DESIGN COMPUTATIONS

PROJECT # BR R600-297
SUBACCOUNT # 16212
STRUCTURE # F-16-XB
MSE PANEL WALLS-F-16-EC, EE AND EF
Volume 2

Designer: Andy Pott

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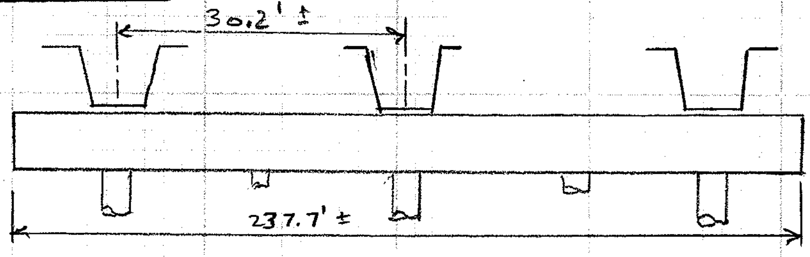
DESIGN COMPUTATIONS (Grid)

ABUTMENT



DEAD LOADS

GIRDERS ≈ 112.6 K
DECK + HAUNCH ≈ 207.9 K
RAIL ≈ 9.1 K
WEARING COURSE ≈ 43.2 K
} PER GIRDER

Approach Slab
SLAB : (197')(1')(10') .15 ≈ 295.5 K
RAIL : 2 (3.22 SF) (10') .15 ≈ 9.7 K
WEARING COURSE : (194)(.25)(10') (.1467) ≈ 71.2 K
} Total Load
Assume Uniform Load on Abutment

ABUTMENT : ~ (11')(2.5')(.15 k/cf) ≈ 4.13 K/ft (final)

6'+5½"+8"
3'-6"
2'-6

~ (3.5)(2.5) (.15 k/cf) ≈ 1.32 k/ft (intermediate)

LIVELOAD ≈ 96.1 K / lane

Assume Intermediate Caissons
$\ell \approx 15.1'$

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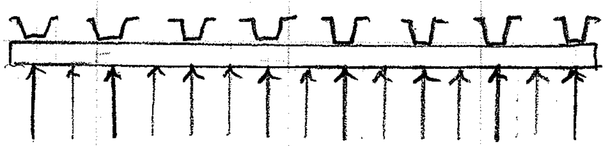
DESIGN COMPUTATIONS (Grid)



← assume 15 Caissons

FINAL DEAD LOAD

SERVICE I

$$DL \approx 8(112.6 + 207.9 + 9.1 + 43.2) + \underbrace{(295.5 + 9.7 + 71.2)}_{\text{approach slab}}$$

$$+ (4.13\ k/ft)(237.7')$$

$$DL \approx 4340.5\ k$$

$$DL/caisson = 4340.5/15 \approx 289.4\ k$$

STRENGTH I

$$DL \approx 8\left[(112.6 + 207.9 + 9.1)1.25 + 43.2(1.5)\right] + 1.25(295.5 + 9.7)$$

$$+ 1.5(71.2) + (4.13\ k/ft)(237.7')\ 1.25$$

$$DL \approx 5529.8\ k$$

$$DL/caisson \approx 5529.8/15 \approx 368.7\ k$$

INTERMEDIATE DEAD LOAD

SERVICE I

$$DL \approx 8(112.6 + 207.9) + (4.13\ k/ft)(237.7')$$

$$DL \approx 3545.7$$

$$DL/caisson \approx 3545.7/15 \approx 236.4\ k$$

STRENGTH I

$$DL \approx (3545.7)\ 1.25 \approx 4432.1\ k$$

$$DL/caisson \approx 4432.1/15 \approx 295.5\ k$$

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Caisson Spacing Along Skew $\approx$ 15.1'

$M_{uniform} \approx .125 w l^2$ (uniform load negative moment, 3 supports p 2-312, AISC Manual of Steel Const.)

INTERMEDIATE

SERVICE I

$w = 3545.7\text{ k}/237.7' \approx 14.92\text{ k/ft}$

$M_{DEAD} \approx .125(14.92)(15.1)^2$

$M_{DEAD} \approx 425.2\text{ ft K}$

STRENGTH I

$w = 4432.1/237.7 \approx 18.65\text{ k/ft}$

$M_{Dead} \approx .125(18.65)(15.1)^2$

$M_{DEAD} \approx 531.5\text{ ft K}$

FINAL

DEAD

SERVICE I

$w \approx 4340.5\text{ k}/237.7 \approx 18.26\text{ k/ft}$

$M_{DEAD} \approx .125(8.26)(15.1)^2$

$M_{DEAD} \approx 520.4\text{ ft k}$

STRENGTH I

$w \approx 5529.8\text{ k}/237.7 \approx 23.26\text{ k/ft}$

$M_{DEAD} \approx .125(23.26)(15.1)^2$

$M_{DEAD} \approx 662.9\text{ ft K}$

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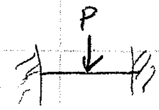
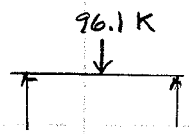
DESIGN COMPUTATIONS (Grid)

FINAL (cont.)



LIVE LOAD



$$M_{max} \approx \frac{P\ell}{8}$$

lanes loaded

$$M \approx 1.2 \, \frac{(96.1)\,15.1}{8}$$

$$M \approx 217.7 \text{ ft K}$$

$$M_{U_{SERVICE}} = 520.4 + 217.7$$

$$M_{SERVICE} = \underline{738.1 \text{ ft K}}$$

$$M_{U\ STRENGTH} = 662.9 + 1.75\,(217.7)$$

$$M_{U\ STRENGTH} = \underline{1043.9 \text{ ft K}}$$

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REINFORCING DESIGN

Intermediate

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 42.00 inches
b= 30.00 inches
bar diameter= 1.410 inches

TOP STEEL

LOAD TYPE	MhTOT
	ft-K
Mh (UNFACTORED)	425.20
STRENGTH I	531.50
SERVICE I	425.20

ASreq'd
in^2
2.44
3.07
2.44

ds= 39.30 inches
per 5.10.8.2 ASteemp= 1.89 sq inches

Use # 11 at top face
use spacing= 15.00 inches
As= 3.120 sq. inches
min. spacing = 15.25 inches

compressive steel:
Use # 5 at bottom face
As'= 0.00 sq. inches

2 #11 T & B

Mn= 600.55 ft-K
Mr= 540.50 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 1.98 inches
de=ds= 39.30 inches (for no prestressing)
c/de= 0.05
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 449.04 ft-K <--- Test 1
1.33MhTOT (max.)= 706.90 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 2.09 sq inches
Use # 4 transverse reinforcement
min. spacing = 2.87 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (t_s)= 127.50 inches
b= 30.00 inches
bar diameter= 1.410 inches

Final

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	738.10	1.32
STRENGTH I	1043.90	1.87
SERVICE I	738.10	1.32

2 #11 T&B

d_s= 124.80 inches
per 5.10.8.2 AS_{temp}= 5.74 sq inches

Use # 11 at top face min. spacing = 25.07 inches
use spacing= 15.00 inches
As= 3.120 sq. inches

compressive steel:

Use # 5 at bottom face
As'= 0.00 sq. inches

M_n= 1934.35 ft-K
M_r= 1740.92 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 1.98 inches
d_e=d_s= 124.80 inches (for no prestressing)
c/d_e= 0.02

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 4138.17 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 1388.39 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 2.09 sq inches
Use # 4 transverse reinforcement
min. spacing = 2.87 inches

SHEAR

INTERMEDIATE

$V_{uniform} \approx .625\, w\, l$ (uniform load, 3 supports p 2-312, AISC Manual of Steel Const.)

$w_{strength\,I} \approx 18.65\ k/ft$

$V_U \approx .625\,(18.65\ k/ft)\,(15.1')$

$V_U \approx 176.0\ K$

$V_c = .0316\,\beta\,\sqrt{f'_c}\ b_v d_v$ (5.8.3.3-3)

assume: $\beta = 2$
$\theta = 45°$
$f'_c = 4.5\ ksi$
$b_v = 30"$
$d_v = 42" - 2" - 5/8" - 1.41"/2 \approx 38.67"$

$V_c = .0316\,(2)\sqrt{4.5}\,(30")\,(38.67")$

$V_c \approx 155.5\ K$

$v_u = \dfrac{|V_u - \phi V_p|}{\phi b_v d_v} = \dfrac{142.8}{30(38.67")} = .123$ (5.8.2.9-1)

$v_u < .125 f'_c = .125(4.5) = .5625$

$S_{max} = .8\, d_v = .8\,(38.67) = 30.94" \leq 24.0"$ (5.8.2.7-1)

$A_v \geq \dfrac{.0316\sqrt{f'_c}\ b_v\, s}{f_y}$ (5.8.2.5-1)

$A_v \geq \dfrac{.0316\sqrt{4.5}\,(30")(12")}{60} = .40\ in^2$

$\#5 = .31\ in^2$

$2\ legs = .62\ in^2$

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DESIGN COMPUTATIONS (Grid)

Use #5 @ 12" (two legs) (Design Manual Sec 7.2 p 3)

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \quad (5.8.3.3-4)$$

$$V_s = \frac{2(.31)\,60\,(38.67")(\cot 45° + \cot 90)\sin 90}{12"}$$

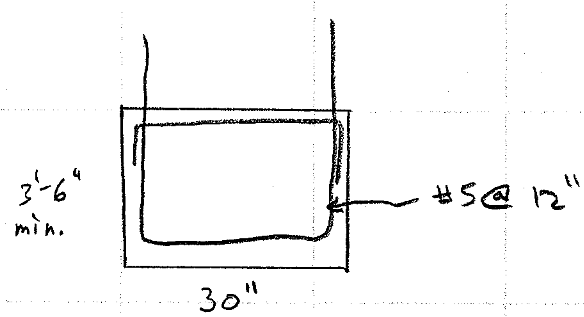
$$V_s \approx 119.9 \text{ K}$$

$$V_n = V_c + V_s = 155.5 + 119.9$$

$$V_n \approx 275.4 \text{ K}$$

$$V_r = \phi V_n = .9\,(275.4)$$

$$V_r \approx 247.9 \text{ K} > V_u = 176.0 \text{ K}$$ ✓ okay



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SHEAR

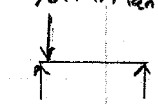
FINAL

$$V_{uniform} \approx .625\, w\, l$$

$$\omega_{strength\ I} \approx 23.26\ k/ft$$

$$V_{u_{DEAD}} \approx .625\,(23.26\,k/ft)\ 15.1'$$

$$V_{u_{DEAD}} \approx 219.5\ K$$



$LL = 96.1\ K/lane$

$$V_{u\,live} \approx 1.2\,(96.1\,k)\ \underset{}{\overset{\text{lanes loaded}}{1.75}}$$

$$V_{u\,live} \approx 201.8\ K$$

$$V_{u\,final} = 219.5 + 201.8\ k$$

$$V_{u\,final} \approx \underline{421.3\ K}$$

$$V_c = .0316\,\beta\sqrt{f'_c}\ b_v\, d_v \qquad (5.8.3.3-3)$$

$$d_v \approx 127.5'' - 2'' - 5/8'' - 1.41/2'' \approx 124''$$

assume $\beta = 2$
$\theta = 45°$
$b_v = 30''$
$f'_c = 4.5\ ksi$

$$V_c = .0316\,(2)\sqrt{4.5}\ (30'')\ 124''$$

$$V_c \approx 498.7\ K$$

Use #5 @ 12" (two legs)

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha) \sin\alpha}{s} \qquad (5.8.3.3\text{-}4)$$

$$V_s = \frac{2(.31)(60)(124")(\cot 45° + \cot 90°) \sin 90°}{12"}$$

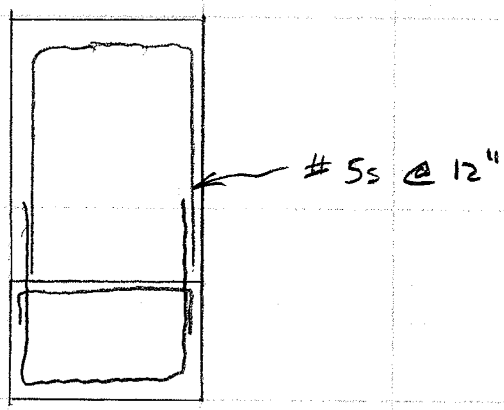
$V_s = 384.4$ K

$V_n = V_c + V_s = 498.7 + 384.4$ K

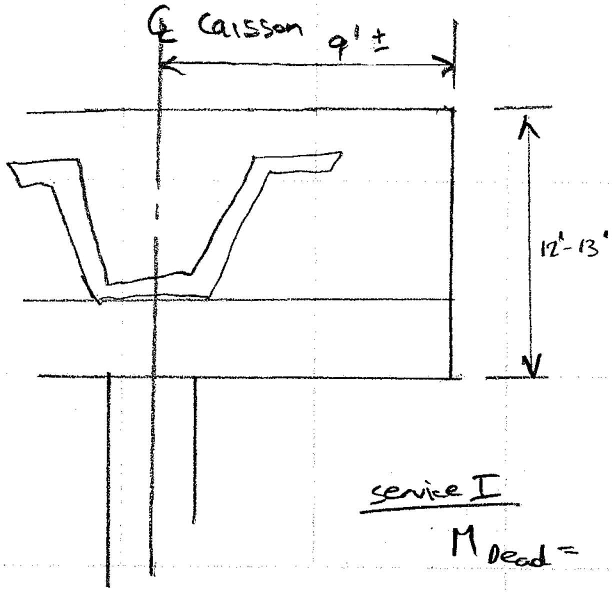
$V_n = 883.1$ K

$V_r = \phi V_n = .9(883.1)$

$V_r = 794.8$ K $> V_u = 421.3$ K ✓ okay



Abutment Phasing



slab ≈ 1.5 k/ft
asphalt ≈ .37 k/ft
rail ≈ .05 k/ft
} approach slab

$$M = \frac{wl^2}{2}$$

service I

$$M_{Dead} = \left\{(1.5\ k/ft) + (.37) + .05\right\}\frac{9^2}{2}$$

$$+ (13)(2.5)(.15)\ 9^2/2$$

$$M_{Dead} \approx 275.2\ k\,ft$$

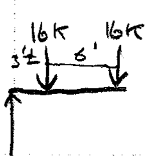
Strength I

$$M_{DEAD} \approx \left\{(1.5 + .05)\,1.25 + .37(1.5)\right\}\frac{9^2}{2}$$

$$+ \left[(13)(2.5)(.15)\right](1.25)\ \frac{9^2}{2}$$

$$M_{DEAD} \approx 347.7\ k\,ft$$

LIVE LOAD



$$M = Pb$$

service I

$$M = (16)9 + 16(3) \approx 192\ k\,ft$$

STRENGTH I

$$M = 192k(1.75) \approx 336\ ft\,K$$

REINFORCING DESIGN

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.00	inches
$\Phi_{flexure}$=	0.90	
Beam Thickness (ts)=	36.00	inches
b=	30.00	inches
bar diameter=	1.410	inches

