}{2} = 1.55$ k/ft

$F_{TOTAL} = 1.55 + 0.62 = 2.17$ k/ft

Parsons Brinckerhoff **Computation Sheet**

Subject: 12th street over ACDC
Retaining wall Design

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Checked by: ____
Date: ____

Longitudinal Bending $\bar{M} = 6.125 w = 6.125(2.17) = 13.29$ k-'/10' width

$d = 4.5''$

$\bar{A}_s = \frac{13.29 \times 12}{.87(20)(4.5)} = 2.04$ in²/10' $= 0.2037$ in²/ft $M_n < 1.2 M_{cr}$ USE 1/3 more steel

$\bar{A}_s = 0.2037 \times 1.33 = 0.27$ in²/ft USE #5 @ 12"

TRANSV. Bending

Assume cantilever from Top of Shaft to ground level

$M = 0.031(2)\left(\frac{2.75}{2}\right)^2 + 0.031\left(\frac{2.75}{6}\right)^3 = 0.342$ k-'/ft

$A_s = \frac{0.342 \times 12}{.87(20)(5)} = 0.0472$ in²/ft USE #5@12" (combind with shear)

Parsons Brinckerhoff **Computation Sheet**

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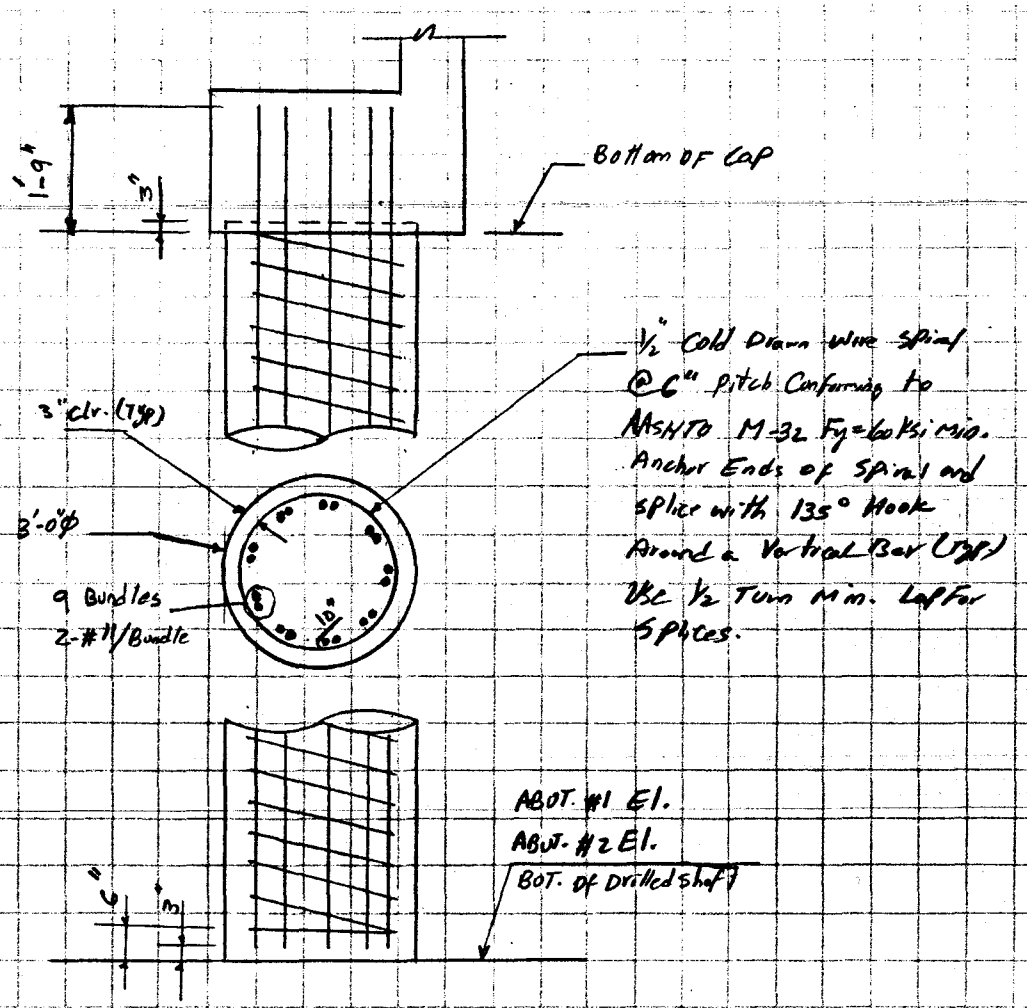
Made by Ali

Date 10-4-89

Checked by

Date

Subject 12th street OVER ACDC
ABUTMENT DRILLED Shaft



DRILLED SHAFT DETAIL

Scale:

Parsons Brinckerhoff **Computation Sheet**

Subject: 12th street OVER ACDC
Drilled Shaft Under Retaining Walls

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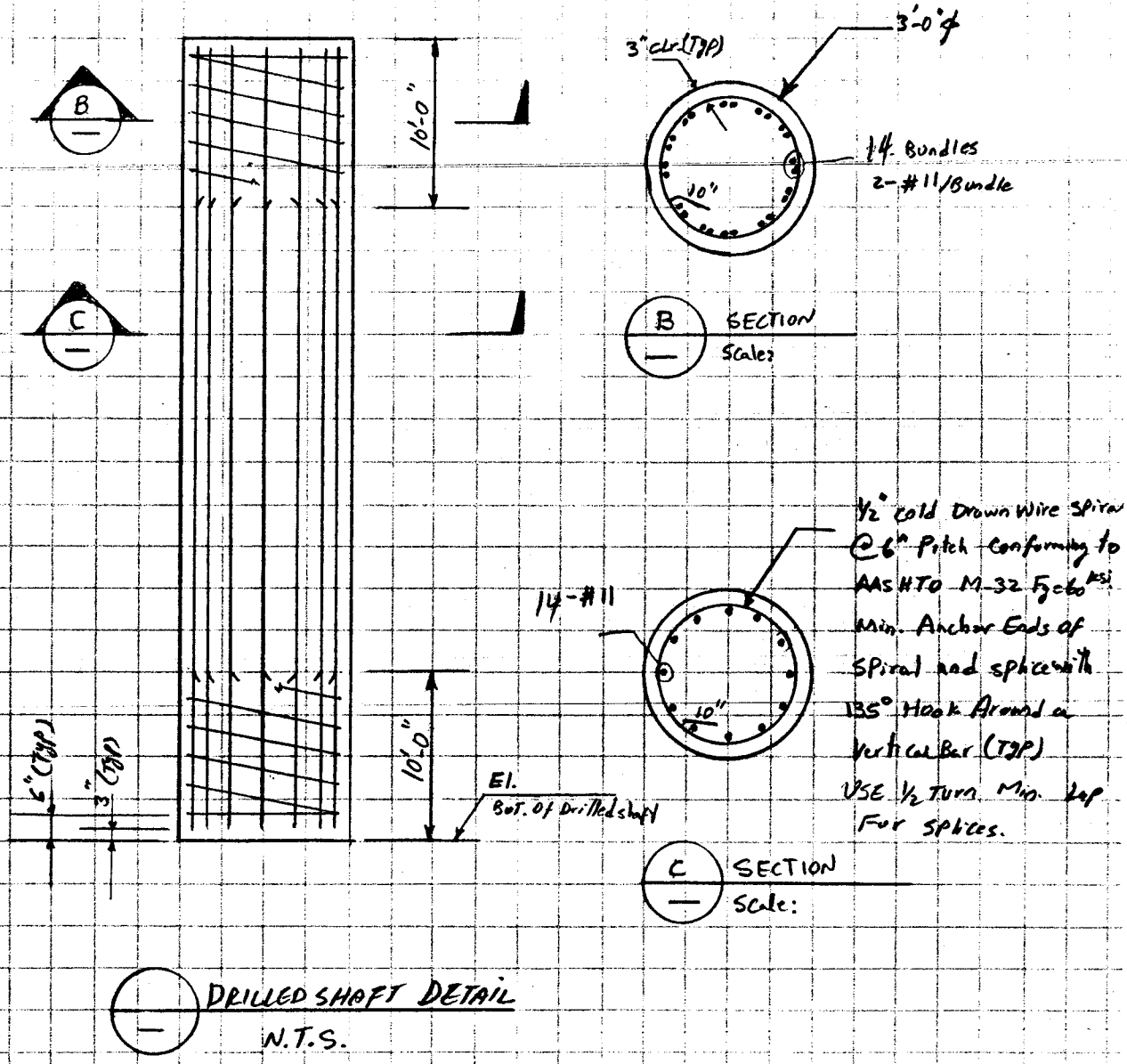
Made by: Ali
Date: 10-4-89
Checked by:
Date:



DRILLED SHAFT DETAIL
N.T.S.

Subject 12th street OVER ACDC
DRILLED SHAFT MOMENTS

Made by Ali
Date 9-26-89
Checked by WVELLIOTT
Date 10-26-89

Condition I - No Lateral Restraint

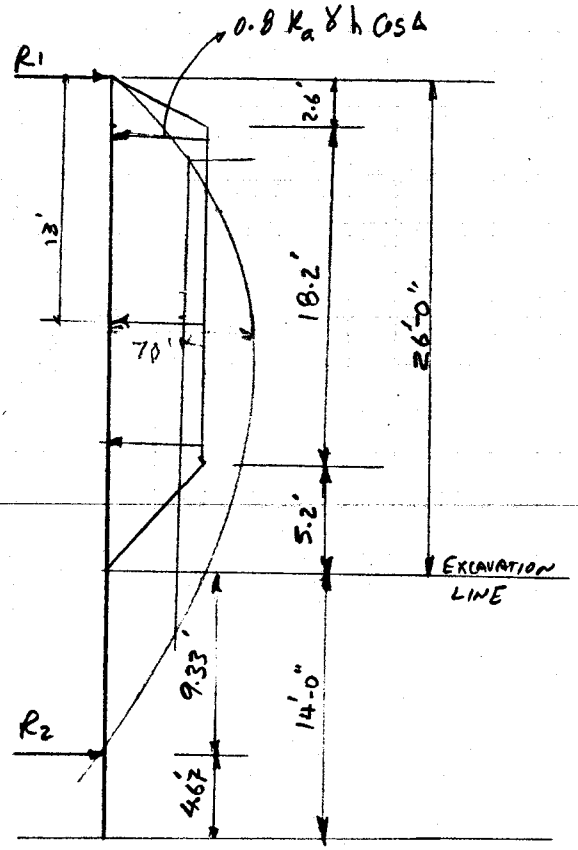
Moment = 91.395 k-ft/ft @ 1' below channel invert (El 1212.69)
and point of fixity @ 10.7' to 15.7' for 28' embedment depth.
This Type of Drilled shaft will be used for the Retaining walls.

$M = 91.4^{k-'} \times 7.5 = 686.^{k-'}$

*** The moment in the Drilled shaft has been modified per section $M = \frac{(30')^3}{6} \times 0.031 \times 5.5'_{spac} \approx 770$ k-'/shaft

Condition II - Lateral Restraint

Per Jim Rose 9/26/89 @ ATL LAB



$K_a = 0.27$

$\gamma = .115$ pcf

$\Delta = 2/3\,\phi = \frac{2}{3} \times 35^\circ = 23.33^\circ$

$h = 26'$

$F = 0.8 \times 0.27 \times .115 \times 26 \times \cos 23.33 = 0.594$ k/ft²

$$R_1 = \frac{0.594\left(\frac{2.6}{2}\right) \times 34.46' + 0.59(18.2)(23.63) + .59\left(\frac{5.2}{2}\right) \times 12.8}{35.33}$$

$R_1 = 8.55^{k/ft}$ $\quad R_2 = 13.1 - 8.55 = 4.58^{k/ft}$

$V = 0$ @ $8.55 = 2.6\left(\frac{.594}{2}\right) + .594(x)$ $\quad x = 13.1'$

$M_{max} = 8.55 \times 13.1 - 2.6\left(\frac{.59}{2}\right) \times 11.37 - .59\left(\frac{10.5}{2}\right)^2 = 70.8^{k-'/ft}$

Distance ℄ to ℄ shaft = 8'-6"

$M = 8.5 \times 70.8 = 602^{k-'}$/shaft

This doesnot include the moment Due to eccentricity of shaft.