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	275.20	1.86
STRENGTH I	347.70	2.36
SERVICE I	275.20	1.86

d_s= 33.30 inches

per 5.10.8.2 AS_{temp}= 1.62 sq inches

Use # 11 at top face min. spacing = 19.79 inches

use spacing= 15.00 inches

As= 3.120 sq. inches

compressive steel:

Use # 5 at bottom face

As'= 0.00 sq. inches

M_n= 506.95 ft-K

M_r= 456.26 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 1.98 inches

d_e=d_s= 33.30 inches (for no prestressing)

c/d_e= 0.06

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 329.91 ft-K <--- Test 1

1.33M_{hTOT} (max.)= 462.44 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft

Flange Width = 43 inches

Flange Overhang = 18 inches

Effective Length S = 8.86 ft

220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement

Required As = 2.09 sq inches

Use # 4 transverse reinforcement

min. spacing = 2.87 inches

REINFORCING DESIGN

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.00	inches
$\Phi_{flexure}$=	0.90	
Beam Thickness (ts)=	127.50	inches
b=	30.00	inches
bar diameter=	1.410	inches

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	467.20	0.83
STRENGTH I	683.70	1.22
SERVICE I	467.20	0.83

d_s= 124.80 inches

per 5.10.8.2 AS_{temp}= 5.74 sq inches

Use # 11 at top face   min. spacing = 38.34 inches

use spacing= 15.00 inches

As= 3.120 sq. inches

compressive steel:

Use # 5 at bottom face

As'= 0.00 sq. inches

M_n= 1934.35 ft-K

M_r= 1740.92 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 1.98 inches

$d_e=d_s$= 124.80 inches (for no prestressing)

c/d_e= 0.02

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

$1.2{*}M_{cracking}$= 4138.17 ft-K <--- Test 1

$1.33M_{hTOT}$ (max.)= 909.32 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing =	10.94	ft
Flange Width =	43	inches
Flange Overhang =	18	inches
Effective Length S =	8.86	ft
220 / sqrt(S) =	73.92	%
Required As =	2.09	sq inches
Use #	4	transverse reinforcement
min. spacing =	2.87	inches

Use **67** % of required main reinforcement

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

@ midpoint $\approx 4.5'$

$$M_{DEAD} = (1.5 + .37 + .05)\frac{4.5^2}{2} + 13(2.5)(.15)\frac{4.5^2}{2}$$

$$M_{DEAD} \approx 68.8 \text{ ft k}$$

STRENGTH I

$$M_{DEAD} \approx 86.9 \text{ ft k}$$

$$LL \approx 16(4.5) \approx 72 \text{ ft k}$$

$$\text{Strength I} \approx 1.75(72) = 126 \text{ ft k}$$

STILL OKAY

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SHEAR

$$V_{max} = \left[1.5 + .37 + .05 + (13)(2.5)(.15)\right] 9 + 32\ K$$

$$V_{max} \approx 93.2\ K$$

$$V_{U\,max} = \left[(1.5 + .05)\,1.25 + .37(1.5) + (13)(2.5)(.15)\,1.25\right] 9 + (32\ K)(1.75)$$

$$V_{U\,max} \approx 133.3\ K$$

$V_c = 155.5\ K$ ✓ ok

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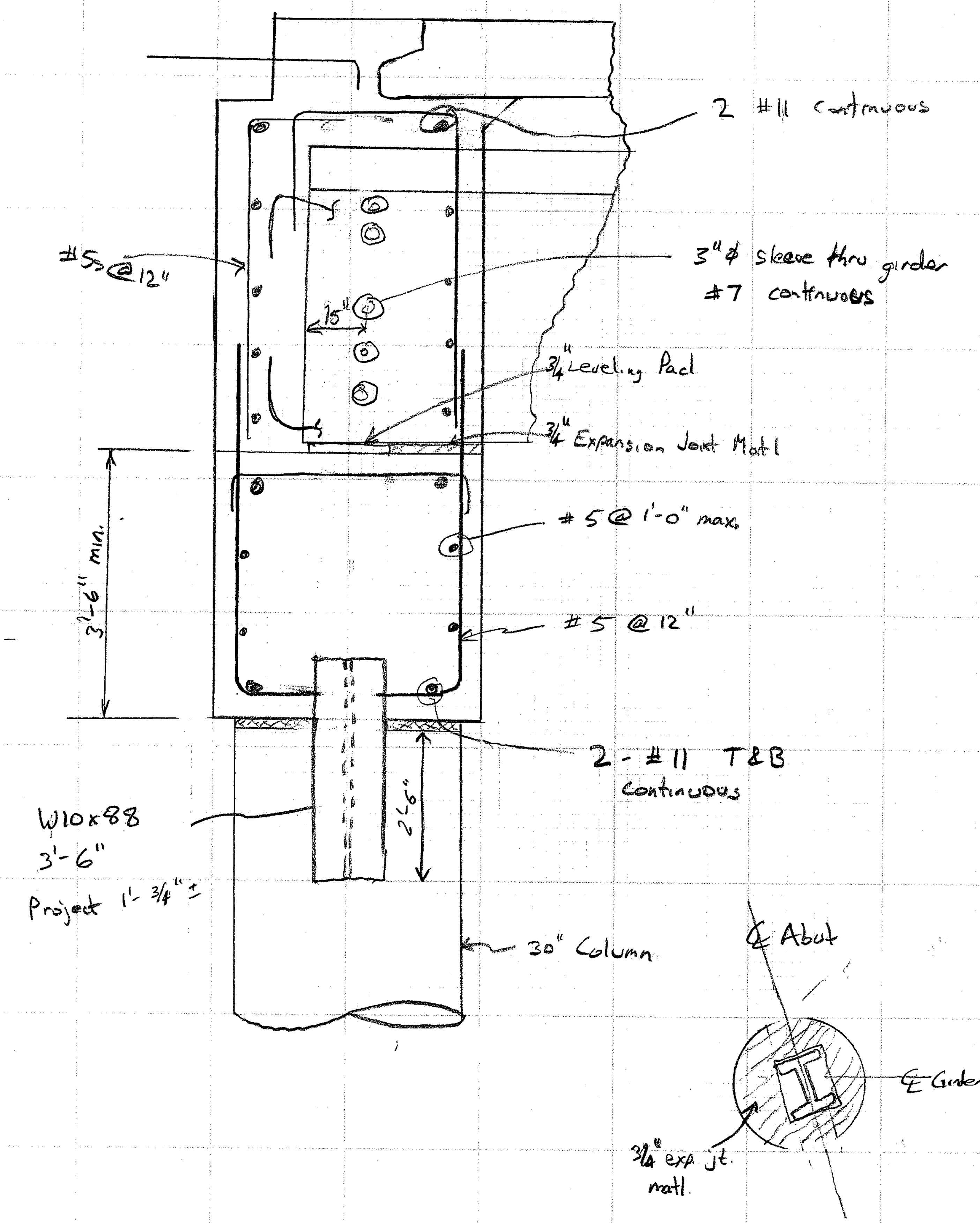

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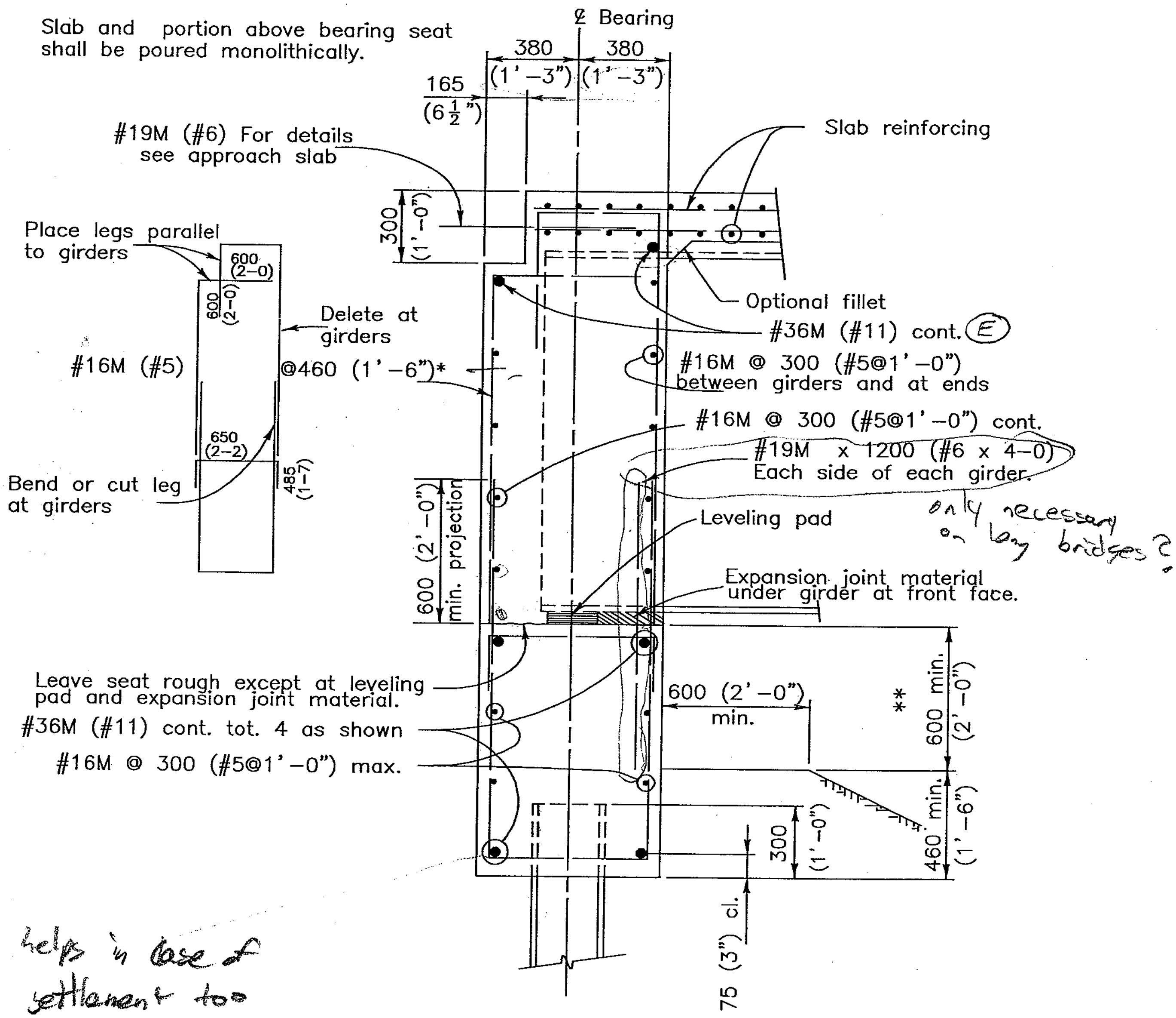


TYPICAL ABUTMENT SECTION

Note: All abutment and wingwall concrete shall be Class D (Bridge)

Extend strands from the bottom of precast sections into abutment, anchor the bottom of steel sections to abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.

* 300 (1'-0") if structure length longer than 90M (300') or ** greater than 1050 (3'-6").

Shear at Top of Caisson/Column

for temperature movement

length of contribution ≈ 368'/2 ≈ 184'

Temp movement = 90° * .000006 * 184' * $\frac{12''}{ft}$ ≈ 1.2"

to account for shrinkage & creep multiply by 1.6

Δ_s = total movement ≈ 1.6 * 1.2 ≈ 1.92"

2.2" used in LPILE

FROM LPILE:

maximum shear ≈ 27.5 Kip - LL only

≈ 24.1 kips - max load (LL + DL)

Horizontal Connection force ≈ .25 Vertical Loads
per 3.10.9.2

Max Dead Load ≈ 5529.8/17 ≈ 325.3 K dead load (strength I)

Max Live Load ≈ 89.2 K

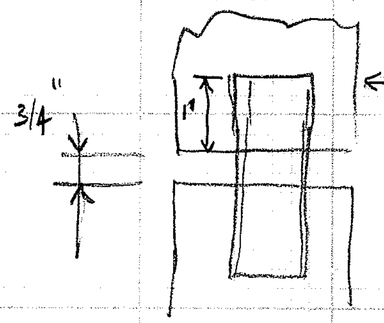
Total Load ≈ 325.3 + 89.2(1.75) ≈ 481.4 K

horizontal force = .25 (481.4) ≈ 120.4 K

Use 120.4 K

Max load ≈ 481.4 K (Strength I)

347.3 K (Service I)



(Seismic)

$M_u \approx (1.0625)\ 120.4\,K \approx 127.93$ ft K

$P_u = 481.4$ K

$$\lambda = \left(\frac{K\ell}{r_s \pi}\right)^2 F_y/E \qquad (6.9.4.1\text{-}3)$$

assume $K = 2.1$

$\ell \approx 12.75''$

$r_s \approx 2.68''$ W10x112

$F_y = 36$ ksi

$$\lambda = \left(\frac{(2.1)(12.75)}{2.68\ (3.1416)}\right)^2 \frac{36 \text{ ksi}}{29000} \approx .01256 \quad \text{W10x112}$$

$$P_n = .66^{\lambda} F_y A_s \qquad (6.9.4.1\text{-}1)$$

$P_n = .66^{.01256}(36)(32.9\ in^2) \approx 1178.2$ K

$\phi = .9$

$P_R = \phi P_N = .9\,(1178.2) \approx 1060.4$ K

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$$M_{Rx} = Z_x F_y \approx (69.2)(36\text{ ksi}) \approx 2491.2 \text{ in kip}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{Rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad (6.9.2.2-2)$$

$$\frac{481.4\text{ k}}{1060.4} + \frac{8}{9}\left(\frac{127.93\,(12)}{2491.2}\right) \leq 1.0$$

$$.454 + .548 = 1.002 > 1.0 \quad \text{no good}$$

for 50 ksi material

W10x88

$r_s = 2.63$

$$\lambda = \left(\frac{(2.1)(12.75)}{2.63\,(\pi)}\right)^2 \frac{50}{29000} \approx 0.0181$$

$$P_n = .66^{.0181}(50)(25.9) \approx 1285.3 \text{ k}$$

$$P_R = \phi P_N = .9(1285.3) \approx 1156.76$$

$$M_{Rx} = (53.1)(50\text{ ksi}) \approx 2655 \text{ in kip}$$

$$\frac{481.4}{1156.76} + \frac{8}{9}\left(\frac{127.93(12)}{2655}\right) \leq 1.0$$

$$.416 + .514 = .93 \leq 1.0 \quad \checkmark \text{ okay}$$

use W10x88, 50 ksi mat'l

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 436 of

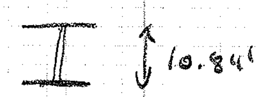
Assume Embed Depth = 1'-6"

$$f_1 = \frac{4Pd + 6M}{d^2}$$

$$f_1 = \frac{4(120.4)(18") + 6(127.93)12}{18^2}$$

$f_t \approx 55.18$ kin $\div$ 10.84 $\approx$ 5.09 ksi > 4.5 ksi

$f_{allow} = \phi .85 f'_c = (.9).85(4.5) \approx 37.32$ in kip



$$d = \frac{-(-4P) + \sqrt{4P^2 - 4ac}}{2a}$$

$$d = \frac{4(120.4) + \sqrt{[(4)120.4]^2 + 4(37.32)\,6(127.93)(12)}}{2(37.32)}$$

$\approx$ 23.44"

Use 2'-6"

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 437 of ____

Pile Embedment

INPUT

Reactions at top of column

V_u =	120.4	kips
e =	15.75	in
e V_u = M_u =	1896.3	kip-in

Pile Properties

size =	W 10 x 88	
w =	88	lbs
fy =	50	ksi
Zweak =	53.1	in^3
Sstrong =	98.5	in^3
D =	10.84	in
t_w =	0.605	in
B_f =	10.265	in
t_f =	0.99	in
Weak or strong axis?	Weak	
M_{cap} =	2655	kip-in
V_{cap} =	589	kip

Caisson Reinforcement

spiral size =	# 0	
pitch =	4	
f_s =	0	kip/in
f_c =	37.3167	kip/in
f_1 =	37.3167	kip/in

Miscellaneous Properties

l =	12	in
expn material =	0.75	in
concrete cover =	3	in
e =	15.75	in
f'c =	4.5	ksi

If f_1 is known

A =	37.3167	kip/in		
B =	-481.6	kips		
C =	-11377.8	kip-in		
d_1 =	25.0684	in		
d_2 =	-12.16265	in		
use d =	**25.0684**	**in (min.)**		
use d =	**2' - 6"**	(embed)		
total L =	**3' - 9 3/4"**			
W =	**336**	**lbs**		
f_1 =	37.3167	kips/in		
f_2 =	-27.71098	kips/in	C/D ratio =	1
x_1 =	25.0684	in		
x_2 =	3.703038	in		
use x =	3.703038	in		
Does V_{dowel} = 0:	yes			
max. M_{dowel} =	2108.246	kip-in	OK	will go plastic once
at x =	3.703038	in		
max. V_{dowel} =	120.4	kips	OK	
at x =	0	in		

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 5, Beam : 3, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.25	3.52	11.65	24.05	36.45	48.85	61.25
Self wt. :	M	0.0	248.7	384.7	1187.2	2176.7	2883.5	3307.5	3448.9
(Max)	V	112.6	108.5	106.2	91.2	68.4	45.6	22.8	0.0
DL-Prec. :	M	-0.0	81.2	125.5	387.4	710.3	941.0	1079.3	1125.5
DC(Max)	V	36.8	35.4	34.6	29.8	22.3	14.9	7.4	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	377.9	584.4	1803.7	3306.9	4380.7	5024.9	5239.7
Haunch (Max)	V	171.1	164.8	161.3	138.5	103.9	69.3	34.6	0.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	-278.7	-248.2	-231.5	-131.1	-1.8	98.8	170.8	214.0
DC(Max)	V	13.8	13.3	13.1	11.6	9.3	7.0	4.6	2.3
DL-Comp :	M	-1324.0	-1179.2	-1099.6	-622.7	-8.4	469.6	811.4	1016.8
DW(Max)	V	65.4	63.4	62.2	55.0	44.0	33.1	22.1	11.1
LL + I :	M+	233.8	218.3	216.5	322.9	878.6	1913.6	2956.0	3646.5
	V	2.4	2.5	2.6	3.3	67.8	83.2	54.4	23.3
LL + I :	M-	-5171.8	-4688.1	-4423.8	-2861.5	-930.9	-41.0	-35.2	-29.3
	V	241.1	234.4	230.7	206.7	136.7	0.5	0.5	0.5
LL + I :	Vmx	236.6	232.4	230.1	214.9	190.2	163.5	135.4	107.1
	M	-2377.3	-2182.4	-2068.4	-1263.8	205.6	1438.5	2323.0	2837.3
Total :	M+	0.0	0.0	0.0	2947.4	7062.4	10687.2	13349.9	14691.4
	V	0.0	0.0	0.0	329.4	315.7	252.9	146.0	36.7
Total :	M-	-6774.5	-5407.7	-4660.2	-237.0	0.0	0.0	0.0	0.0
	V	589.1	569.8	559.0	489.4	0.0	0.0	0.0	0.0
Total :	Vmx	636.2	617.8	607.5	541.1	438.2	333.3	227.0	120.5
	M	-3979.9	-2902.0	-2304.9	1360.7	6389.4	10212.0	12717.0	13882.2

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	73.65	86.05	98.45	110.85	118.98	120.25	122.50
Self wt. :	M	3307.5	2883.5	2176.7	1187.2	384.7	248.7	0.0
(Max)	V	22.8	45.6	68.4	91.2	106.2	108.5	112.6
DL-Prec. :	M	1079.3	941.0	710.3	387.4	125.5	81.2	-0.0
DC(Max)	V	7.4	14.9	22.3	29.8	34.6	35.4	36.8
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	5024.9	4380.7	3306.9	1803.7	584.4	377.9	0.0
Haunch (Max)	V	34.6	69.3	103.9	138.5	161.3	164.8	171.1
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	228.6	214.5	171.7	100.2	37.3	26.4	6.2
DC(Max)	V	0.0	2.3	4.6	6.9	8.4	8.7	9.1
DL-Comp :	M	1086.0	1018.9	815.6	475.9	177.4	125.5	29.7
DW(Max)	V	0.1	10.9	21.9	32.9	40.1	41.2	43.2
LL + I :	M+	3899.2	3745.4	3081.8	1856.0	696.3	490.3	107.5
	V	49.2	82.6	127.5	163.1	90.3	79.0	58.8
LL + I :	M-	-23.5	-17.6	-11.7	-5.9	-2.0	-1.4	-0.4
	V	0.5	0.5	0.5	0.5	35.6	41.0	50.7
LL + I :	Vmx	81.1	101.5	131.2	164.0	185.5	188.8	194.7

439

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
	M	2895.1	3328.4	2868.7	1792.3	697.5	499.3	128.7
Total :	M+	14625.6	13183.9	10262.9	5810.4	2005.6	1349.9	143.5
	V	114.2	225.6	348.6	462.4	440.9	437.5	431.6
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	146.1	244.5	352.4	463.3	536.0	547.4	567.5
	M	13621.5	12766.9	10049.9	5746.7	2006.8	1358.9	164.7

REACTIONS (kips), SERVICE I

Load Type		Left Support	Right Support
Self Wt.		112.6	112.6
Deck+Haunch		171.1	171.1
Diaphragm		0.0	0.0
DL-Prec.(DC)		36.8	36.8
DL-Prec.(DW)		0.0	0.0
DL-Comp.(DC)		160.1	72.8
DL-Comp.(DW)		760.7	345.7
Live	(Max)	286.5	96.1
Live	(Min)	-117.9	-0.2
Pedestrian	(Max)	-0.0	-0.0
Pedestrian	(Min)	-0.0	-0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

SHEAR AND MOMENT ENVELOPE : Span : 5, Beam : 3, SERVICE III
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.25	3.52	11.65	24.05	36.45	48.85	61.25
Self wt. :	M	0.0	248.7	384.7	1187.2	2176.7	2883.5	3307.5	3448.9
(Max)	V	112.6	108.5	106.2	91.2	68.4	45.6	22.8	0.0
DL-Prec. :	M	-0.0	81.2	125.5	387.4	710.3	941.0	1079.3	1125.5
DC(Max)	V	36.8	35.4	34.6	29.8	22.3	14.9	7.4	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	377.9	584.4	1803.7	3306.9	4380.7	5024.9	5239.7
Haunch (Max)	V	171.1	164.8	161.3	138.5	103.9	69.3	34.6	0.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	-278.7	-248.2	-231.5	-131.1	-1.8	98.8	170.8	214.0
DC(Max)	V	13.8	13.3	13.1	11.6	9.3	7.0	4.6	2.3
DL-Comp :	M	-1324.0	-1179.2	-1099.6	-622.7	-8.4	469.6	811.4	1016.8
DW(Max)	V	65.4	63.4	62.2	55.0	44.0	33.1	22.1	11.1
LL + I :	M+	187.0	174.7	173.2	258.3	702.9	1530.9	2364.8	2917.2
	V	1.9	2.0	2.1	2.6	54.2	66.5	43.5	18.6

440

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
LL + I :	M-	-4137.4	-3750.5	-3539.0	-2289.2	-744.7	-32.8	-28.1	-23.5
	V	192.9	187.6	184.6	165.4	109.4	0.4	0.4	0.4
LL + I :	Vmx	189.3	185.9	184.1	172.0	152.2	130.8	108.3	85.7
	M	-1901.8	-1745.9	-1654.8	-1011.1	164.5	1150.8	1858.4	2269.9
Total :	M+	0.0	0.0	0.0	2882.8	6886.6	10304.5	12758.7	13962.1
	V	0.0	0.0	0.0	328.8	302.2	236.3	135.1	32.0
Total :	M-	-5740.1	-4470.1	-3775.5	0.0	0.0	0.0	0.0	0.0
	V	540.8	522.9	512.8	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	588.9	571.3	561.5	498.1	400.1	300.6	199.9	99.1
	M	-3504.5	-2465.5	-1891.2	1613.5	6348.3	9924.3	12252.4	13314.8

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	73.65	86.05	98.45	110.85	118.98	120.25	122.50
Self wt. :	M	3307.5	2883.5	2176.7	1187.2	384.7	248.7	0.0
(Max)	V	22.8	45.6	68.4	91.2	106.2	108.5	112.6
DL-Prec. :	M	1079.3	941.0	710.3	387.4	125.5	81.2	-0.0
DC(Max)	V	7.4	14.9	22.3	29.8	34.6	35.4	36.8
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	5024.9	4380.7	3306.9	1803.7	584.4	377.9	0.0
Haunch (Max)	V	34.6	69.3	103.9	138.5	161.3	164.8	171.1
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	228.6	214.5	171.7	100.2	37.3	26.4	6.2
DC(Max)	V	0.0	2.3	4.6	6.9	8.4	8.7	9.1
DL-Comp :	M	1086.0	1018.9	815.6	475.9	177.4	125.5	29.7
DW(Max)	V	0.1	10.9	21.9	32.9	40.1	41.2	43.2
LL + I :	M+	3119.4	2996.3	2465.4	1484.8	557.0	392.2	86.0
	V	39.4	66.1	102.0	130.5	72.2	63.2	47.1
LL + I :	M-	-18.8	-14.1	-9.4	-4.7	-1.6	-1.1	-0.3
	V	0.4	0.4	0.4	0.4	28.5	32.8	40.6
LL + I :	Vmx	64.9	81.2	105.0	131.2	148.4	151.0	155.8
	M	2316.1	2662.7	2295.0	1433.8	558.0	399.4	103.0
Total :	M+	13845.8	12434.8	9646.6	5439.2	1866.3	1251.9	122.0
	V	104.4	209.1	323.1	429.8	422.8	421.7	419.8
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	129.9	224.2	326.1	430.5	499.0	509.6	528.6
	M	13042.5	12101.2	9476.1	5388.2	1867.3	1259.1	138.9