Axial load/shaft = 106.2^k See Prem. Design Ali 8-29-89
eccent. = 3"

$M = 106.2 \times \frac{3}{12} + 602 = 629^{k-'}$/Drilled shaft

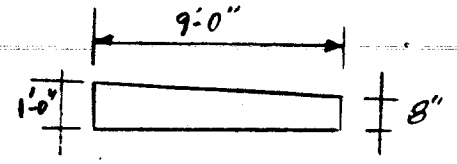
Parsons Brinckerhoff **Computation Sheet**

Subject 12TH ST. OVER ACDC
PRELIMINARY DESIGN

Page 1 of 2 | 94
Made by Ali
Date 8-29-89
Checked by
Date

$L = 50.23$ ROADWAY Width = 40' 2, 8'-0" Side walk & 2 Barrier

Typ. Section BI-48 wt. = 722 #/ft (14 - Box Beams)
Future Wearing Surface = 25 PSF (40) = 1 k/ft
2" ASPHALT CONCRETE wt. = $\frac{2}{12} \times 40 \times 0.15 = 1$ k/ft



wt. = $\frac{8+12"/12}{2} \times 9' \times 0.15 = 1.1$ k/ft side walk

Assume Barrier wt. of 0.4 k/ft (including Fence)

DIAPH. wt. = $3'1667 \times \frac{16"}{12} \times 14 \times (3')(.15)(3) = 80$ k/2 = 40k per ABUT.

DL = $\frac{722}{1000} \times 50.23 \times 14 + 50.23\,(2+2.2+0.8) = 759$ k/2 = 380k per Abut

8" STEEL Pipe = 28.81 #/ft $D_{inside} = 7.981$ in $A_i = 50.$ in^2
6" " " = 19.19 #/ft $D_i = 6.065$ in $A_i = 28.9$ in^2

Assuming BOTH Pipe is carrying water

wt. = $\frac{28.81+19.19}{1000} \times 50.23 + \frac{62.4\,\#/ft^3}{1000} \times \frac{(50+28.9)}{144} \times 50.23 = 4.13$ k/2 = 2.k/ABUT

TOTAL DL = 380 + 2 + 40 = 422k per ABUT.

ABUT width = 65.47 Assume 8 - 3'ϕ DRILLED SHAFT @ 8'-6"

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
DRILLED SHAFT DESIGN

Page ____ of ____ 95
Made by Ali
Date 10-4-89
Checked by WVELLIOTT
Date 10-26-89

NOTE 10-24-89 Ali

DRILLED SHAFT UNDER RETAINING WALLS (See calculation below for 5.5' spacing) For Latest moments

$M = 686.^{k-'}$ $M_{max} = \frac{686}{.35} = 1960.^{k-'}$

$P = \frac{9''}{12} \times 10' \times 7.5' \times 0.15 = 8.4^{k}$ $P_{max} = \frac{8.4}{.35} = 24.^{k}$

USE 12-Bundle of 2-#11
7.85" c to c of Bundles

DRILLED SHAFT UNDER ABUTMENT

$M = 629^{k-'}$ $M_{max} = \frac{629}{.35} = 1797.^{k-'}$

$P = 106.2^{k}$ $P_{max} = \frac{106.2}{.35} = 303.^{k}$

USE 9-Bundle of 2#11
10.47" c to c of Bundels

FOR DESIGN SEE Calc. By AL 10-4-89
ALSO see computer RUN D-SHAFT.OUT; # 4-OCT-1989 11:59 page 2

DRILLED Shaft UNDER Retaining wall (5'-6" spacing)

$M = 770^{k-'}$ $M_{max} = \frac{770}{.35} = 2200^{k-'}$

$P = \frac{9''}{12} \times 10' \times 5.5' \times 0.15 = 6.2^{k}$

$P_{max} = \frac{6.2}{.35} = 18^{k}$

USE 14-Bundle of 2-#11
spacing 6.3" c to c of Bnd 2

Also see computer Run D-shaft.out
25-OCT-1989

Parsons Brinckerhoff **Computation Sheet**

Page ____ of ____ 96
Made by Ali
Date 10-4-89
Checked by
Date

Subject 12th street over ACDC
DRILLED SHAFT DESIGN

Retaining wall Drilled shaft 7'6" spacing

$h = 36''$ cover 3" $\gamma = \frac{36-6-1}{36} = 0.8$

$f_c' = 4^{ksi}$ $f_y = 60^{ksi}$ 3'-0" ϕ shaft

$M_n = 1960^{k-'}$ $P_n = 24^k$

$\mu = \frac{f_y}{.85 f_c'} = \frac{60}{.85(4)} = 17.65$

$e = \frac{M}{P} = \frac{1960}{24} = 81.6'$

$\frac{e}{h} = \frac{81.6 \times 12}{36} = 27$ $\frac{P_n e}{f_c' h^3} = \frac{24 \times 81.6 \times 12}{4 \times 36^3} = 0.026$

$PM = 0.60$ From graph 7 Colum Interaction Diagram

$\rho = \frac{0.60}{17.65} = 0.034$ $A_s = 0.034 \times \pi (18)^2 = 34.6\ in^2$ 23 #11

ABUTMENT, DRILLED shaft

$M_n = 1797^{k-'}$ $P = 303^k$ $\gamma = 0.8$ $\mu = 17.65$

$e = \frac{M}{P} = \frac{1797}{303} \times 12 = 71.2''$

$\frac{e}{h} = \frac{71.2}{36} = 1.98$ $\frac{P_n e}{f_c' h^3} = \frac{303 \times 71.2}{4 \times 36^3} = 0.115$

$PM = 0.48$ From graph 7 Column Interaction Diagram

$\rho = \frac{0.48}{17.65} = 0.0272$ $A_s = 0.0272 \times \pi (18)^2 = 27.68\ in^2$ 18 #11

Retaing wall Drilled shaft 5'6" spacing $M = 2200^{k-'}$ $P = 18^k$

$\mu = 17.65$

$e = \frac{2200}{18} = 122.2'$

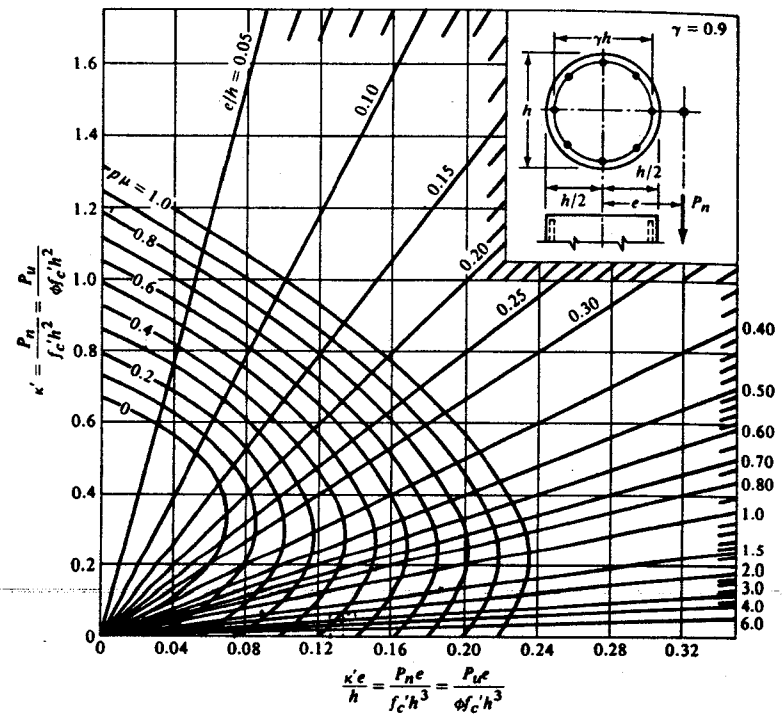
$\frac{P_n e}{f_c' h^3} = \frac{18 \times 122.2 \times 12}{4 \times 36^3} = 0.1414$

$PM = 0.7$ From graph 7

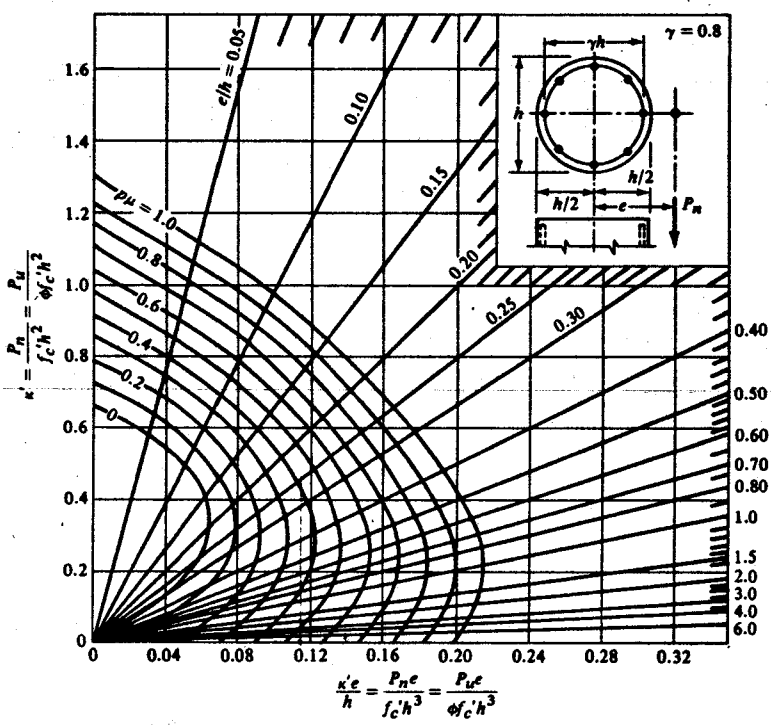
$\frac{e}{h} = \frac{122.2 \times 12}{36} = 40.7$

$\rho = \frac{.7}{17.65} = .0397$ $A_s = .0397 \times \pi\, 18^2 = 40.4\ in^2$

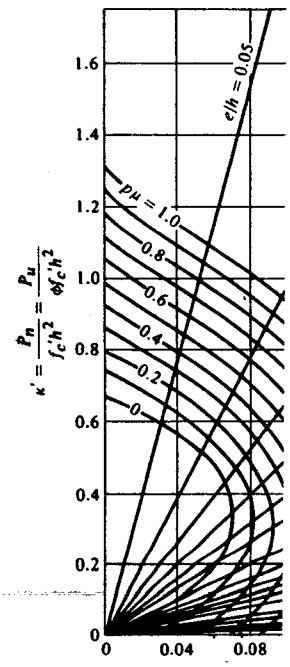
USE 28 #11



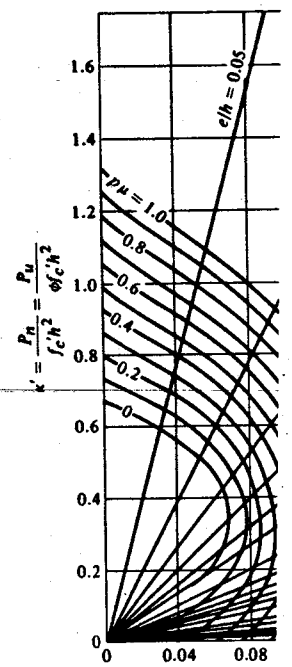
Graph 6 Column Interaction Diagram—Circular Section: $\gamma = 0.9$, $f_c' = 4$ ksi, $f_y = 60$ ksi



Graph 7 Column Interaction Diagram—Circular $\gamma = 0.8$, $f_c' = 4$ ksi, $f_y = 60$ ksi



Graph 8 Col
Section

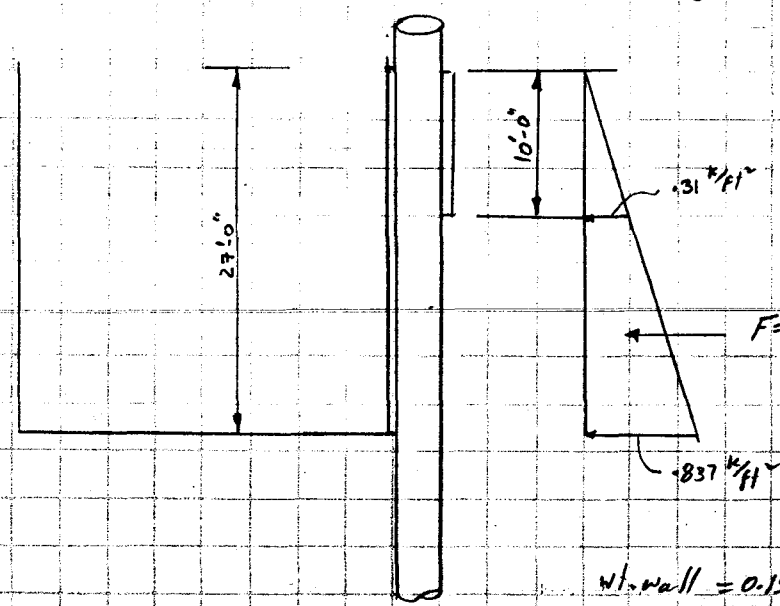


Graph 9 Co
Sectio

Subject 12th Street OVER ACDC
anchor inserts for wall & Drilled shaft

Made by Aki
Date 10-25-89
Checked by
Date

CASE I. NO Lateral Restraint. (5.5' spacing of shafts)



$F = .31(17') + (.837-.31)\left(\frac{17}{2}\right) = 9.75$ k/ft width

$F = 9.75/17 = 0.5735$ k/ft²

Wt. wall $= 0.15(1') = 0.15$ k/ft²

TOTAL $N = (0.15 + 0.5735)(5.5') = 3.98$ k/ft (spacing)

AASHTO 8.16.6.4.4 $V_n = A_{vf} F_y \mu$ $\mu = 0.7\lambda$ where $\lambda = 1.0$

$$A_{vf} = \frac{V_n}{F_y \mu} = \frac{3.98}{20 \times 0.7} = 0.28 \text{ in}^2$$

CASE II Lateral Restraint @ Top shaft (8.5' spacing of shafts)

USE Jim Ruse 9-26-89 SOIL PROFILE By ATL
Also see Drilled shaft Design by AQ

$F = 0.594$ k/ft²

$N = (0.594 + 0.15)(8.5) = 6.32$ k/ft

$$A_{vf} = \frac{6.32}{20 \times 0.7} = 0.45 \text{ in}^2$$

USE #6 @ 12" insert Bars

```
AAA    L      T
A   A  L      T
A   A  L      T
A   A  L      T
AAAAA  L      T
A   A  L      T
A   A  LLLLL  TTT
```

```
DDDDDDD        SSSSSSS  MM      MM    AAAAAA    TTTTTTTTTT  TTTTTTTTTT
DDDDDDDD       SSSSSSSS MM      MM    AAAAAA    TTTTTTTTTT  TTTTTTTTTT
DD     DD     SS       MM      MM   AA    AA   TT              TT
DD     DD     SS       MM      MM   AA    AA   TT              TT
DD     DD     SS       MM      MM   AA    AA   TT              TT
DD     DD     SS       MM      MM   AA    AA   TT              TT
DD     DD      SSSSSS  MMMMMMMMMM   AA    AA   TTTTTTTT        TT
DD     DD      SSSSSS  MMMMMMMMMM   AA    AA   TTTTTTTT        TT
DD     DD            SS MM      MM  AAAAAAAAAA  TT              TT
DD     DD            SS MM      MM  AAAAAAAAAA  TT              TT
DD     DD            SS MM      MM  AA    AA    TT              TT
DD     DD            SS MM      MM  AA    AA    TT              TT
DDDDDDDD       SSSSSSSS MM      MM  AA    AA    TT              TT
DDDDDDDD       SSSSSSSS MM      MM  AA    AA    TT              TT
```

```
DDDDDDD     AAAAAA   TTTTTTTTTT
DDDDDDDD    AAAAAA   TTTTTTTTTT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DD     DD  AAAAAAAAAA   TT
DD     DD  AAAAAAAAAA   TT
DD     DD  AA    AA     TT
DD     DD  AA    AA     TT
DDDDDDDD   AA    AA     TT
DDDDDDDD   AA    AA     TT
```

```
****    SSSSSSSSSS
****    SSSSSSSSSS
****    SS
****    SS
****    SSSSSS
        SSSSSS
****        SS
****        SS
****          SS
****          SS
  **    SS    SS
  **    SS    SS
**        SSSSSS
**        SSSSSS
```

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Digital Equipment Corporation - VAX/VMS Version V5.0-2

100

```
* [illegible]/2159   DRILLED SHAFT  12 ST. OVER ACDC 3.'DIAM. 3' COVER
*234567890123456789012345678901234567890123456789012345678901234567890123456
F F
36.0   30.0 4.0   60.   1.0 1.0
1    3.00  0011      5010
3 1
  1     24 1960    1
**
* N109/2159   DRILLED SHAFT  12 ST. OVER ACDC 3.'DIAM. 3' COVER
*234567890123456789012345678901234567890123456789012345678901234567890123456
D R
36.0   30.0 4.0   60.   1.0 1.0
1    3.00  0011      5010
3 1
  1    302 1797    1
```

101

V.S.L.A. STRENGTH DESIGN OF R.C. COMPRESSION MEMBERS

A 8109/2159 DRILLED SHAFT 12 FT. OVER ACDC 3.'DIAM. 3" COVER

*234567890123456789012345678901234567890123456789012345678901234567890123456789012345

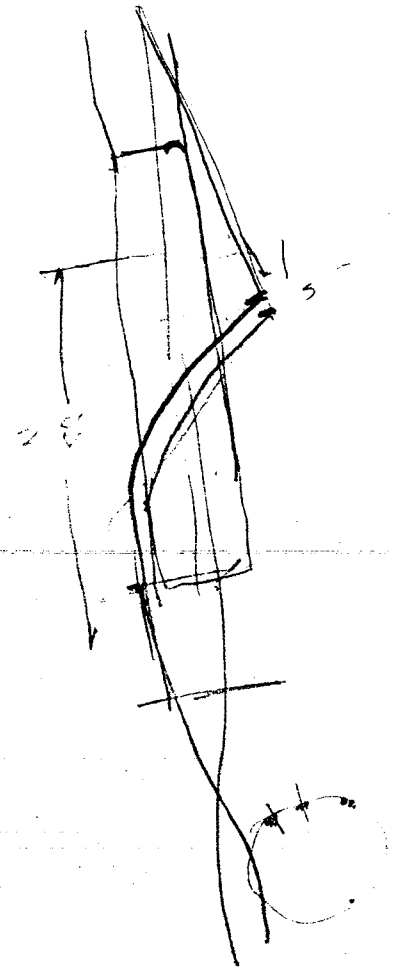
DESIGN OF
ROUND COMPRESSION MEMBERS

B= 36.00 T= 36.00 FC= 4.000 FY= 60.000 PHIC= 1.000 PHIP= 1.000

USE- 22 NO.11 BARS. AST = 34.32 SQ.IN. = 3.38 PCT. COVER = 3.000 IN.
18 # 12 40.50

LOAD CASE	APPLIED LOADS AP	AMX	AMY	COMPUTED STRENGTH UP	UMX	UMY	UP/AP
1	34.	1960.	1.	74.	1961.	1.	1.001

18 = 2200



A 8109/2159 DRILLED SHAFT 12 FT. OVER ACDC 3.'DIAM. 3" COVER

*234567890123456789012345678901234567890123456789012345678901234567890123456789012345

DESIGN OF
ROUND COMPRESSION MEMBERS

B= 36.00 T= 36.00 FC= 4.000 FY= 60.000 PHIC= 1.000 PHIP= 1.000

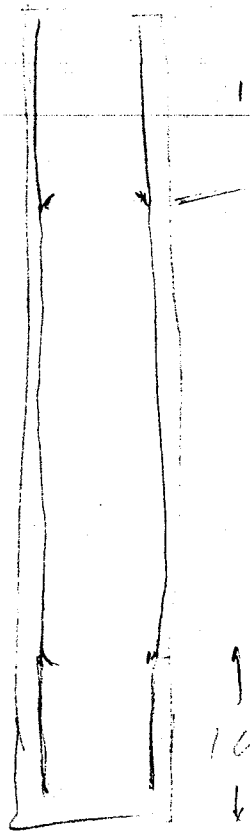
USE- 18 NO.11 BARS. AST = 28.08 SQ.IN. = 2.76 PCT. COVER = 3.000 IN.

LOAD CASE	APPLIED LOADS AP	AMX	AMY	COMPUTED STRENGTH UP	UMX	UMY	UP/AP
1	303.	1797.	1.	306.	1818.	1.	1.012

$\frac{770}{.33} = 2200$

$P = \frac{9''}{12} \times 10' \times 5.5 \times 0.15 = 6.2$

$P = 18^{1/}$



20 x 26 x 5.3
= 2600 LB
1.40
1 KIP.

P.C.A. - STRENGTH DESIGN OF R.C. COMPRESSION MEMBERS

A 0109/2159 DRILLED SHAFT 12 ST. OVER ACDC 2.7DIAM. 3" COVER

123456789012345678901234567890123456789012345678901234567890123456789012345

DESIGN OF
ROUND COMPRESSION MEMBERS

B= 36.00 T= 36.00 FC= 4.000 FY= 60.000 PHIC= 1.000 PHIB= 1.000

USE 18 NO.12 BARS. AST = 40.50 SQ.IN. = 3.98 PCT. COVER = 3.000 IN.