SHEAR AND MOMENT ENVELOPE : Span : 5, Beam : 3, STRENGTH I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.25	3.52	11.65	24.05	36.45	48.85	61.25
Self wt. :	M	0.0	310.9	480.8	1484.0	2720.9	3604.3	4134.4	4311.1
(Max)	V	140.8	135.6	132.7	114.0	85.5	57.0	28.5	0.0
Self wt. :	M	0.0	223.9	346.2	1068.5	1959.0	2595.1	2976.8	3104.0
(Min)	V	101.4	97.6	95.5	82.1	61.6	41.0	20.5	0.0
DL-Prec. :	M	-0.0	101.5	156.9	484.3	887.9	1176.2	1349.2	1406.8
DC(Max)	V	45.9	44.3	43.3	37.2	27.9	18.6	9.3	0.0

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
DL-Prec. :	M	-0.0	73.1	113.0	348.7	639.3	846.9	971.4	1012.9
DC(Min)	V	33.1	31.9	31.2	26.8	20.1	13.4	6.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	472.4	730.5	2254.6	4133.6	5475.8	6281.2	6549.6
Haunch (Max)	V	213.9	206.0	201.6	173.2	129.9	86.6	43.3	0.0
Deck + :	M	0.0	340.1	526.0	1623.3	2976.2	3942.6	4522.4	4715.7
Haunch (Min)	V	154.0	148.3	145.1	124.7	93.5	62.3	31.2	0.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	-348.3	-310.2	-289.3	-163.8	-2.2	123.6	213.5	267.5
DC(Max)	V	17.2	16.7	16.4	14.5	11.6	8.7	5.8	2.9
DL-Comp :	M	-250.8	-223.4	-208.3	-118.0	-1.6	89.0	153.7	192.6
DC(Min)	V	12.4	12.0	11.8	10.4	8.3	6.3	4.2	2.1
DL-Comp :	M	-1986.0	-1768.8	-1649.4	-934.0	-12.6	704.4	1217.0	1525.2
DW(Max)	V	98.0	95.0	93.4	82.6	66.1	49.6	33.1	16.6
DL-Comp :	M	-860.6	-766.5	-714.8	-404.8	-5.5	305.3	527.4	660.9
DW(Min)	V	42.5	41.2	40.5	35.8	28.6	21.5	14.3	7.2
LL + I :	M+	409.2	382.0	378.8	565.0	1537.6	3348.9	5172.9	6381.3
	V	4.1	4.5	4.6	5.7	118.6	145.6	95.2	40.8
LL + I :	M-	-9050.7	-8204.2	-7741.6	-5007.7	-1629.1	-71.8	-61.6	-51.3
	V	421.9	410.3	403.7	361.7	239.3	0.9	0.9	0.9
LL + I :	Vmx	414.1	406.8	402.6	376.1	332.9	286.1	236.9	187.5
	M	-4160.2	-3819.2	-3619.8	-2211.7	359.8	2517.3	4065.3	4965.3
Total :	M+	0.0	276.9	824.0	4265.2	9272.9	14433.2	18368.2	20441.7
	V	0.0	502.0	491.9	427.2	439.6	366.0	215.2	60.3
Total :	M-	-11385.0	-9646.3	-8695.2	-3065.1	0.0	0.0	0.0	0.0
	V	731.6	708.8	695.9	613.2	0.0	0.0	0.0	0.0
Total :	Vmx	929.9	904.3	890.0	797.6	653.9	506.6	356.9	207.0
	M	-6494.6	-5013.5	-4190.3	913.3	8087.4	13601.7	17260.6	19025.7

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	73.65	86.05	98.45	110.85	118.98	120.25	122.50
Self wt. :	M	4134.4	3604.3	2720.9	1484.0	480.8	310.9	0.0
(Max)	V	28.5	57.0	85.5	114.0	132.7	135.6	140.8
Self wt. :	M	2976.8	2595.1	1959.0	1068.5	346.2	223.9	0.0
(Min)	V	20.5	41.0	61.6	82.1	95.5	97.6	101.4
DL-Prec. :	M	1349.2	1176.2	887.9	484.3	156.9	101.5	-0.0
DC(Max)	V	9.3	18.6	27.9	37.2	43.3	44.2	45.9
DL-Prec. :	M	971.4	846.9	639.3	348.7	113.0	73.1	-0.0
DC(Min)	V	6.7	13.4	20.1	26.8	31.2	31.9	33.1
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	6281.2	5475.8	4133.7	2254.6	730.5	472.4	0.0
Haunch (Max)	V	43.3	86.6	129.9	173.2	201.6	206.0	213.9
Deck + :	M	4522.4	3942.6	2976.2	1623.3	526.0	340.1	0.0
Haunch (Min)	V	31.2	62.3	93.5	124.7	145.1	148.3	154.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0

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		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
DL-Comp :	M	285.7	268.1	214.6	125.2	46.7	33.0	7.8
DC(Max)	V	0.0	2.9	5.8	8.7	10.5	10.8	11.4
DL-Comp :	M	205.7	193.0	154.5	90.2	33.6	23.8	5.6
DC(Min)	V	0.0	2.1	4.1	6.2	7.6	7.8	8.2
DL-Comp :	M	1629.0	1528.4	1223.3	713.9	266.0	188.2	44.5
DW(Max)	V	0.1	16.4	32.8	49.3	60.1	61.8	64.8
DL-Comp :	M	705.9	662.3	530.1	309.3	115.3	81.6	19.3
DW(Min)	V	0.1	7.1	14.2	21.4	26.1	26.8	28.1
LL + I :	M+	6823.6	6554.4	5393.1	3248.0	1218.5	858.0	188.2
	V	86.2	144.6	223.1	285.4	158.0	138.2	103.0
LL + I :	M-	-41.0	-30.8	-20.5	-10.3	-3.5	-2.5	-0.6
	V	0.9	0.9	0.9	0.9	62.2	71.8	88.7
LL + I :	Vmx	141.9	177.7	229.7	287.0	324.5	330.4	340.8
	M	5066.4	5824.6	5020.2	3136.5	1220.6	873.7	225.3
Total :	M+	20503.1	18607.2	14573.5	8310.0	2899.4	1963.9	240.5
	V	167.4	326.0	505.0	667.7	606.3	596.7	579.7
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	223.2	359.1	511.6	669.3	772.8	788.9	817.5
	M	18745.9	17877.5	14200.6	8198.5	2901.6	1979.7	277.6

REACTIONS (kips), STRENGTH I

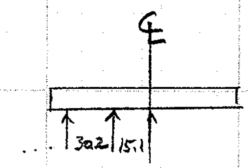
Load Type		Left Support	Right Support
Self Wt.		140.8	140.8
Deck+Haunch		213.9	213.9
Diaphragm		0.0	0.0
DL-Prec.(DC)		45.9	45.9
DL-Prec.(DW)		0.0	0.0
DL-Comp.(DC)		200.1	91.0
DL-Comp.(DW)		1141.0	518.6
Live	(Max)	501.4	168.1
Live	(Min)	-206.4	-0.4
Pedestrian	(Max)	-0.0	-0.0
Pedestrian	(Min)	-0.0	-0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

ECCENTRICITY CHECK

15 Caissons

$P = 96.1\ K/Lane$



$I \approx \Sigma n d^2$

$I = 2\left[15.1^2 + 30.2^2 + 45.3^2 + 60.4^2 + 75.5^2 + 90.6^2 + 105.7^2\right]$

$I \approx 63842.8\ ft^2$

$d_{max} \approx 107.8'$

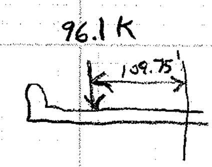
$$Q = P\left(\frac{1}{A} + e/s\right) \approx P\left(\frac{1}{A} + \frac{ed}{I}\right)$$

where $A = \#$ of Caissons $= 15$

$e =$ eccentricity of load (dist. from ℄)

$S =$ section modulus $= I/d$

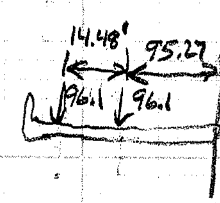
FOR 1 LANE:



$$Q = 1.2\left\{96.1\left(\frac{1}{15} + \frac{109.75\ (107.8)}{63842.8}\right)\right\}$$

$Q \approx 29.1\ K$

FOR 2 LANES



$$Q = 1.0\left\{24.2 + 96.1\left(\frac{1}{15} + \frac{95.27\ (107.8)}{63842.8}\right)\right\}$$

$Q = 46.1\ K$

FOR 3 LANES

$$Q = .85\left\{46.1 + 96.1\left(\frac{1}{15} + \frac{80.79\ (107.8)}{63842.8}\right)\right\}$$

$Q = 55.8\ K$

FOR 4 LANES

$$Q = .65 \left\{ 65.6 + 96.1\left(\frac{1}{15} + \frac{66.31\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 53.8 \text{ K}$$

FOR 5 LANES

$$Q = .65 \left\{ 82.8 + 96.1\left(\frac{1}{15} + \frac{51.83\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 63.5 \text{ K}$$

FOR 6 LANES

$$Q = .65 \left\{ 97.6 + 96.1\left(\frac{1}{15} + \frac{37.35\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 71.5 \text{ K}$$

FOR 7 LANES

$$Q = .65 \left\{ 110.1 + 96.1\left(\frac{1}{15} + \frac{22.87\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 78.1 \text{ K}$$

FOR 8 LANES

$$Q = .65 \left\{ 120.2 + 96.1\left(\frac{1}{15} + \frac{8.39\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 83.2 \text{ K}$$

FOR 9 LANES

$$Q = .65 \left\{ 128.0 + 96.1\left(\frac{1}{15} + \frac{-6.09\,(107.8)}{63842.8} \right) \right\}$$

$$Q = 86.7 \text{ K}$$

FOR 10 LANES

$$Q = .65 \left\{ 133.4 + 96.1 \left(\frac{1}{15} + \frac{-20.57\ (107.8)}{63842.8} \right) \right\}$$

$$Q = 88.7\ K$$

FOR 11 LANES

$$Q = .65 \left\{ 136.5 + 96.1 \left(\frac{1}{15} + \frac{-35.05\ (107.8)}{63842.8} \right) \right\}$$

$$Q = \underline{89.2\ K} \quad \leftarrow \text{CONTROLS}$$

FOR 12 LANES

$$Q = .65 \left\{ 137.2 + 96.1 \left(\frac{1}{15} + - \frac{49.53\ (107.8)}{63842.8} \right) \right\}$$

$$Q = 88.1\ K$$

MAX CAISSON LOAD

$LL = 89.2K$

$DL/CAISSON = 368.7\ K$

$P_u = 89.2\ (1.75) + 368.7\ K$

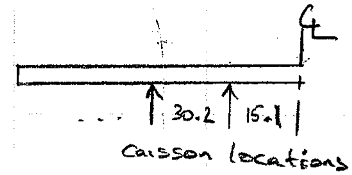
$P_u = \underline{524.8\ K}$ (STRENGTH 1)

$P_u = 289.4 + 89.2$

$P_u \approx \underline{378.6\ K}$ (SERVICE 1)

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

ECCENTRICITY CHECK (REVISED)



17 Caissons $\quad P = 96.1$ K/lane

$$I = \Sigma n d^2$$

$$I = 2\left[15.1^2 + 30.2^2 + 45.3^2 + 60.4^2 + 75.5^2 + 90.6^2 + 105.7^2 + 116.4^2\right]$$

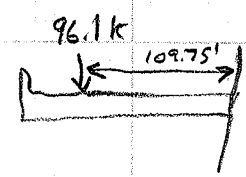
$$I \approx 90940.7 \text{ ft}^2$$

$$d_{max} \approx 117.65'$$

$$Q = P\left(\frac{1}{A} + e/s\right) \approx P\left(\frac{1}{A} + \frac{ed}{I}\right)$$

where A = # of caissons = 17
e = eccentricity of load (dist from ℄)
S = section modulus = I/d

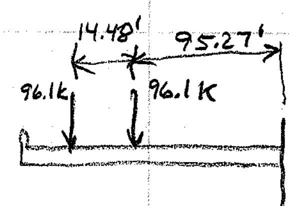
FOR 1 LANE:



$$Q = 1.2\left\{96.1\left(\frac{1}{17} + \frac{109.75\,(117.65)}{90940.7}\right)\right\}$$

$$Q \approx 23.2 \text{ K}$$

FOR 2 LANES:



$$Q = 1.0\left\{19.30 + 96.1\left(\frac{1}{17} + \frac{95.27\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 25.1 \text{ K}$$

FOR 3 LANES

$$Q = .85\left\{25.1 + 96.1\left(\frac{1}{17} + \frac{80.79\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 34.7 \text{ K}$$

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 447 of

FOR 4 LANES

$$Q = .65\left\{40.8 + 96.1\left(\frac{1}{17} + \frac{66.31\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 35.6\ \text{K}$$