LOAD CASE	APPLIED LOADS AP	ANX	ANY	COMPUTED STRENGTH UP	UNX	UNY	UP/AP
1	18.	2200.	1.	18.	2225.	1.	1.011

A 0109/2159 DRILLED SHAFT 12 ST. OVER ACDC 2.7DIAM. 3" COVER

123456789012345678901234567890123456789012345678901234567890123456789012345

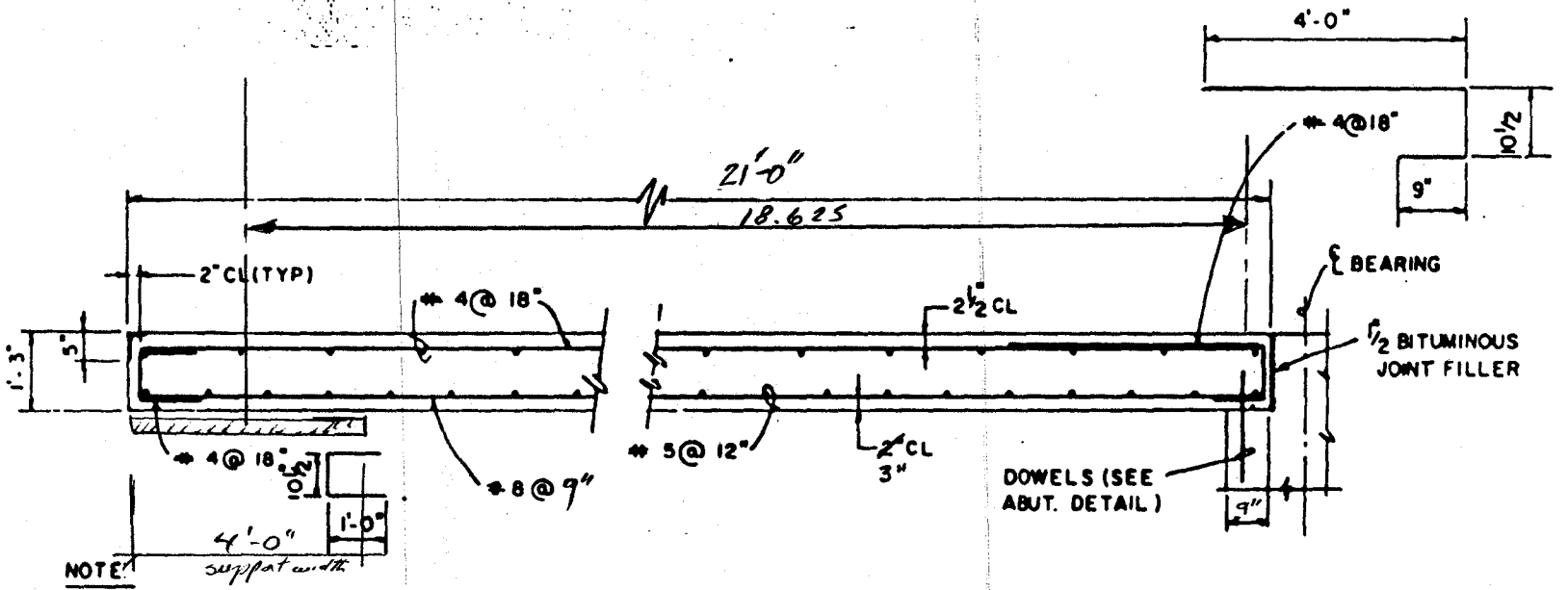
DESIGN OF
ROUND COMPRESSION MEMBERS

B= 36.00 T= 36.00 FC= 4.000 FY= 60.000 PHIC= 1.000 PHIB= 1.000

USE 18 NO.11 BARS. AST = 28.08 SQ.IN. = 3.76 PCT. COVER = 3.000 IN.

LOAD CASE	APPLIED LOADS AP	ANX	ANY	COMPUTED STRENGTH UP	UNX	UNY	UP/AP
1	303.	1797.	1.	308.	1818.	1.	1.012

Approach Slab
Sidewalks &
Barrier Design



NOTE:
REFER TO ADOT B19.10 FOR DETAIL NOT SHOWN.

SECTION APPROCH SLAB ~~(NORTH ONLY)~~

SCALE ½" = 1'-0"

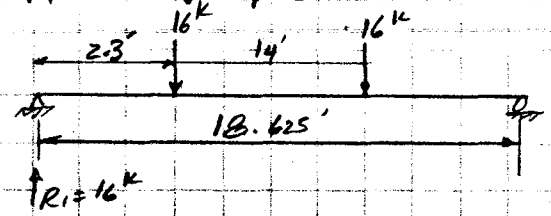
104

Approach Slab Design (TABLE 8.9.2 AASHTO)

Min Thickness $= 1.2\left(\frac{S+10}{30}\right) = 1.2\left(\frac{166+10}{30}\right) = 1.15'$ USE 1'-3" AASHTO 8.9.3.2.

Assume 2" Asphalt on the approach slab & 25 psf wearing surface

NOTE:
As per instruction of County the approach slab will be designed as simple span. USE 4' support @ Roadway and ℄ of seat



Distribution $S = (4 + .06S) = 5.1175'$

$M_{LL} = 16(9.312) - 16(7.0) = 37^{k\text{-}'} * 1.3 = 48.1^{k\text{-}'} / 5.1175$ wide lane $= 9.399^{k\text{-}'/ft}$

$M_{DL} = (1.42 + 0.15 + .025)\frac{18.625^2}{8} = 10.32^{k\text{-}'}/ft$

$M = 13.1 + 7.3 = 19.72$ k-'/ft $d = 15" - 3" - 0.5 = 11.5$

$f'_c = 4000$ psi $E_c = 33(150)^{1.5}\frac{\sqrt{4000}}{1000} = 3834$ ksi $f_c = 0.4 f'_c = 1600$ psi $f_s = 24$ ksi

$n = \frac{29000}{3834} = 7.5$ $k = \frac{f_c}{f_c + f_s/n} = \frac{1.6}{1.6 + \frac{24}{7.5}} = 0.333$ $j = 1 - \frac{k}{3} = 0.8889$

$A_s = \frac{19.72 * 12}{.889(24)(11.5)} = 0.94$ in²/ft USE #8@9" SAME AS ADOT STD.

Temperature & Shrinkage AASHTO 8.20.1

1/8 in²/ft USE #4 @ 18" = 0.13 in²/ft

Trasv. Reinforcement AASHTO 3.24.10.2

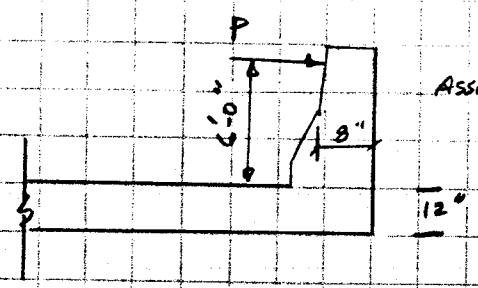
$\% = \frac{100}{\sqrt{18.62}} = 23.2\%$ $A_s = 0.23 * 0.94 = 0.22$ in²/ft USE #5@12"

Subject 12th street over ACDC
Sidewalk on the Bridge (Design)

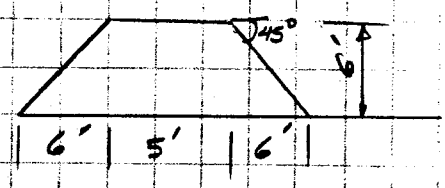
Side walk on the Bridge

AASHTO 8.2.1 Shrinkage & Temp.

1/8 in²/ft USE #4 @ 18"



Assume 2k/ft for 5'-0" Impact @ 6' above Deck.
Load Dist. Pattern of

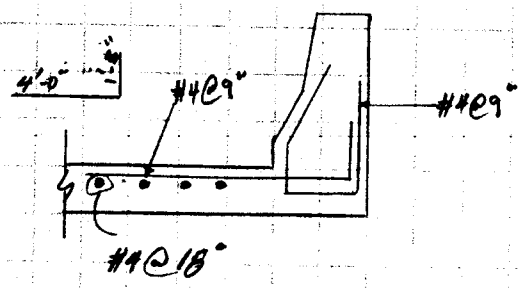


Impact on Barrier of sidewalk

$P = 10/17 = 0.588$ k/ft $\qquad M = 0.588(6') = 3.53$ k'/ft

$d = 12" - 2 - .5 = 9.5"$

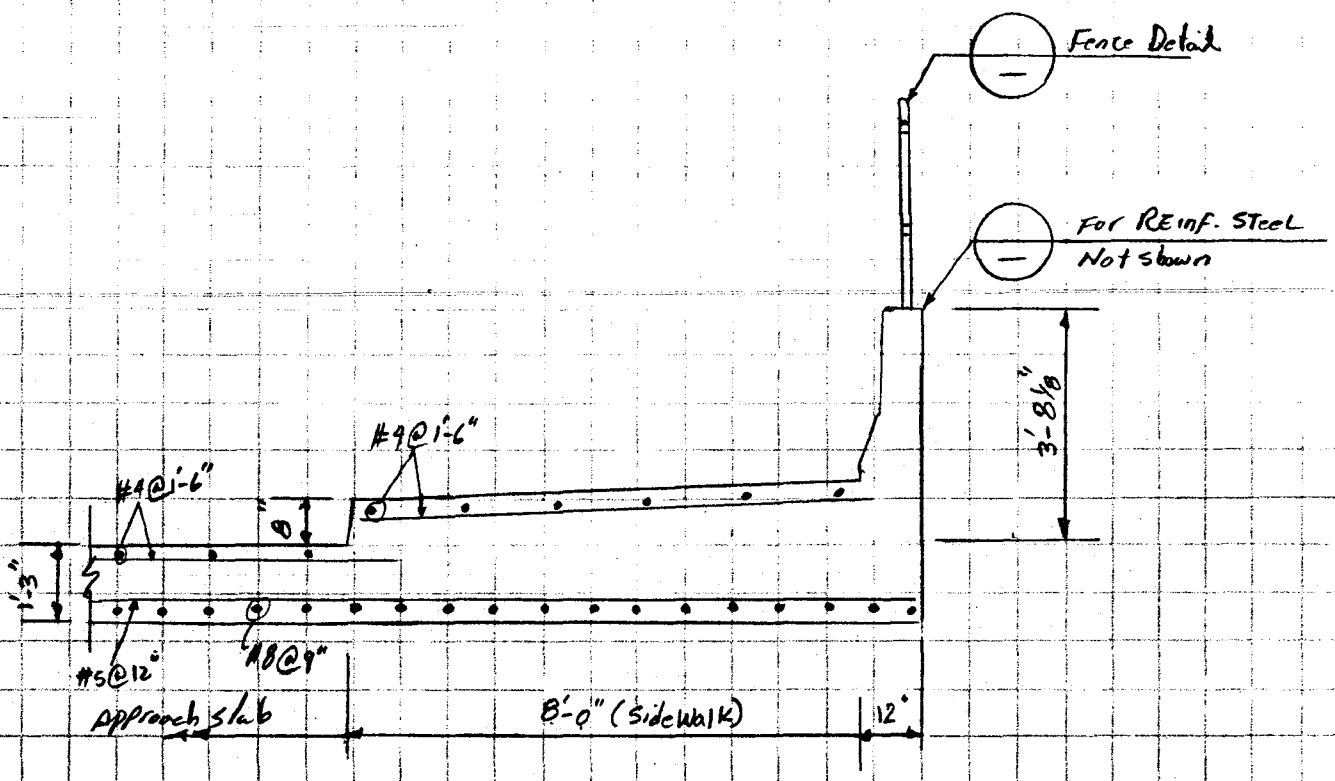
$A_s = \dfrac{3.53 \times 12}{.875(20)(9.5)} = 0.26$ in²/ft $\qquad$ USE #4 @ 9" ✓



Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
Sidewalk @ approach slab

Page ____ of ____ 108
Made by ALi
Date 10-3-89
Checked by
Date



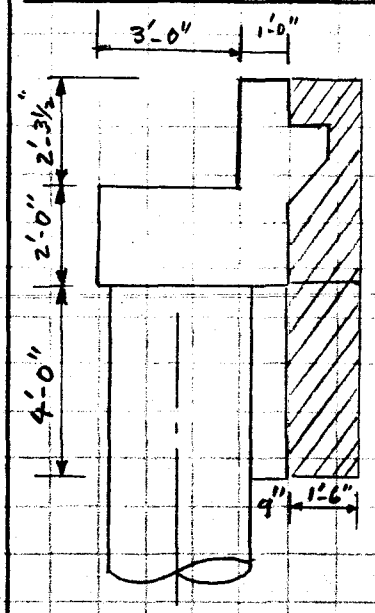
Sidewalk @ Approach Slab
Scale:

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street over ACDC
BRIDGE QUANTITY

Page 1 of 7 | 110
Made by ALi
Date 8-17-89
Checked by DJ Church
Date 10/24/89

(F) STRUCTURAL EXCAVATION



ABUT EXC. VOL. = $\frac{7}{27}(4.3')\left(\frac{56}{\cos 27.626} + 3'\right) * 2$ ABUT $= \underline{148 \text{ cy.}}$

WALL EXC. VOL. = $\frac{(3' + 0.75)}{27}(4')\left(\frac{56}{\cos 27.625} + 19 + 19 + 3'\right) * 2 = \underline{116 \text{ cy}}$

STRUCTURAL EXC. = 148 + 116 = 264 cy.