FOR 5 LANES

$$Q = .65\left\{54.7 + 96.1\left(\frac{1}{17} + \frac{51.83\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 43.4\ \text{K}$$

FOR 6 LANES

$$Q = .65\left\{66.8 + 96.1\left(\frac{1}{17} + \frac{37.35\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 50.1\ \text{K}$$

FOR 7 LANES

$$Q = .65\left\{77.1 + 96.1\left(\frac{1}{17} + \frac{22.87\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 55.7\ \text{K}$$

FOR 8 LANES

$$Q = .65\left\{85.6 + 96.1\left(\frac{1}{17} + \frac{8.39\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 60.0\ \text{K}$$

FOR 9 LANES

$$Q = .65\left\{92.3 + 96.1\left(\frac{1}{17} + \frac{-6.09\,(117.65)}{90940.7}\right)\right\}$$

$$Q = 63.2\ \text{K}$$

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

FOR 10 LANES

$$Q = .65 \left\{ 97.2 + 96.1 \left(\frac{1}{17} + \frac{-20.57\,(117.65)}{90940.7} \right) \right\}$$

$$Q = 65.2\ K$$

FOR 11 LANES

$$Q = .65 \left\{ 100.3 + 96.1 \left(\frac{1}{17} + \frac{-35.05\,(117.65)}{90940.7} \right) \right\}$$

$Q = 66.1\ K$ ← controls

FOR 12 LANES

$$Q = .65 \left\{ 101.6 + 96.1 \left(\frac{1}{17} + \frac{-49.53\,(117.65)}{90940.7} \right) \right\}$$

$$Q = 65.7\ K$$

MAX CAISSON LOAD

$LL = 66.1\ K$

DL/caisson $\approx 325.3\ K$ (strength I)

$\approx 255.3\ K$ (Service I)

$P_u = 66.1(1.75) + 325.3\ K$

$P_u \approx 441.0\ K$ (STRENGTH I)

$P_u = 66.1 + 255.3$

$P_u = 321.4\ K$ (SERVICE I)

wt of 55' caisson

$\approx \frac{\pi\, 2.5^2}{4}(55').15 \approx 40.5\ K$

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Chk'd:	Date	Structure no.	Sheet 449 of

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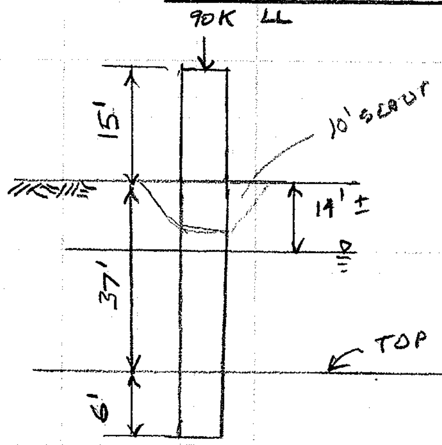
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

ABUTMENT CAISSONS



$\gamma_t = 120$ lb/ft³ $= .0694$ lb/in³
$\phi = 32°$
$c = 0$
$K_n = 64$ lb/in³

$\gamma_t = 135$ lb/ft³ $= .0781$ lb/in³
$\phi = 0$
$c = 10000$ lb/ft² $= 69.44$ lb/in²
$K_n = 2000$ lb/in³
$\varepsilon_{50} = .005$

LPILE INPUT

LINE 1: TITLE

LINE 2: UNITS 1 = ENGLISH

LINE 3: VARIABLES: NI, NDIAM, XGS, XLN

NI = # of Increments = 100

NDIAM = # of segments with different ϕ dia or E ≈ 2

XGS ≈ 15' = 180"

XLN = 58' ≈ 696" 840 = 18' embed

LINE 4: XDIAM(I), DIAM(I), MINERT(I), AREA(I), EPILE(I)

XDIAM = X coordinate = 0 TOP, 180" BOTTOM } COLUMN; 180" TOP, 696" BOTTOM } CAISSON

DIAM = DIAMETER OF PILE = 36" - COLUMN, 36" - CAISSON

MINERT = MOMENT OF INERTIA = $\frac{\pi d^4}{64}$ = 82448 = .824D+05

39760 for 30"

AREA = $\pi d^2/4$ ≈ 1017.9

707

EPILE = 3640 ksi

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Chk'd:	Date	Structure no.	Sheet 450 of

LINE 5: NL, NGI, NSTR, NPY

NL = # of soil layers = 2
NGI = # of points on plot = 8
NSTR = # of points in input = 8
NPY = # of input p-y curves = 0

LINE 6: KSOIL(I), XTOP(I), XBOT(I), XK(I)

KSOIL - PY curve code
4 - Sand
3 - stiff clay

XTOPI = 180" → 624" sand — 300" for scour condition
624 → 10000 stiff clay

XK = KN = 64 lb/in³ sand
Kn = 2000 lb/in³ clay

LINE 7: XGI(I), GAM(I)

XGI = location

GAM = unit weight = .0694 lb/in³ dry — .0723 lb/in³ wet
.0781 lb/in³ bedrock

LINE 8: XSTR(I), CI(I) PHI(I), EE50(I)

SOIL 1 XSTR = 180 - 624
C = 0
PHI = 32
EE50 = 0

SOIL 2 XSTR = 624 - 10000
C = 10000 lb/ft³ = 69.44 lb/in³
PHI = 0 EE50 = .005

LINE 12: NW = 5
KYCL = 1
RCYCL = 0

LINE 13: XW(I), WW(I) — 400 KIP LOAD. NONE ON ABUT

SOIL @ 180"
4' above 132"

0	0
120	0
120	16666.7 lb/in
144	16666.7 lb/in
144	0

$\}$ 400 Kip load distributed over 2'

LINE 14: NLDS = 1

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Chk'd:	Date	Structure no.	Sheet 451 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

LINE 15: KBC3 (I), BC1(I) BC2(I), Q(II)

Q = axial load = 90 K max 525 K total

BC1 = displacement = 2.2" worst case

BC2 = moment = 0 ft kips = 0 in kips

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 452 of

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CAISSON DESIGN ABUTMENT, Structure F-16-XP,

```
   1
 100     4   .180D+03   .696D+03
 .000D+00   .360D+02   .824D+05   .102D+04   .364D+07
 .180D+03   .360D+02   .824D+05   .102D+04   .364D+07
 .180D+03   .360D+02   .824D+05   .102D+04   .364D+07
 .696D+03   .360D+02   .824D+05   .102D+04   .364D+07
   2     8     8     0
   4   .180D+03   .624D+03   .640D+02
   3   .624D+03   .100D+04   .200D+04
 .100D+01   .000D+00
 .180D+03   .000D+00
 .180D+03   .666D-01
 .348D+03   .666D-01
 .348D+03   .694D-01
 .624D+03   .694D-01
 .624D+03   .780D-01
 .100D+04   .780D-01
 .100D+01   .000D+00   .000D+00   .000D+00
 .180D+03   .000D+00   .000D+00   .000D+00
 .180D+03   .000D+00   .320D+02   .000D+00
 .348D+03   .000D+00   .320D+02   .000D+00
 .348D+03   .000D+00   .320D+02   .000D+00
 .624D+03   .000D+00   .320D+02   .000D+00
 .624D+03   .694D+02   .000D+00   .500D-02
 .100D+04   .694D+02   .000D+00   .500D-02
    5    1   .000D+00
 .100D+01   .000D+00
 .120D+03   .000D+00
 .120D+03   .000D+05
 .144D+03   .000D+05
 .144D+03   .000D+00
   1
   4   .220D+01   .000D+07   .900D+5
   0
   1     1     0
 100   .100D-05   .360D+03
```


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```
*******************************************************
*    PROGRAM LPILE1                                   *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                  *
*    ALL RIGHTS RESERVED                              *
*   ----------------------------------------------    *
*                                                     *
*              PREPARED ESPECIALLY FOR                *
*                                                     *
*            STATE DEPARTMENT OF HIGHWAYS             *
*                                                     *
*              DENVER, COLORADO 80222                 *
*                                                     *
*                 LICENSE NO. 138                     *
*                                                     *
*******************************************************
```

PROGRAM LPILE1

CAISSON DESIGN ABUTMENT, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

36" column/caisson

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 696.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	36.000	.824D+05	.102D+04	.364D+07
180.00	36.000	.824D+05	.102D+04	.364D+07
180.00	36.000	.824D+05	.102D+04	.364D+07
696.00	36.000	.824D+05	.102D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 180.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER          =     180.00 IN
X AT THE BOTTOM OF THE LAYER       =     624.00 IN
MODULUS OF SUBGRADE REACTION       =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER          =     624.00 IN
X AT THE BOTTOM OF THE LAYER       =    1000.00 IN
MODULUS OF SUBGRADE REACTION       =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                   8 POINTS
            X,IN     WEIGHT,LBS/IN**3
            1.00        .00D+00
          180.00        .00D+00
          180.00        .67D-01
          348.00        .67D-01
          348.00        .69D-01
          624.00        .69D-01
          624.00        .78D-01
         1000.00        .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                   8 POINTS
            X,IN      C,LBS/IN**2      PHI,DEGREES      E50
            1.00      .000D+00        .000D+00        -----
          180.00      .000D+00        .000D+00        -----
          180.00      .000D+00        .320D+02        -----
          348.00      .000D+00        .320D+02        -----
          348.00      .000D+00        .320D+02        -----
          624.00      .000D+00        .320D+02        -----
          624.00      .694D+02            .000        .500D-02
         1000.00      .694D+02            .000        .500D-02


BOUNDARY AND LOADING CONDITIONS
------------------------------


DISTRIBUTED LOAD CURVE             5 POINTS
                          X,IN      LOAD,LBS/IN
                          1.00         .000D+00
                        120.00         .000D+00
                        120.00         .000D+00
                        144.00         .000D+00
                        144.00         .000D+00


LOADING NUMBER    1

BOUNDARY CONDITION CODE                  =          4
DEFLECTION AT THE PILE HEAD              =   .220D+01 IN
MOMENT AT THE PILE HEAD                  =   .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =   .900D+05 LBS


FINITE-DIFFERENCE PARAMETERS
   NUMBER OF PILE INCREMENTS                                  =        100
   DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =   .100D-05 IN
   MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =        100
```

MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
120.00	.000D+00
120.00	.000D+00
144.00	.000D+00
144.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .220D+01 IN
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .900D+05 LBS

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
.00	.220D+01	.000D+00	.472D+05	.000D+00	.882D+02	.300D+12
6.96	.213D+01	.334D+06	.472D+05	.000D+00	.161D+03	.300D+12
13.92	.207D+01	.668D+06	.472D+05	.000D+00	.234D+03	.300D+12
20.88	.201D+01	.100D+07	.472D+05	.000D+00	.307D+03	.300D+12
27.84	.194D+01	.134D+07	.472D+05	.000D+00	.380D+03	.300D+12
34.80	.188D+01	.167D+07	.472D+05	.000D+00	.453D+03	.300D+12
41.76	.181D+01	.200D+07	.472D+05	.000D+00	.526D+03	.300D+12
48.72	.175D+01	.234D+07	.472D+05	.000D+00	.599D+03	.300D+12
55.68	.168D+01	.267D+07	.472D+05	.000D+00	.672D+03	.300D+12
62.64	.162D+01	.301D+07	.472D+05	.000D+00	.745D+03	.300D+12
69.60	.156D+01	.334D+07	.472D+05	.000D+00	.818D+03	.300D+12
76.56	.150D+01	.367D+07	.472D+05	.000D+00	.891D+03	.300D+12
83.52	.143D+01	.401D+07	.472D+05	.000D+00	.964D+03	.300D+12
90.48	.137D+01	.434D+07	.472D+05	.000D+00	.104D+04	.300D+12
97.44	.131D+01	.468D+07	.472D+05	.000D+00	.111D+04	.300D+12
104.40	.125D+01	.501D+07	.472D+05	.000D+00	.118D+04	.300D+12
111.36	.120D+01	.534D+07	.472D+05	.000D+00	.126D+04	.300D+12
118.32	.114D+01	.568D+07	.472D+05	.000D+00	.133D+04	.300D+12
125.28	.108D+01	.601D+07	.472D+05	.000D+00	.140D+04	.300D+12
132.24	.103D+01	.634D+07	.472D+05	.000D+00	.147D+04	.300D+12
139.20	.971D+00	.668D+07	.472D+05	.000D+00	.155D+04	.300D+12
146.16	.917D+00	.701D+07	.472D+05	.000D+00	.162D+04	.300D+12
153.12	.864D+00	.734D+07	.472D+05	.000D+00	.169D+04	.300D+12
160.08	.813D+00	.767D+07	.472D+05	.000D+00	.176D+04	.300D+12

167.04	.763D+00	.801D+07	.472D+05	.000D+00	.184D+04	.300D+12
174.00	.714D+00	.834D+07	.472D+05	.000D+00	.191D+04	.300D+12
180.96	.666D+00	.867D+07	.471D+05	-.151D+02	.198D+04	.300D+12
187.92	.620D+00	.900D+07	.466D+05	-.132D+03	.206D+04	.300D+12
194.88	.576D+00	.933D+07	.452D+05	-.258D+03	.213D+04	.300D+12
201.84	.532D+00	.964D+07	.430D+05	-.388D+03	.219D+04	.300D+12
208.80	.491D+00	.994D+07	.398D+05	-.515D+03	.226D+04	.300D+12
215.76	.451D+00	.102D+08	.359D+05	-.634D+03	.232D+04	.300D+12
222.72	.413D+00	.104D+08	.311D+05	-.745D+03	.237D+04	.300D+12
229.68	.376D+00	.106D+08	.255D+05	-.840D+03	.241D+04	.300D+12
236.64	.341D+00	.108D+08	.194D+05	-.922D+03	.245D+04	.300D+12
243.60	.308D+00	.109D+08	.127D+05	-.992D+03	.247D+04	.300D+12
250.56	.276D+00	.110D+08	.567D+04	-.104D+04	.249D+04	.300D+12
257.52	.247D+00	.110D+08	-.170D+04	-.108D+04	.249D+04	.300D+12
264.48	.219D+00	.110D+08	-.927D+04	-.110D+04	.248D+04	.300D+12
271.44	.193D+00	.109D+08	-.169D+05	-.109D+04	.246D+04	.300D+12
278.40	.168D+00	.107D+08	-.244D+05	-.106D+04	.243D+04	.300D+12
285.36	.146D+00	.105D+08	-.315D+05	-.982D+03	.239D+04	.300D+12
292.32	.125D+00	.103D+08	-.380D+05	-.896D+03	.234D+04	.300D+12
299.28	.105D+00	.100D+08	-.440D+05	-.805D+03	.228D+04	.300D+12
306.24	.878D-01	.969D+07	-.492D+05	-.709D+03	.221D+04	.300D+12
313.20	.717D-01	.934D+07	-.538D+05	-.611D+03	.213D+04	.300D+12
320.16	.571D-01	.895D+07	-.577D+05	-.512D+03	.204D+04	.300D+12
327.12	.440D-01	.854D+07	-.610D+05	-.414D+03	.195D+04	.300D+12
334.08	.322D-01	.810D+07	-.635D+05	-.318D+03	.186D+04	.300D+12
341.04	.218D-01	.765D+07	-.654D+05	-.225D+03	.176D+04	.300D+12
348.00	.126D-01	.719D+07	-.666D+05	-.136D+03	.166D+04	.300D+12
354.96	.457D-02	.673D+07	-.673D+05	-.512D+02	.156D+04	.300D+12
361.92	-.238D-02	.626D+07	-.674D+05	.277D+02	.146D+04	.300D+12
368.88	-.832D-02	.579D+07	-.669D+05	.101D+03	.135D+04	.300D+12
375.84	-.133D-01	.533D+07	-.660D+05	.167D+03	.125D+04	.300D+12
382.80	-.175D-01	.487D+07	-.646D+05	.227D+03	.115D+04	.300D+12
389.76	-.208D-01	.443D+07	-.629D+05	.280D+03	.106D+04	.300D+12
396.72	-.235D-01	.400D+07	-.608D+05	.326D+03	.962D+03	.300D+12
403.68	-.255D-01	.358D+07	-.584D+05	.365D+03	.871D+03	.300D+12
410.64	-.269D-01	.319D+07	-.557D+05	.397D+03	.784D+03	.300D+12
417.60	-.278D-01	.281D+07	-.529D+05	.423D+03	.702D+03	.300D+12
424.56	-.282D-01	.245D+07	-.498D+05	.442D+03	.624D+03	.300D+12
431.52	-.283D-01	.211D+07	-.467D+05	.455D+03	.550D+03	.300D+12
438.48	-.280D-01	.180D+07	-.435D+05	.463D+03	.481D+03	.300D+12
445.44	-.274D-01	.151D+07	-.403D+05	.466D+03	.418D+03	.300D+12
452.40	-.266D-01	.124D+07	-.371D+05	.464D+03	.359D+03	.300D+12
459.36	-.256D-01	.992D+06	-.338D+05	.457D+03	.305D+03	.300D+12
466.32	-.244D-01	.768D+06	-.307D+05	.447D+03	.256D+03	.300D+12
473.28	-.231D-01	.565D+06	-.276D+05	.434D+03	.212D+03	.300D+12
480.24	-.217D-01	.383D+06	-.247D+05	.417D+03	.172D+03	.300D+12
487.20	-.202D-01	.221D+06	-.218D+05	.398D+03	.137D+03	.300D+12
494.16	-.188D-01	.786D+05	-.191D+05	.377D+03	.105D+03	.300D+12
501.12	-.172D-01	-.456D+05	-.166D+05	.354D+03	.982D+02	.300D+12
508.08	-.157D-01	-.153D+06	-.142D+05	.331D+03	.122D+03	.300D+12
515.04	-.143D-01	-.244D+06	-.120D+05	.306D+03	.141D+03	.300D+12
522.00	-.128D-01	-.320D+06	-.996D+04	.281D+03	.158D+03	.300D+12
528.96	-.114D-01	-.383D+06	-.809D+04	.256D+03	.172D+03	.300D+12
535.92	-.101D-01	-.433D+06	-.640D+04	.231D+03	.183D+03	.300D+12
542.88	-.887D-02	-.472D+06	-.488D+04	.206D+03	.191D+03	.300D+12
549.84	-.770D-02	-.501D+06	-.352D+04	.182D+03	.198D+03	.300D+12
556.80	-.660D-02	-.521D+06	-.234D+04	.159D+03	.202D+03	.300D+12
563.76	-.559D-02	-.534D+06	-.130D+04	.137D+03	.205D+03	.300D+12
570.72	-.467D-02	-.539D+06	-.419D+03	.117D+03	.206D+03	.300D+12
577.68	-.383D-02	-.540D+06	.326D+03	.975D+02	.206D+03	.300D+12
584.64	-.308D-02	-.535D+06	.943D+03	.797D+02	.205D+03	.300D+12
591.60	-.242D-02	-.527D+06	.144D+04	.636D+02	.203D+03	.300D+12
598.56	-.184D-02	-.515D+06	.183D+04	.492D+02	.201D+03	.300D+12

605.52	-.134D-02	-.501D+06	.213D+04	.365D+02	.198D+03	.300D+12
612.48	-.925D-03	-.485D+06	.235D+04	.256D+02	.194D+03	.300D+12
619.44	-.589D-03	-.469D+06	.250D+04	.166D+02	.191D+03	.300D+12
626.40	-.328D-03	-.451D+06	.558D+04	.868D+03	.187D+03	.300D+12
633.36	-.140D-03	-.391D+06	.990D+04	.375D+03	.174D+03	.300D+12
640.32	-.147D-04	-.313D+06	.113D+05	.399D+02	.157D+03	.300D+12
647.28	.598D-04	-.233D+06	.109D+05	-.165D+03	.139D+03	.300D+12
654.24	.966D-04	-.161D+06	.940D+04	-.270D+03	.123D+03	.300D+12
661.20	.107D-03	-.102D+06	.740D+04	-.304D+03	.111D+03	.300D+12
668.16	.102D-03	-.580D+05	.533D+04	-.291D+03	.101D+03	.300D+12
675.12	.866D-04	-.280D+05	.344D+04	-.251D+03	.943D+02	.300D+12
682.08	.669D-04	-.101D+05	.188D+04	-.197D+03	.904D+02	.300D+12
689.04	.457D-04	-.176D+04	.726D+03	-.136D+03	.886D+02	.300D+12
696.00	.242D-04	.000D+00	.000D+00	-.727D+02	.882D+02	.300D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.146D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.257D-06 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .220D+01 IN
MAXIMUM BENDING MOMENT = .110D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.674D+05 LBS
NO. OF ITERATIONS = 10
NO. OF ZERO DEFLECTION POINTS = 2

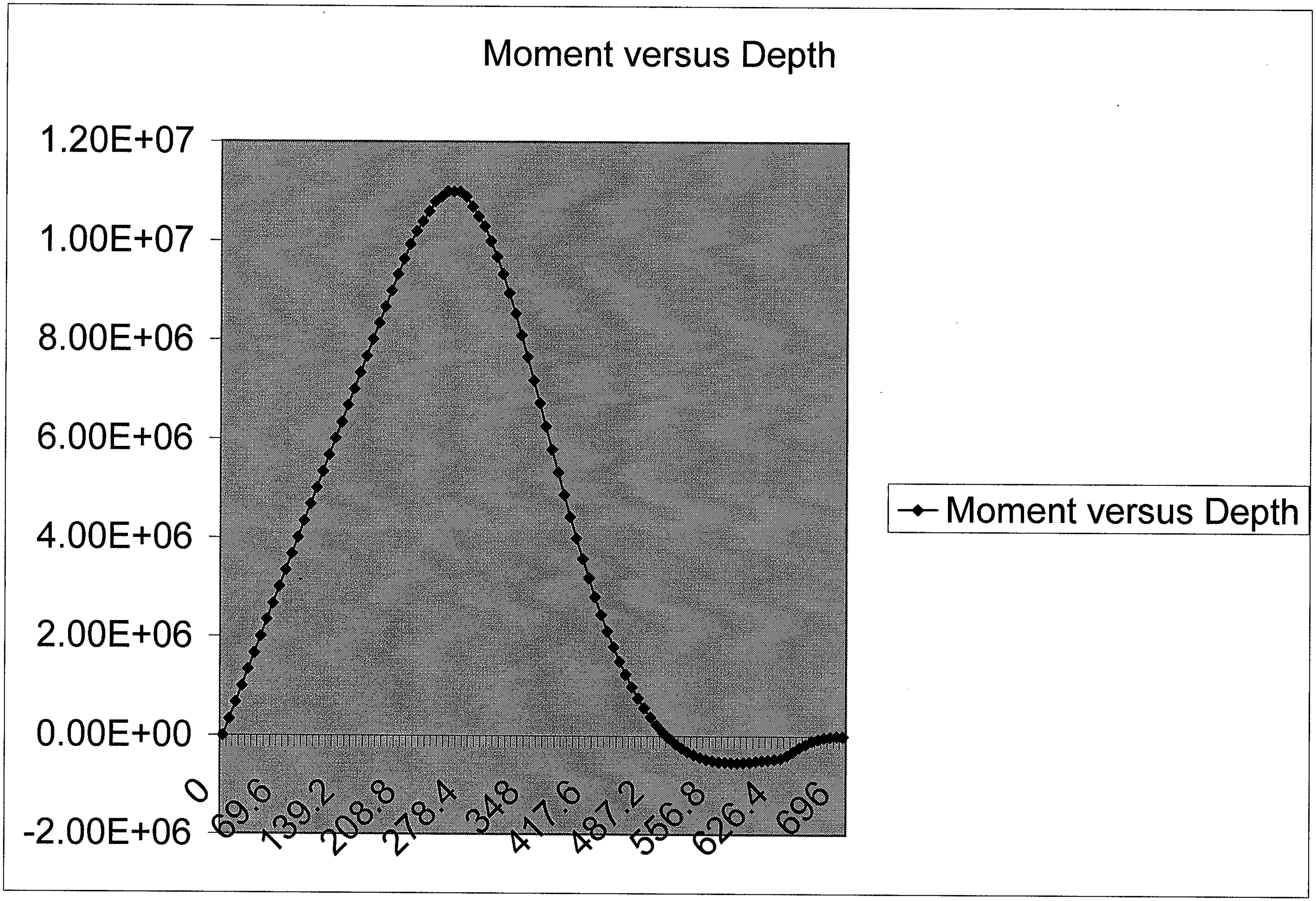
S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.2200D+01	.0000D+00	.9000D+05	.2200D+01	.1100D+08	-.6737D+05