BOX EXCAVATION = $(4.29')\left(\frac{43.5}{27}\right)(58 + 3) = 422$ cy

TOTAL = 264 + 422 = 686 cy

(G) SPECIAL BACKFILL

Vol. Concrete ABUT & Wall = 66.2 c.y. (See Concrete QUANT.)

BACKFILL = 148 + 116 − 66.2 = 198. cy

(H) MISCELLANEOUS STEEL

NONE PLANNED

(I) BOX BEAM (BI-48)

L = 50.23' + 2.8 = 53'

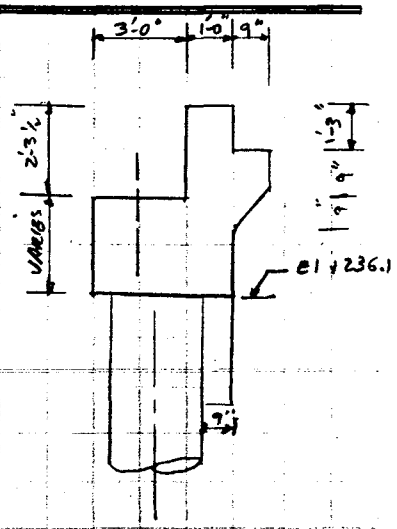
PRICE = $80/L.F.

TOTAL PRICE = 53 * 80 = $4240.

Parsons Brinckerhoff Computation Sheet

Subject 12th street over ACDC
Concrete Quantity

Page 2 of 7 | 111
Made by Ali
Date 10-23-89
Checked by DJ Church
Date 10/25/89



ABUT #1 Seat El. 1239.50 @ ℄ 12st.
Seat El. 1238.78 @ Exter. corner Right
El. 1239.13 " " " Left

① VARIABLE DIST @ LEFT = $-1236.19 + \frac{1239.13 + 1239.50}{2} = 3.125'$

② VARIABLE DIST @ Right = $-1236.19 + \frac{1239.50 + 1238.78}{2} = 2.95$

$A_1 = 1'(2.292') + .75(.75) + .75\left(\frac{.75}{2}\right) + 3.125(4') = 15.636 \ ft^2$

$A_2 = 1(2.292) + .75(.75) + .75\left(\frac{.75}{2}\right) + 2.95(4') = 14.94 \ ft^2$

Vol. $= (A_1 + A_2)(32'-9") = \frac{(15.636 + 14.94)(32.75)}{27} = \underline{37.09 \ cy}$ ABUT #1

Vol. Side walls $= 2\left(\frac{2'-3\frac{1}{2}"}{}\right)\left(\frac{1'-0\frac{3}{8}"}{27}\right)(4') = \underline{.7 \ cy}$ per ABUT.

Vol. Backwall $= 6'\left(\frac{9"}{12}\right)\left(\frac{65.5'}{27}\right) = \underline{10.92 \ cy}$ Per ABUT

Vol. Retaining wall $= 10'\left(\frac{9"}{12}\right)\left(\frac{22'}{27}\right) = \underline{6.11 \ cy}$ 1 Retaining wall TOTAL OF 4

ABUT #2 seat El. 1238.75 @ ℄ 13th st.
seat El. 1238.74 @ Exter. corner Right
(DJC) seat El. 1238.19 @ " " Left

① Variable DIST. @ Left $= -1236.19 + \frac{1238.74 + 1238.75}{2} = 2.55'$

② " " @ Right $= -1236.19 + \frac{1238.74 + 1237.94}{2} = 2.28$

$A_1 = 1(2.292) + .75(.75) + .75\left(\frac{.75}{2}\right) + 2.555(4') = 13.3558 \ ft^2$

$A_2 = 1(2.292) + .75(.75) + .75\left(\frac{.75}{2}\right) + 2.28(4') = 12.26 \ ft^2$

Vol. $= (A_1 + A_2)\left(\frac{32.75}{27}\right) = \underline{31.08 \ cy}$ ABUT #2

Vol. Concrete @ ABUT #1 + #2 $= 37.09 + 2(.7) + 2(10.92) + 4(6.11) + 31.08 = \underline{115.85 \ cy}$

TOTAL OF THIS Page

Parsons Brinckerhoff **Computation Sheet**

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Made by Ali

Date 10-23-89

Checked by DJ Church

Date 10/25/89

Subject 12th street over ACDC

Concrete QUANTITY

Sidewalk

Vol. $= \frac{2.25'+2.16'}{2} \times 9' \times \left(\frac{26.36+17.99}{27}\right) \times 2 = \underline{61.6}$ cy sidewalk @ approach slab DETAIL (R)

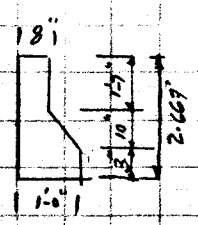
Vol. Transition sidewalk = by civil

Vol. sidewalk out side of Bridge = 61.6 cy ($f'_c = 4000$ psi)

Vol. sidewalk on Bridge $= \frac{8''+12''}{24} \times 9'(53.33) \times \frac{2}{27} = \underline{\underline{29.6}}$ cy ($f'_c = 3000$ psi)

Barrier (on Bridge & Approach slab)

$A = \frac{8''}{12} \times 2.667 + \frac{10'' \times 4''}{2 \times 144} + \frac{3'' \times 4''}{144} = 2.0\ ft^2$



Vol. $= 2.0\left(\frac{53.90' + 2' + 2'}{27}\right) \times 2 = \underline{\underline{14.51}}$ cy

Approach slab ($f'_c = 4000$ psi)

Vol. $= 1.25'(2')(40') \times \frac{2}{27} = \underline{\underline{77.8}}$ cy

TOTAL Vol. $f'_c = 3000$ psi concrete $=$ 115.85 (prev. page) $+ 31.0 + 14.51 = 161.$ cy

TOTAL Vol. $f'_c = 4000$ psi concrete $= 61.6 + 77.8 = 139$ cy

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
DRILLED SHAFT, FENCE & ASPHALT QUANTITY

Page 4 of 7 | 113
Made by ALi
Date 10-24-89
Checked by DJ Church
Date 10/25 89

DRILLED SHAFT

16 DRILLED SHAFT @ Retaining walls BOT. El 1184.00 (3'-0" ϕ)
Top Elevation Assume 1236.19 -6 + 11 -2.75 = El 1238.44 see SECTION (K)
El. BOT Ftg.

L = (1184 - 1238.44) = 54.44', Say 55'-0" / shaft
TOTAL = 16 * 55 = 880 ft

16 DRILLED SHAFT @ Abut. BOT. El. 1198.00 (3'-0" ϕ)
TOP El. 1236.19 + 0.25 = El 1236.44

L = (1236.44 - 1198) = 38.44'
TOTAL = 39 * 16 = 624'
TOTAL Length = 880 + 624 = 1504' (3'-0" ϕ) DRILLED SHAFT

FENCE

L = (sta 12+34.56 - sta 11+36.56) * 2 = 196'
98

ASPHALTIC CONCRETE 2"

Vol. = 40 * (sta 12+34.56 - sta 11+36.56) * 2"/12 = 653.33 ft³

Wt = 653.33 * 145/2000 = 47 TONS

Box BEAM (BT-48)

L = 53'-5"
price = 80.00 $/LF TOTAL Cost = 53.417 * 80 = 4273.$ / Box

Parsons Brinckerhoff Computation Sheet

Subject: 12th Street OVER ACDC
STEEL QUANTITY

Page 5 of 7 | 114
Made by: Ali
Date: 10-24-89
Checked by: DJ Church
Date: 10/25/89

ABUTMENT : See SECTION (A/-)

RETAINING WALL : See SECTION (K/-)

BAR SIZE	BAR LENGTH	NO OF BARS	TOTAL LENGTH	Wt. Per Linear Ft	TOTAL WEIGHT
#8	65.0 +3' Lap	12+2	952	2.67	2542
#5	65.0 +2'	6	402	1.043	419
#8@12" 1'-4"	5.75	66	379.5	2.67	1013.2
#4 @ 12" 1'-10"	3.08	66	2033	.668	136
#5@12" 17 5	2.25	66	148.5	1.043	154.9
#4@12" 8	5.09	66	335.9	.668	224
#6@12" 22 #9	12.5	66	825	1.502	1239.1
BACKWALL					
#5@12"	5.58	66	368.3	1.043	384.1
#5@12" 9 10	1.75	5(8)	70	1.043	73
#5@12"	65.+2	6	402	1.043	419
					TOTAL = 6604. * 2 (ABUT) = 13209 lbs
RETAINING WALL	(20' Long)				
#5@12"	10.6	20	212	1.043	221
#5@12"	19.6	10	190	1.043	204
#5@12" 9 10	1.75	4(8)	56	1.043	58. TOTAL 483 * 4 = 1932 lbs

TOTAL THIS Page = 15,141 lbs

ALL ABUTMENTS & Retaining walls

Parsons Brinckerhoff Computation Sheet

Page 6 of 7 | 115

Made by Ali

Date 10-24-89

Checked by DJ Church

Date 10/26/89

Subject 12th street over ACDC

STEEL QUANTITY

Barrier: DETAIL H sta 12+34.56 - sta 11+36.56 = 98'

Sidewalk " H

Approach SLAB " P

BAR SIZE	BAR LENGTH	NO OF BARS	TOTAL LENGTH	Wt. Per Linear Ft	TOTAL WEIGHT
~~Barrier~~ #4	98	2	196	1.502	131
#4	98	5	490	.668	327
#4@9" 1'6" 8" 2'	4.92	131	644	.668	431
#4@9" 1'5" 2'	4.92	131	644	.668	431
					TOTAL = 1320 * 2 = 2640 lbs
Sidewalk					
#4@9" 8'9" 8	9.25	71	657	.668	439
#4	5.433	3	163	.668	109
#4@1'-6"	98'	6	588	.668	392.8
#4@18" 1'3" 4'	2.67	36	96	.668	64
#4@9" 8'1" 1'5"	11.83	35+24	698	.668	466
					TOTAL = 1471 * 2 = 2942 lbs
Approach slab					
Longitudinal ⊥ #8@9" (58')	20.6	78	1606.8	2.67	4290.
#4@18" (40')	20.6	27	556.2	.668	371.5
Transv. 1'0"					
#5@12" (21')	70.5	21	1480	1.043	1544
#4@18" (21')	52'		728	.668	486
#4@18" 8" 1'0"		27	74	.668	50
#4@18" 4' 9" 10.5		27	150	.668	100
					TOTAL = 6842 * 2 = 13684 lbs

TOTAL THIS Page = 19,266

Parsons Brinckerhoff **Computation Sheet**

Subject 12th street OVER ACDC
QUANTITY SUMMARY

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Made by Ali
Date 10-25-89
Checked by DJ Church
Date 10/26/89

	STEEL LBS		
ABUTMENT + WALLS + RETA. WALLS	15,141		
Barrier + Sidewalk + Approach slab	19,266		
TOTAL	34,407		

f'_c = 3000 PSI Concrete (ABUT. + Sidewalk on Bridge + Barrier) = 161 cy

f'_c = 4000 PSI Concrete (Approach slab + Sidewalk on Approach slab) = 139 cy

Fence = 296'