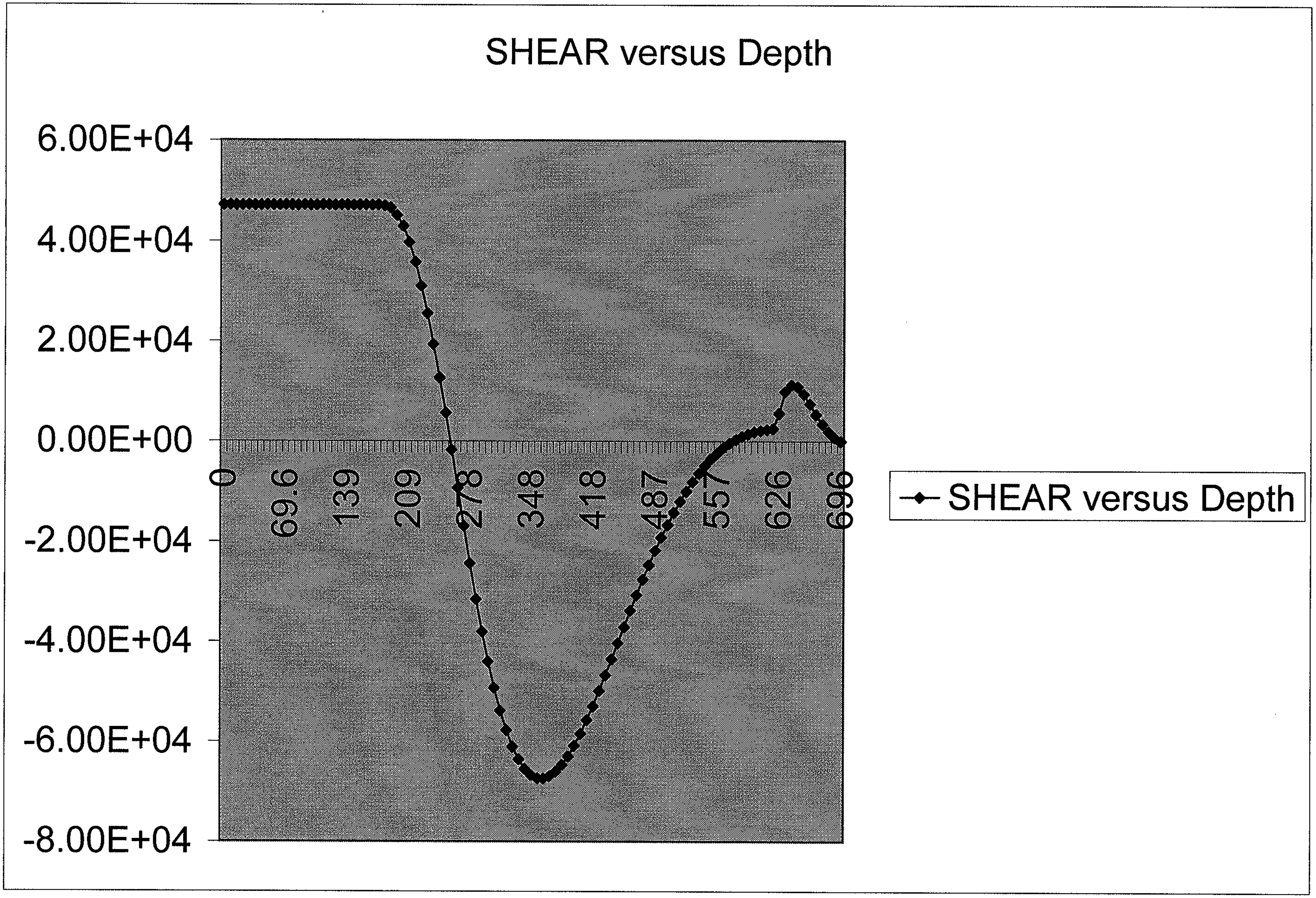
= 916.7 ft k

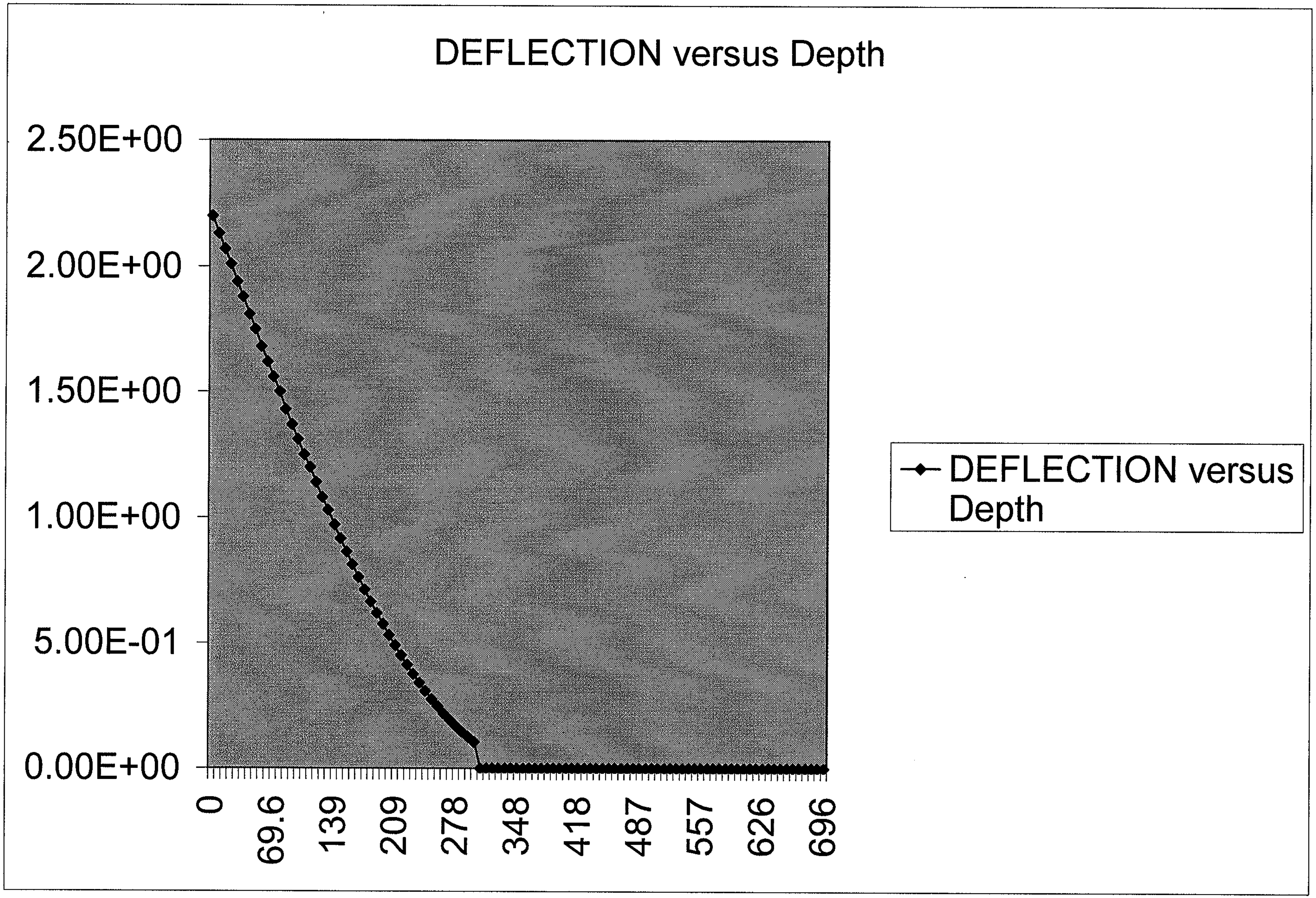
68 Kips

458


Moment versus Depth
1.20E+07
1.00E+07
8.00E+06
6.00E+06
4.00E+06
2.00E+06
0.00E+00
-2.00E+06
0
69.6
139.2
208.8
278.4
348
417.6
487.2
556.8
626.4
696
Moment versus Depth

459


SHEAR versus Depth
6.00E+04
4.00E+04
2.00E+04
0.00E+00
-2.00E+04
-4.00E+04
-6.00E+04
-8.00E+04
0
69.6
139
209
278
348
418
487
557
626
696
SHEAR versus Depth