ASPHALTIC Concrete = 47. TONS

BOX BEAM (BT-48) = 4273 #/Box (14-Box)

Parsons Brinckerhoff **Computation Sheet**

Subject 12th St. Over ACDC
Preliminary Cross Section

Page 1 of 117
Made by DJ Church
Date 8-8-89
Checked by Al.
Date 8-9-89

A. Bridge Width

1. Match existing roadway between curbs.

a. Existing roadway width = 2(20.5') = 41.0' between back of curbs.

b. Assume 7" curb width per MAG Standard Detail 220.

c. Existing Roadway Width Between Curbs –

$W_R = 41.0' - 2(.58') = 39.83'$

d. Use 40'-0" Bridge Roadway Width

2. Use 2-8'-0" Sidewalks one on each side of bridge.

3. Barriers – Use 1'-0" barriers each side of bridge.

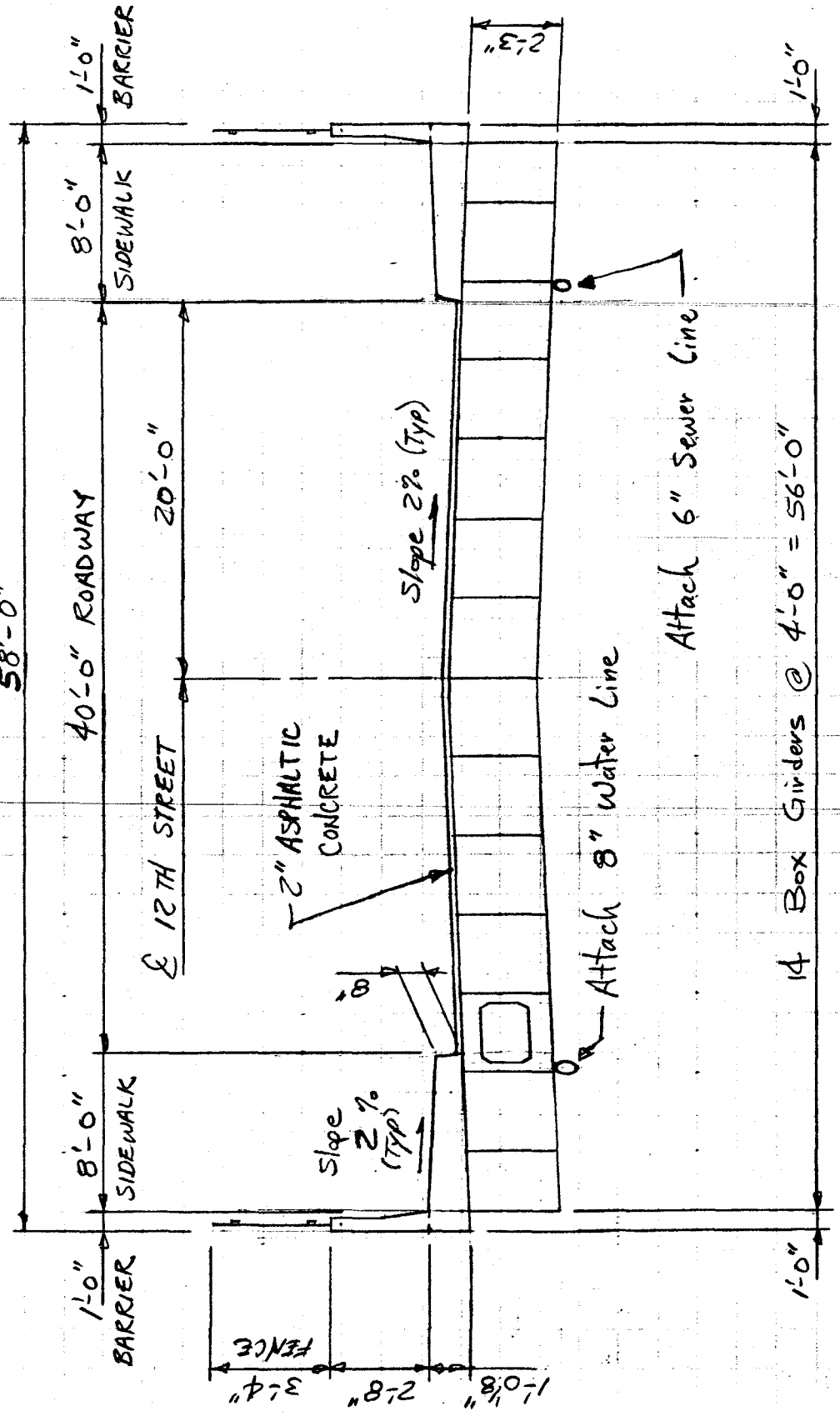
4. Total Bridge Width =

40'-0"	(Roadway)
16'-0"	(Sidewalks)
2'-0"	(Barriers)
58'-0"	

Parsons Brinckerhoff **Computation Sheet**

Subject: 12TH ST. OVER ACDC
PRELIMINARY CROSS SECTION

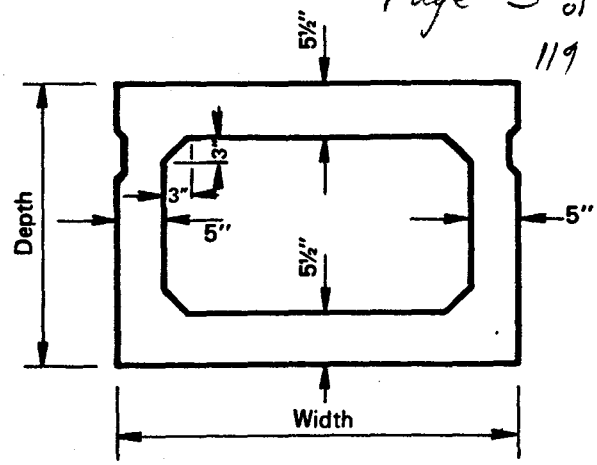
Page 2 of 118
Made by DJ Church
Date 8/11/89
Checked by Ali
Date 8-25-89

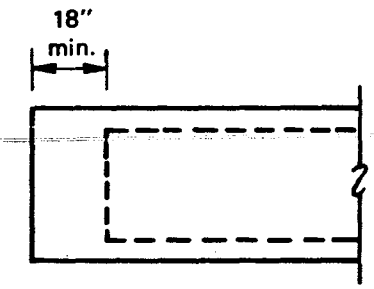


BRIDGE CROSS-SECTION

Box beams

The box beams shown are the former AASHO-PCI standard sections. They can be used either as adjacent units with or without an added wearing surface or spaced apart in which case the deck slab is cast-in-place using integral precast, prestressed deck panels. Box beams for railway loadings have been standardized by AREA.

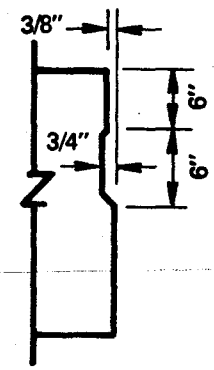




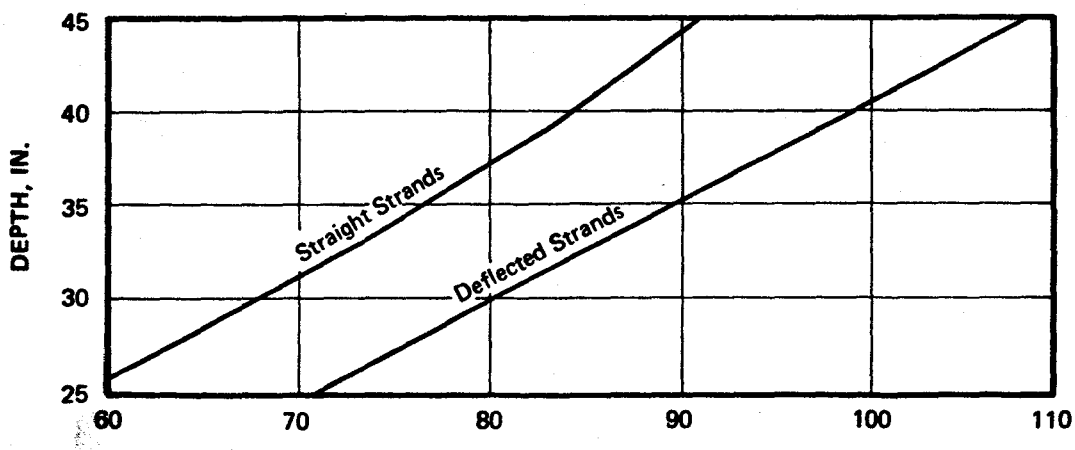
TYPICAL LONGITUDINAL SECTION

TYPICAL SECTION PROPERTIES

Type	Width ft	Depth in.	Weight lb/ft	Net Area in.^{2}	I_x in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}
B I-36	3	27	584	561	50,334	13.35	3770	3687
B II-36	3	33	647	621	85,153	16.29	5227	5096
B III-36	3	39	709	681	131,145	19.25	6813	6640
B IV-36	3	42	740	711	158,644	20.73	7653	7459
B I-48	4	27	722	693	65,941	13.37	4932	4838
B II-48	4	33	784	753	110,499	16.33	6767	6629
B III-48	4	39	847	813	168,367	19.29	8728	8542
B IV-48	4	42	878	843	203,088	20.78	9773	9571



TYPICAL KEYWAY DETAIL



APPROXIMATE MAXIMUM SPAN, FEET — HS20 LOADING

B. Bridge Length

1. Future Channel Width

a. Clear Width, W_c = 40'-0"

b. Walls - assume 12" thick walls

2. Abutment Width

a. Drilled Shafts - use 3.0' dia. per Geotech. Report

b. Edge distance from shaft to front face of abutment - use 3" min.

c. Distance from ℄ of bearing to front face of abutment - use 1'-6"

d. Distance from ℄ bearing to backwall - use 1'-6"

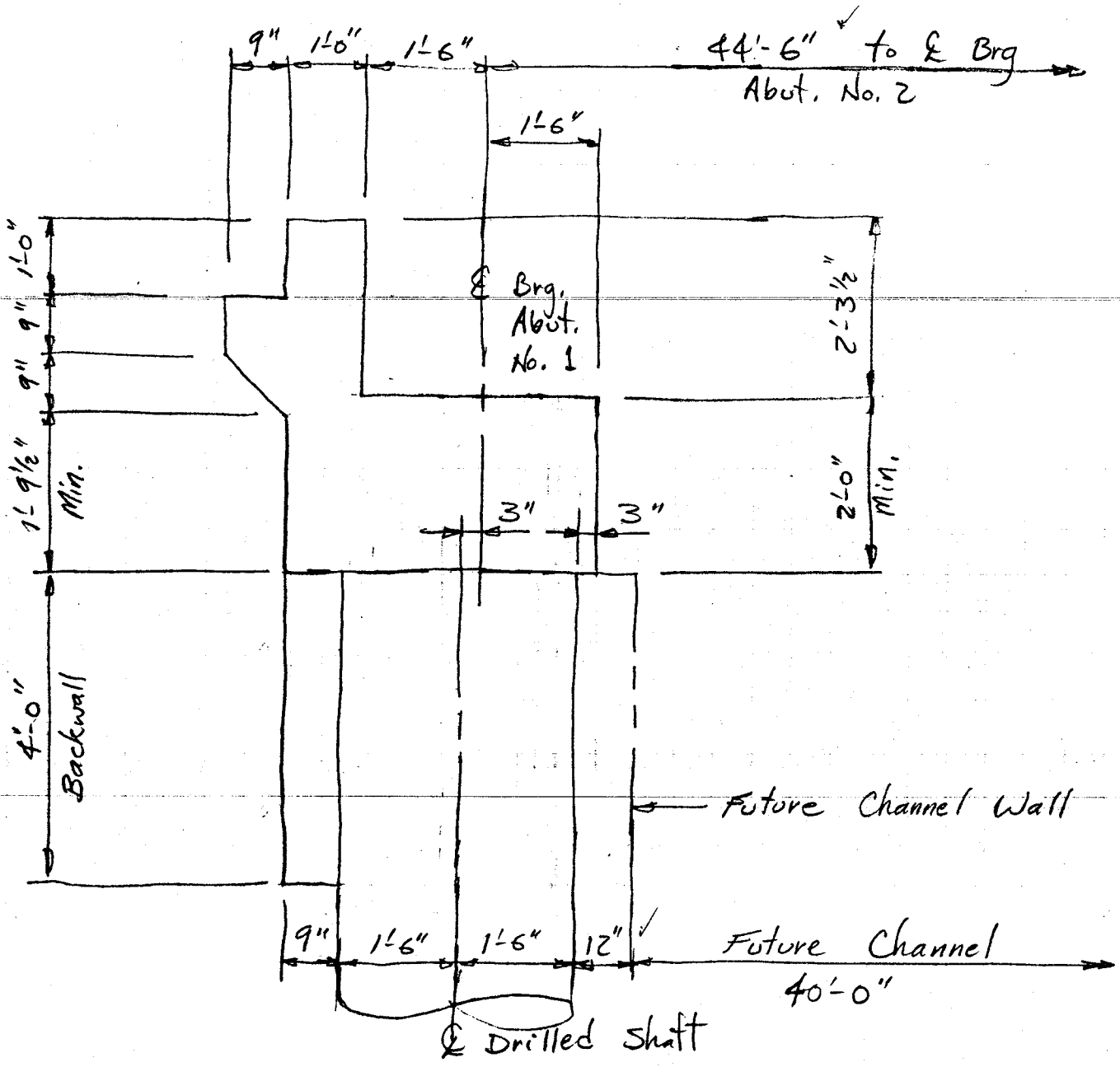
e. Backwall thickness - 1'-0"

f. Total width, W_A = 1.5' + 1.5' + 1.00' = 4.00'

Parsons Brinckerhoff **Computation Sheet**

Subject: 12th Street over ACDC
Preliminary Design

Page 4 of 121
Made by: DJ Church
Date: 8-11-89
Checked by: AR
Date: 8-25-89



ABUT. SECTION
Normal to ℄ ACDC

omputation Sheet

over ACDC
sign

Use lateral ties stepped at center diaphragm per AASHTO / PCI

(AASHTO / PCI)

, T-PAC, 8-9-89)

> L/1000 = 5/8"

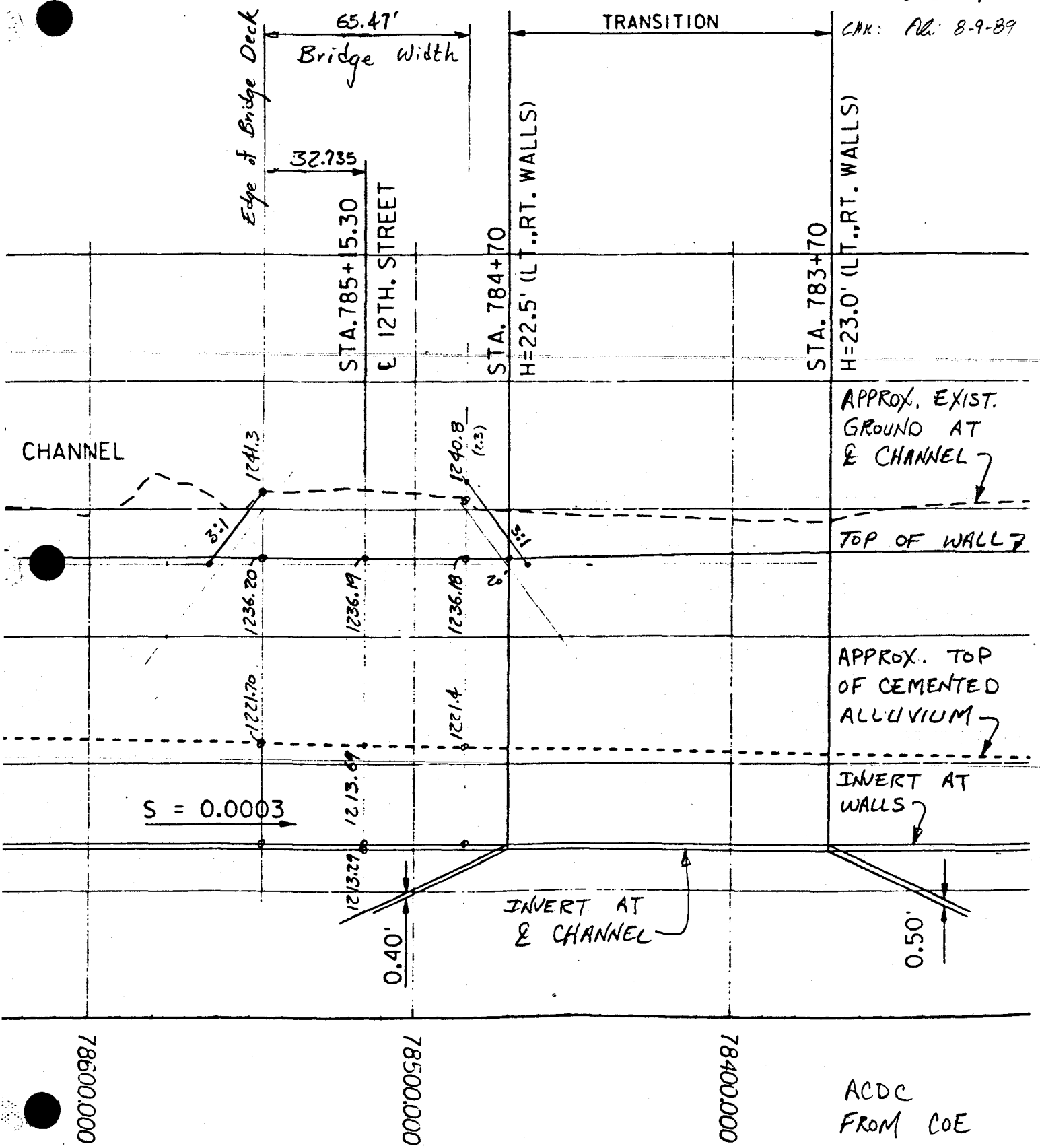
in, T-PAC, 8-9-89)

≤ L/1000 = 5/8"

oncrete Box Beams
ction to L/1000.
Diaphragm at

Page 8 of

BY: DJ CHURCH
8-3-89
CHK: AL 8-9-89



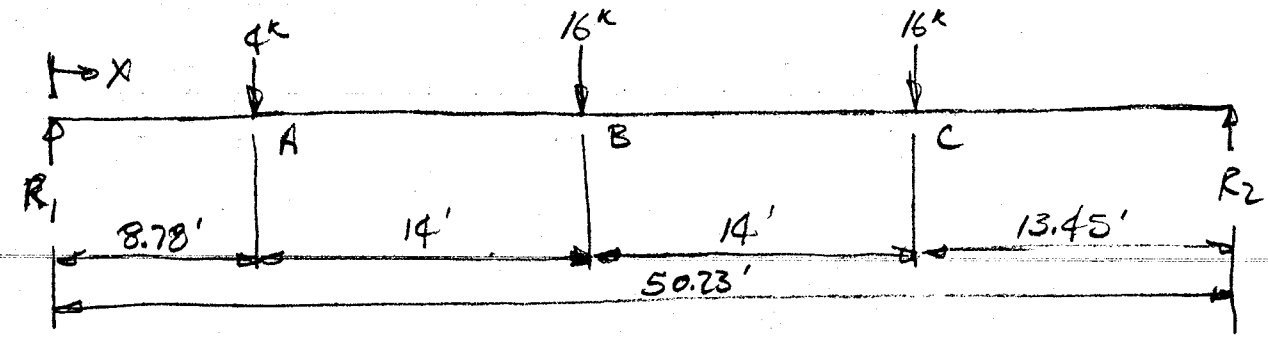
ACDC
FROM COE
SHEET NO. 29

PROFILE

VERT. SCALE: 1 IN. = 10 FT.
HORIZ. SCALE: 1 IN. = 40 FT.

Subject 12th St. Over ACDC
Preliminary Design

Live Load Deflection:



Max. Δ @ $X = 22.78'$

Δ @ B due to 16^k @ B: (AISC Simple Beam Δ)

$$\Delta_{BB} = \frac{Pa^2b^2}{3EI\ell} = \frac{16^k(22.78)^2(27.45)^2}{3EI\ell} = \frac{2{,}085{,}407}{EI\ell}\ k\text{-}ft^4$$

$$\Delta_{BA} = \frac{Pbx}{6EI\ell}(\ell^2-b^2-x^2) = \frac{4^k(8.78')27.45}{6EI\ell}\left(\overline{50.23}^2-\overline{8.78}^2-\overline{27.45}^2\right)$$

$$= \frac{4^k(241.01\ ft^2)}{6EI\ell}(1692.46\ ft^2) = \frac{271{,}934}{EI\ell}\ k\text{-}ft^4$$

$$\Delta_{BC} = \frac{16^k\overbrace{(13.45)(22.78)}^{306.39}}{6EI\ell}\left(\overline{50.23}^2-\overbrace{\overline{13.45}^2-\overline{22.78}^2}^{1823.22}\right) = \frac{1{,}489{,}650}{EI\ell}$$

$$\Sigma\Delta_B = \Delta_{BB}+\Delta_{BA}+\Delta_{BC} = \frac{3{,}846{,}991}{EI\ell}\ k\text{-}ft^4$$

$$\Sigma\Delta_B \text{ w/ Impact} = 1.285\left(\frac{3{,}846{,}991}{EI(50.23)}\right) = \frac{98{,}415}{EI}$$

Voided Slab $\Delta = 98415/1027558 = .0958' = 1.15'' > \ell/1000 = .60''$

Box Beam $\Delta = 98415/1963039 = .0501' = 0.60'' \leq \ell/1000$, O.K.

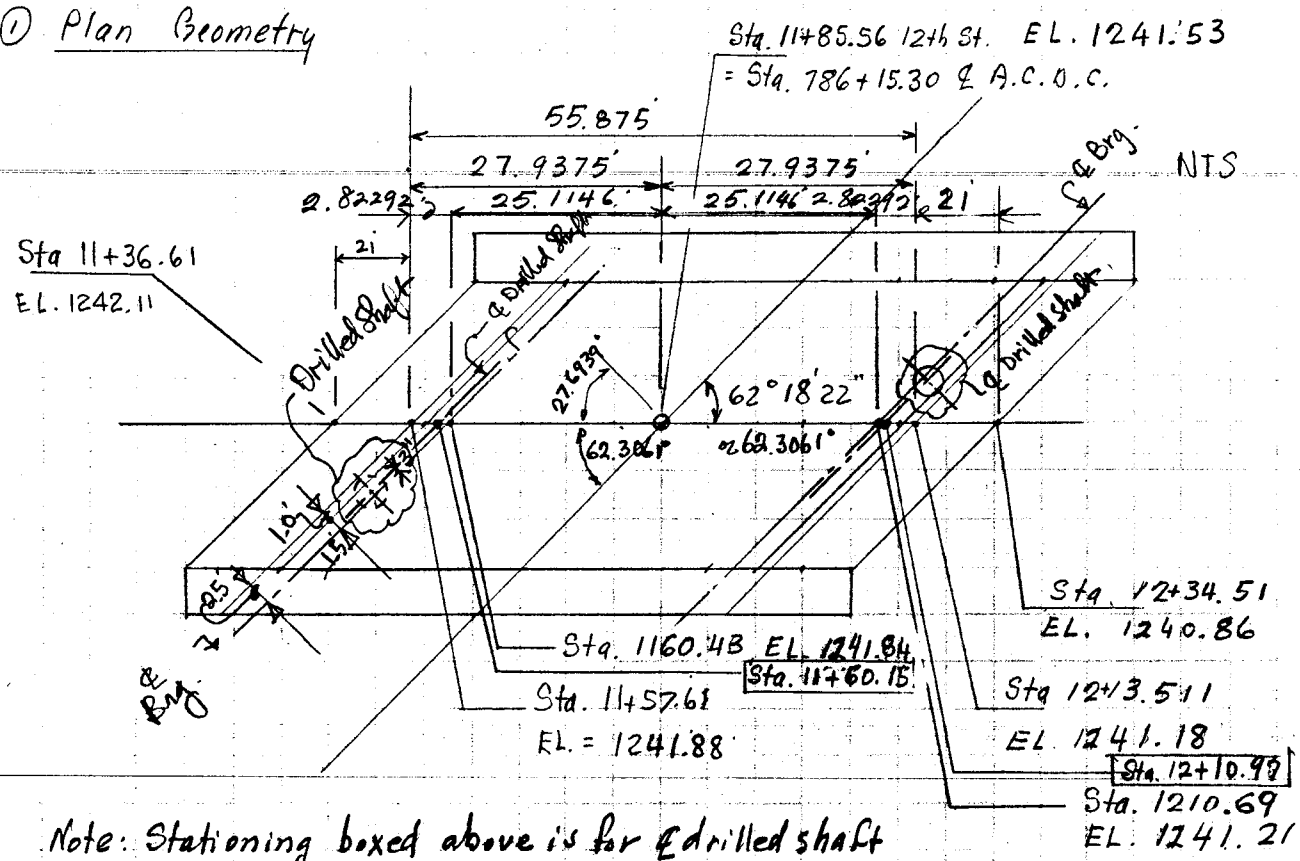
Parsons Brinckerhoff Computation Sheet

Subject: 12th St. over ACDC
Geometry

Page ____ of ____ 127
Made by: WVELLIOTT
Date: 10-17-89
Checked by: ____
Date: ____

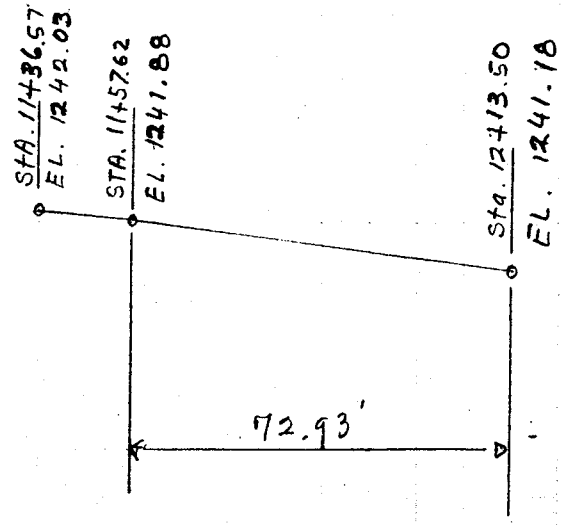
Geometric layout & Elevations
Refer to Civil Drawing by Dave Healy, Steve Scott as of 10-17-89

① Plan Geometry



Note: Stationing boxed above is for ℄ drilled shaft fdn. layout plan.

② EL. ℄ Road
See next page for summary of elevations.



③ Skewed Dimensions at Abut.

where cos 27.6939° = 0.88544

1.0' × 0.88544 = 1.12938
1.5' × 0.88544 = 1.6941
.25 × 0.88544 = 0.28235
2.5 × 0.88544 = 2.8235
2.25 × 0.88544 = 2.5411
1.25' × $\frac{1}{.88544}$ = 1.41173'

Parsons Brinckerhoff **Computation Sheet**

Subject 12th St over ACDC
Geometry

Page ____ of ____ 128
Made by WVELLIOTT
Date 10-17-89
Checked by
Date

Geometric Layout and Elevations

(4) Channel Invert
Refer to Hydraulic Plan & Profile US Army Corps of Engineers
Sta. 820+00 To Sta. 784+20

(785+15.30) − (784+20) = 95.30'
(95.30')(0.0003) = 0.02859'
1213.66 + .02859 = 1213.69
1213.69 − 0.40 = 1213.29

(5)

Sta. 11+57.61	Δ = 1.13'	* EL. 1241.88 r		Back of Backwall Abut 1
11+58.74	Δ = 1.41'	1241.87 r		Face of Backwall Abut. 1
11+60.15	Δ = 0.28'	1241.85 r		℄ Drilled Shaft Abut. 1
11+60.43	Δ = 25.13'	1241.84 r		℄ Brg. Abut. 1
11+85.56	Δ = 25.13'	1241.53 r		Center of ACDC
1210.69	Δ = 0.28'	1241.22 r		℄ Brg. Abut. #2
1210.97	Δ = 1.41'	1241.21 r		℄ Drilled Shaft Abut. #2
1212.38	Δ = 1.13'	1241.19 r		Face of Backwall Abut. 2
1213.51		* EL. 1241.18 r		Back of Backwall Abut. 2

* All elevations to top of pavement

For Abut #1 - T/Backwall, High Pt. 1241.87 − .1667' = 1241.70
Seat/℄ Brg (1) 1241.84 − .1667 − 2.2917' = 1239.38
Seat/℄ Brg (2) 1241.22 − .1667 − 2.2917 = 1238.76
T/Backwall, High Pt. 1241.19 − .1667' = 1241.023'

(6) Determine Distance between ℄ Bearing
40' + 2' + 1.25' + 1.25' = 44.50'
44.50 / 0.88544 = 50.26'

Parsons Brinckerhoff Computation Sheet

Subject: 12th St. over ACDC

Page ____ of ____ 129

Made by: W V ELLIOTT

Date: 10-18-89

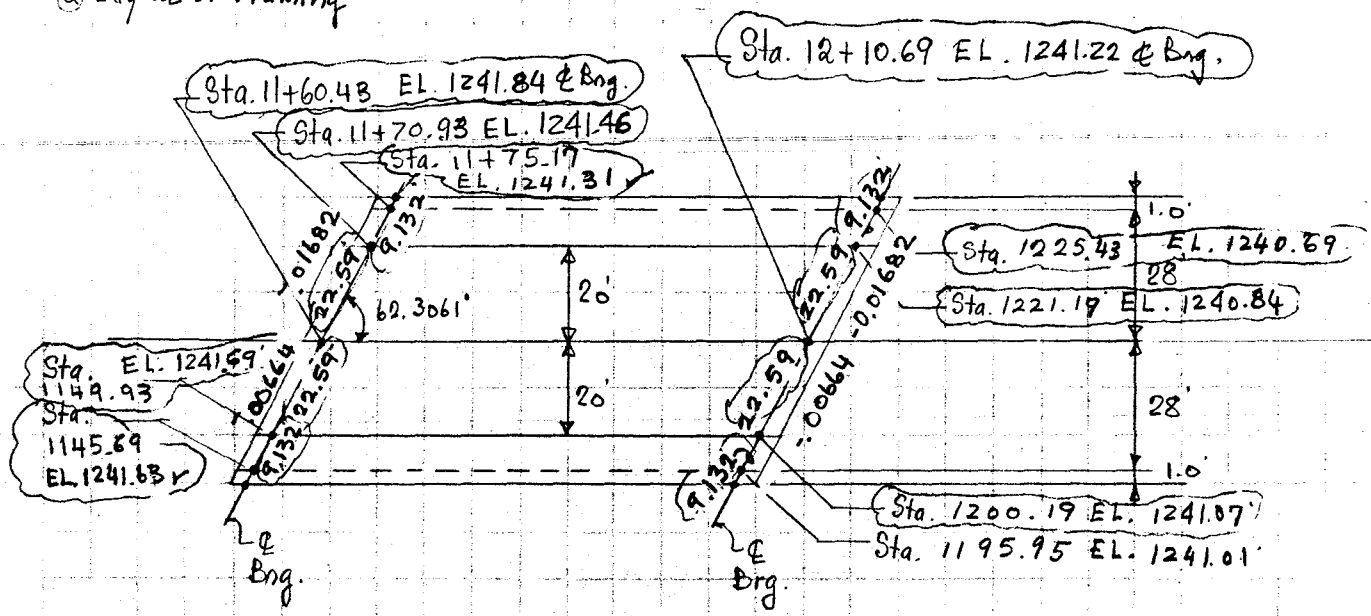
Checked by: ____

Date: ____

Geometry – Layout & Elevations

(7) Determine elevation of drilled shaft & abutment seat

(a) Layout of Framing



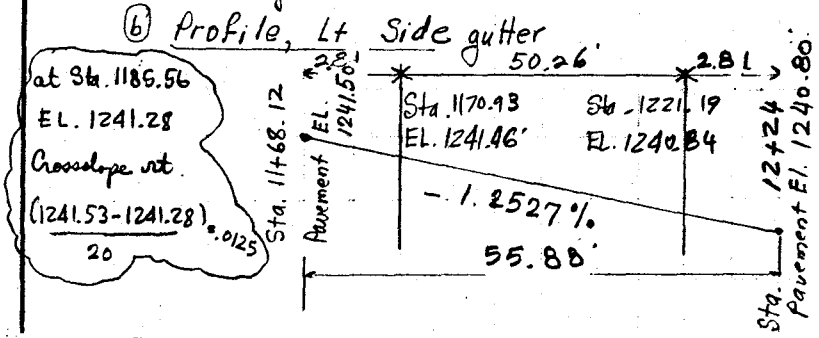
Note: 28' is to edge of exterior box

28.0833' is to edge of joint filler or to remote corner of abutment seat.

$\frac{28.0833'}{adj.} = \tan 62.3061°$, adj = 14.74', $\frac{28.0833'}{hyp.} = \sin 62.3061°$, hyp. = 31.72'

$\frac{20'}{adj.} = \tan 62.3061°$, adj. = 10.4975', $\frac{20}{hyp.} = \sin 62.3061°$, hyp. = 22.5876'

(b) Profile, Lt Side gutter



Sta. 1175+17 EL. 1241.31'

Sta. 1225.43 EL. 1240.69

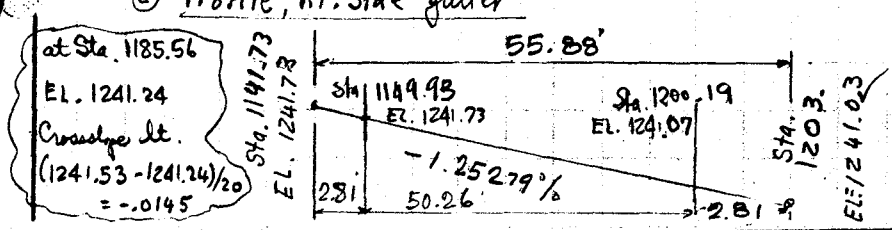
EL. Seat Far Lt. Abut. 1

1241.31 - .1667' - 2.292' = 1238.85

EL. Seat Far Lt. Abut. 2

1240.69 - .1667' - 2.292 = 1238.23

(c) Profile, Rt. Side gutter



Sta. 1145.69 EL. 1241.63'

Sta. 1195.95 EL. 1241.01'

EL. Seat Far Rt. Abut. 1

1241.63 - .1667' - 2.292' = 1239.17

EL. Seat Far Rt. Abut. 2

1241.01 - .1667' - 2.292' = 1238.55

T/wall 1213.69 + 22.5 = 1236.19