DEFLECTION versus Depth
2.50E+00
2.00E+00
1.50E+00
1.00E+00
5.00E-01
0.00E+00
0
69.6
139
209
278
348
418
487
557
626
696
DEFLECTION versus Depth

```
*******************************************************
*    PROGRAM LPILE1                                   *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                  *
*    ALL RIGHTS RESERVED                              *
*   -----------------------------------------------   *
*                                                     *
*              PREPARED ESPECIALLY FOR                *
*                                                     *
*           STATE DEPARTMENT OF HIGHWAYS              *
*                                                     *
*              DENVER, COLORADO 80222                 *
*                                                     *
*                 LICENSE NO. 138                     *
*                                                     *
*******************************************************
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PROGRAM LPILE1

CAISSON DESIGN ABUTMENT, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

30" column/caisson

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 696.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	30.000	.397D+05	.707D+03	.364D+07
180.00	30.000	.397D+05	.707D+03	.364D+07
180.00	30.000	.397D+05	.707D+03	.364D+07
696.00	30.000	.397D+05	.707D+03	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 180.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER          =     180.00 IN
X AT THE BOTTOM OF THE LAYER       =     624.00 IN
MODULUS OF SUBGRADE REACTION       =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER          =     624.00 IN
X AT THE BOTTOM OF THE LAYER       =    1000.00 IN
MODULUS OF SUBGRADE REACTION       =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                  8 POINTS
          X,IN      WEIGHT,LBS/IN**3
          1.00         .00D+00
        180.00         .00D+00
        180.00         .67D-01
        348.00         .67D-01
        348.00         .69D-01
        624.00         .69D-01
        624.00         .78D-01
       1000.00         .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                  8 POINTS
          X,IN       C,LBS/IN**2      PHI,DEGREES     E50
          1.00       .000D+00        .000D+00        -----
        180.00       .000D+00        .000D+00        -----
        180.00       .000D+00        .320D+02        -----
        348.00       .000D+00        .320D+02        -----
        348.00       .000D+00        .320D+02        -----
        624.00       .000D+00        .320D+02        -----
        624.00       .694D+02            .000        .500D-02
       1000.00       .694D+02            .000        .500D-02


BOUNDARY AND LOADING CONDITIONS
------------------------------


DISTRIBUTED LOAD CURVE              5 POINTS
                           X,IN      LOAD,LBS/IN
                           1.00        .000D+00
                         120.00        .000D+00
                         120.00        .000D+00
                         144.00        .000D+00
                         144.00        .000D+00


LOADING NUMBER    1

BOUNDARY CONDITION CODE                  =          4
DEFLECTION AT THE PILE HEAD              =    .220D+01 IN
MOMENT AT THE PILE HEAD                  =    .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =    .900D+05 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                 =        100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE          =   .100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =        100
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MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
120.00	.000D+00
120.00	.000D+00
144.00	.000D+00
144.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .220D+01 IN
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .900D+05 LBS ← LIVE LOAD ONLY

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
.00	.220D+01	.000D+00	.275D+05	.000D+00	.127D+03	.145D+12
6.96	.213D+01	.197D+06	.275D+05	.000D+00	.202D+03	.145D+12
13.92	.206D+01	.395D+06	.275D+05	.000D+00	.276D+03	.145D+12
20.88	.199D+01	.592D+06	.275D+05	.000D+00	.351D+03	.145D+12
27.84	.192D+01	.789D+06	.275D+05	.000D+00	.426D+03	.145D+12
34.80	.185D+01	.987D+06	.275D+05	.000D+00	.500D+03	.145D+12
41.76	.178D+01	.118D+07	.275D+05	.000D+00	.575D+03	.145D+12
48.72	.171D+01	.138D+07	.275D+05	.000D+00	.649D+03	.145D+12
55.68	.165D+01	.158D+07	.275D+05	.000D+00	.724D+03	.145D+12
62.64	.158D+01	.178D+07	.275D+05	.000D+00	.798D+03	.145D+12
69.60	.151D+01	.197D+07	.275D+05	.000D+00	.873D+03	.145D+12
76.56	.145D+01	.217D+07	.275D+05	.000D+00	.947D+03	.145D+12
83.52	.138D+01	.237D+07	.275D+05	.000D+00	.102D+04	.145D+12
90.48	.132D+01	.256D+07	.275D+05	.000D+00	.110D+04	.145D+12
97.44	.125D+01	.276D+07	.275D+05	.000D+00	.117D+04	.145D+12
104.40	.119D+01	.296D+07	.275D+05	.000D+00	.124D+04	.145D+12
111.36	.113D+01	.315D+07	.275D+05	.000D+00	.132D+04	.145D+12
118.32	.107D+01	.335D+07	.275D+05	.000D+00	.139D+04	.145D+12
125.28	.101D+01	.355D+07	.275D+05	.000D+00	.147D+04	.145D+12
132.24	.948D+00	.374D+07	.275D+05	.000D+00	.154D+04	.145D+12
139.20	.891D+00	.394D+07	.275D+05	.000D+00	.162D+04	.145D+12
146.16	.835D+00	.414D+07	.275D+05	.000D+00	.169D+04	.145D+12
153.12	.781D+00	.433D+07	.275D+05	.000D+00	.176D+04	.145D+12
160.08	.727D+00	.453D+07	.275D+05	.000D+00	.184D+04	.145D+12

167.04	.676D+00	.472D+07	.275D+05	.000D+00	.191D+04	.145D+12
174.00	.626D+00	.492D+07	.275D+05	.000D+00	.199D+04	.145D+12
180.96	.577D+00	.511D+07	.274D+05	-.128D+02	.206D+04	.145D+12
187.92	.531D+00	.531D+07	.270D+05	-.113D+03	.213D+04	.145D+12
194.88	.486D+00	.550D+07	.258D+05	-.220D+03	.220D+04	.145D+12
201.84	.443D+00	.568D+07	.239D+05	-.330D+03	.227D+04	.145D+12
208.80	.402D+00	.584D+07	.212D+05	-.433D+03	.233D+04	.145D+12
215.76	.362D+00	.598D+07	.179D+05	-.528D+03	.239D+04	.145D+12
222.72	.325D+00	.609D+07	.139D+05	-.608D+03	.243D+04	.145D+12
229.68	.290D+00	.618D+07	.948D+04	-.676D+03	.246D+04	.145D+12
236.64	.257D+00	.623D+07	.459D+04	-.728D+03	.248D+04	.145D+12
243.60	.226D+00	.625D+07	-.604D+03	-.764D+03	.249D+04	.145D+12
250.56	.197D+00	.623D+07	-.599D+04	-.784D+03	.248D+04	.145D+12
257.52	.170D+00	.617D+07	-.114D+05	-.783D+03	.246D+04	.145D+12
264.48	.146D+00	.607D+07	-.168D+05	-.764D+03	.242D+04	.145D+12
271.44	.123D+00	.594D+07	-.220D+05	-.720D+03	.237D+04	.145D+12
278.40	.102D+00	.577D+07	-.267D+05	-.645D+03	.231D+04	.145D+12
285.36	.836D-01	.557D+07	-.309D+05	-.564D+03	.223D+04	.145D+12
292.32	.667D-01	.534D+07	-.346D+05	-.479D+03	.215D+04	.145D+12
299.28	.516D-01	.509D+07	-.376D+05	-.394D+03	.205D+04	.145D+12
306.24	.382D-01	.482D+07	-.401D+05	-.309D+03	.195D+04	.145D+12
313.20	.264D-01	.454D+07	-.419D+05	-.225D+03	.184D+04	.145D+12
320.16	.162D-01	.424D+07	-.432D+05	-.145D+03	.173D+04	.145D+12
327.12	.733D-02	.394D+07	-.440D+05	-.690D+02	.162D+04	.145D+12
334.08	-.186D-03	.363D+07	-.442D+05	.183D+01	.150D+04	.145D+12
341.04	-.649D-02	.332D+07	-.439D+05	.668D+02	.138D+04	.145D+12
348.00	-.117D-01	.302D+07	-.433D+05	.125D+03	.127D+04	.145D+12
354.96	-.158D-01	.272D+07	-.422D+05	.177D+03	.116D+04	.145D+12
361.92	-.191D-01	.243D+07	-.408D+05	.222D+03	.105D+04	.145D+12
368.88	-.215D-01	.215D+07	-.392D+05	.260D+03	.941D+03	.145D+12
375.84	-.233D-01	.189D+07	-.372D+05	.292D+03	.841D+03	.145D+12
382.80	-.244D-01	.164D+07	-.351D+05	.316D+03	.746D+03	.145D+12
389.76	-.249D-01	.140D+07	-.329D+05	.334D+03	.656D+03	.145D+12
396.72	-.250D-01	.118D+07	-.305D+05	.346D+03	.573D+03	.145D+12
403.68	-.246D-01	.976D+06	-.281D+05	.353D+03	.496D+03	.145D+12
410.64	-.240D-01	.789D+06	-.256D+05	.354D+03	.425D+03	.145D+12
417.60	-.231D-01	.619D+06	-.231D+05	.351D+03	.361D+03	.145D+12
424.56	-.220D-01	.467D+06	-.207D+05	.344D+03	.304D+03	.145D+12
431.52	-.207D-01	.331D+06	-.184D+05	.333D+03	.252D+03	.145D+12
438.48	-.193D-01	.211D+06	-.161D+05	.319D+03	.207D+03	.145D+12
445.44	-.178D-01	.106D+06	-.139D+05	.303D+03	.167D+03	.145D+12
452.40	-.163D-01	.164D+05	-.119D+05	.285D+03	.134D+03	.145D+12
459.36	-.148D-01	-.596D+05	-.998D+04	.265D+03	.150D+03	.145D+12
466.32	-.133D-01	-.123D+06	-.821D+04	.244D+03	.174D+03	.145D+12
473.28	-.119D-01	-.174D+06	-.658D+04	.223D+03	.193D+03	.145D+12
480.24	-.105D-01	-.215D+06	-.510D+04	.202D+03	.208D+03	.145D+12
487.20	-.920D-02	-.245D+06	-.377D+04	.181D+03	.220D+03	.145D+12
494.16	-.797D-02	-.267D+06	-.258D+04	.160D+03	.228D+03	.145D+12
501.12	-.683D-02	-.282D+06	-.154D+04	.140D+03	.234D+03	.145D+12
508.08	-.579D-02	-.289D+06	-.625D+03	.121D+03	.236D+03	.145D+12
515.04	-.484D-02	-.290D+06	.159D+03	.104D+03	.237D+03	.145D+12
522.00	-.399D-02	-.287D+06	.824D+03	.873D+02	.236D+03	.145D+12
528.96	-.323D-02	-.279D+06	.138D+04	.722D+02	.233D+03	.145D+12
535.92	-.257D-02	-.268D+06	.183D+04	.585D+02	.228D+03	.145D+12
542.88	-.200D-02	-.254D+06	.220D+04	.464D+02	.223D+03	.145D+12
549.84	-.151D-02	-.237D+06	.248D+04	.358D+02	.217D+03	.145D+12
556.80	-.111D-02	-.219D+06	.270D+04	.267D+02	.210D+03	.145D+12
563.76	-.772D-03	-.200D+06	.286D+04	.190D+02	.203D+03	.145D+12
570.72	-.505D-03	-.179D+06	.297D+04	.126D+02	.195D+03	.145D+12
577.68	-.299D-03	-.158D+06	.304D+04	.761D+01	.187D+03	.145D+12
584.64	-.146D-03	-.137D+06	.308D+04	.378D+01	.179D+03	.145D+12
591.60	-.386D-04	-.116D+06	.310D+04	.102D+01	.171D+03	.145D+12
598.56	.300D-04	-.940D+05	.310D+04	-.804D+00	.163D+03	.145D+12

605.52	.671D-04	-.724D+05	.309D+04	-.183D+01	.155D+03	.145D+12
612.48	.799D-04	-.510D+05	.308D+04	-.221D+01	.147D+03	.145D+12
619.44	.756D-04	-.296D+05	.306D+04	-.213D+01	.138D+03	.145D+12
626.40	.614D-04	-.838D+04	.240D+04	-.186D+03	.130D+03	.145D+12
633.36	.443D-04	.385D+04	.128D+04	-.136D+03	.129D+03	.145D+12
640.32	.286D-04	.946D+04	.496D+03	-.893D+02	.131D+03	.145D+12
647.28	.160D-04	.108D+05	.112D+02	-.501D+02	.131D+03	.145D+12
654.24	.708D-05	.962D+04	-.240D+03	-.221D+02	.131D+03	.145D+12
661.20	.134D-05	.742D+04	-.331D+03	-.418D+01	.130D+03	.145D+12
668.16	-.191D-05	.501D+04	-.325D+03	.598D+01	.129D+03	.145D+12
675.12	-.349D-05	.289D+04	-.266D+03	.109D+02	.128D+03	.145D+12
682.08	-.409D-05	.130D+04	-.184D+03	.128D+02	.128D+03	.145D+12
689.04	-.426D-05	.327D+03	-.933D+02	.133D+02	.127D+03	.145D+12
696.00	-.432D-05	.000D+00	.000D+00	.135D+02	.127D+03	.145D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .482D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .749D-07 LBS

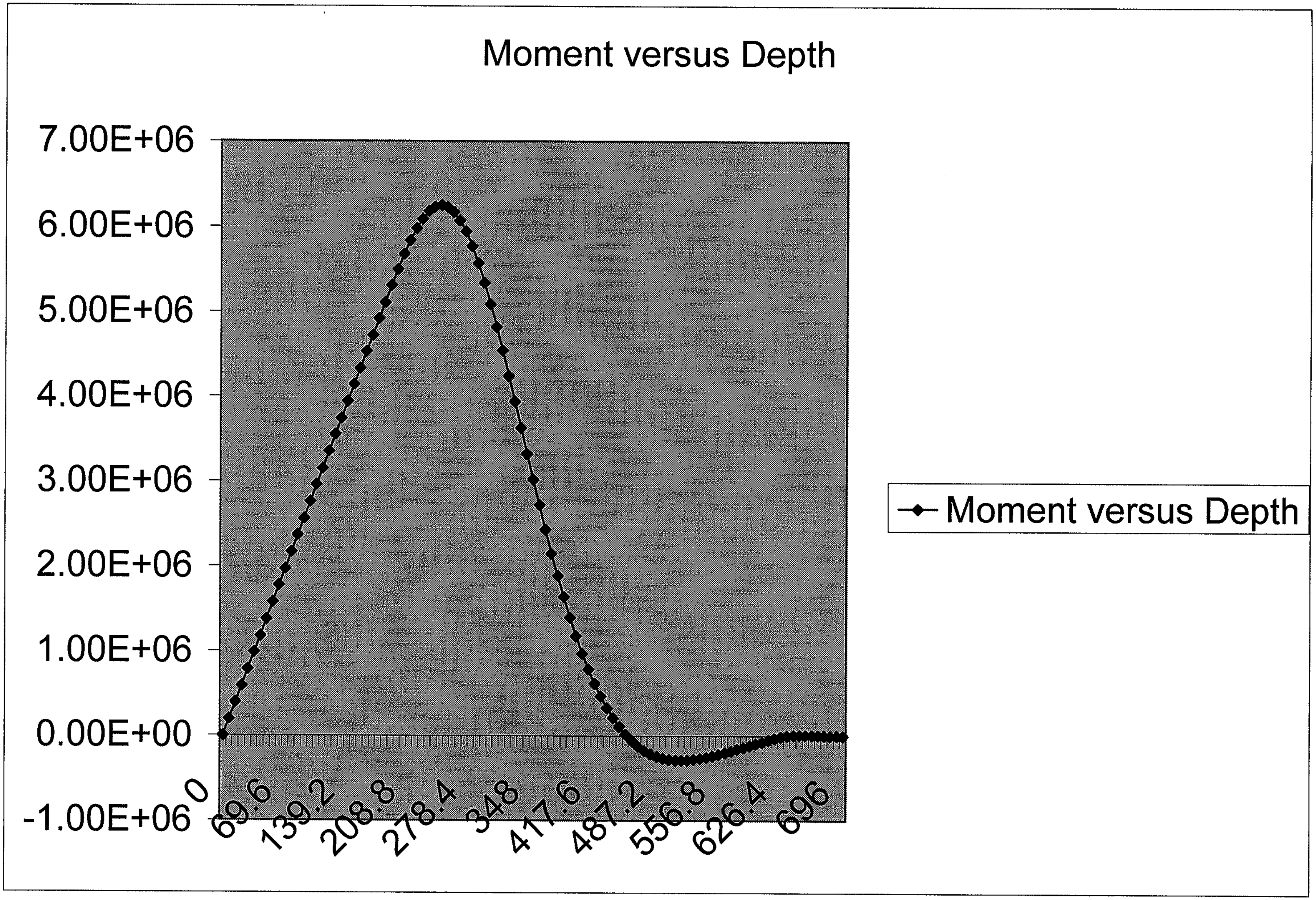
OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .220D+01 IN
MAXIMUM BENDING MOMENT = .625D+07 LBS-IN
MAXIMUM SHEAR FORCE = -.442D+05 LBS
NO. OF ITERATIONS = 10
NO. OF ZERO DEFLECTION POINTS = 3

S U M M A R Y   T A B L E
****

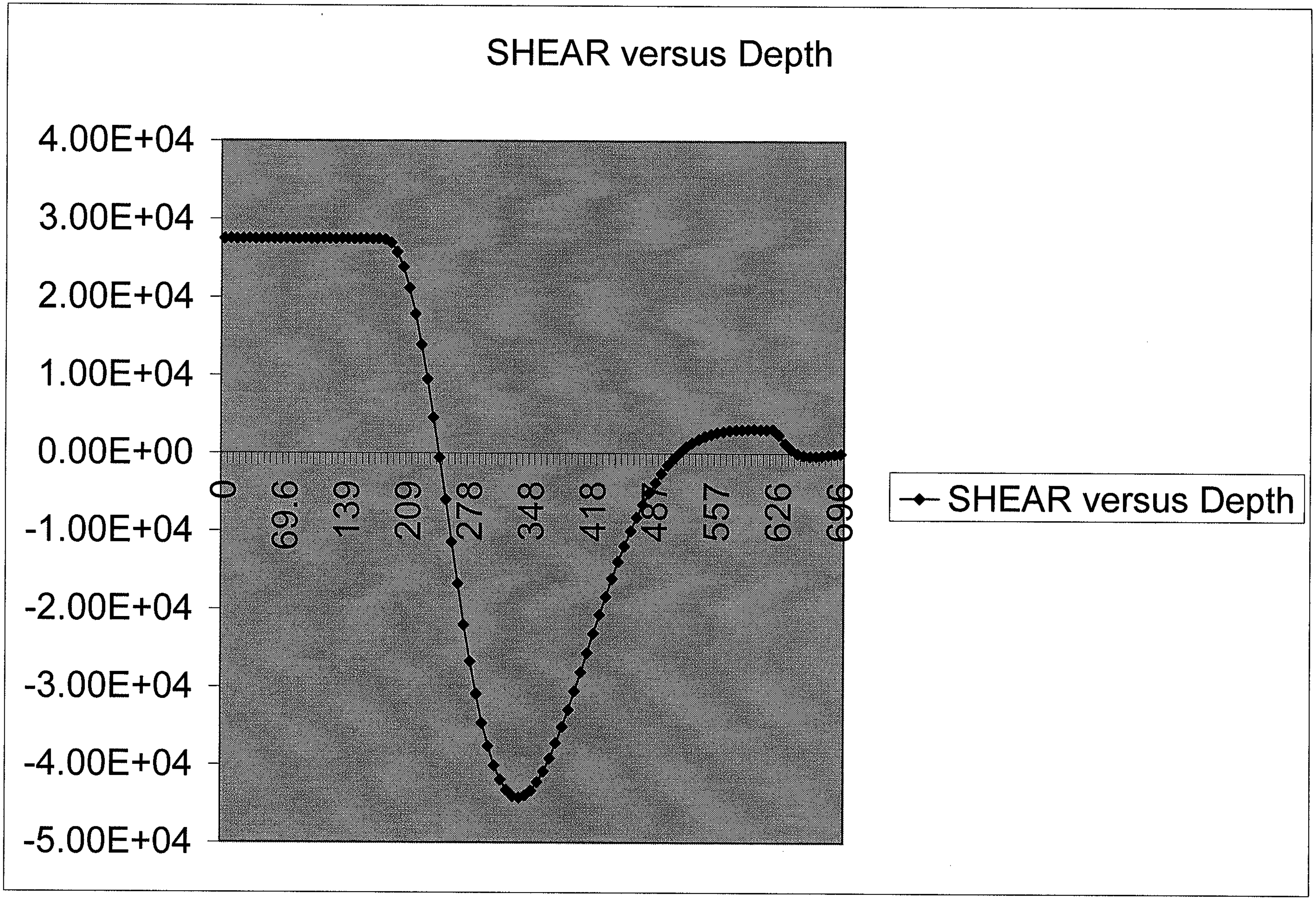
BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.2200D+01	.0000D+00	.9000D+05	.2200D+01	.6248D+07	-.4419D+05

466

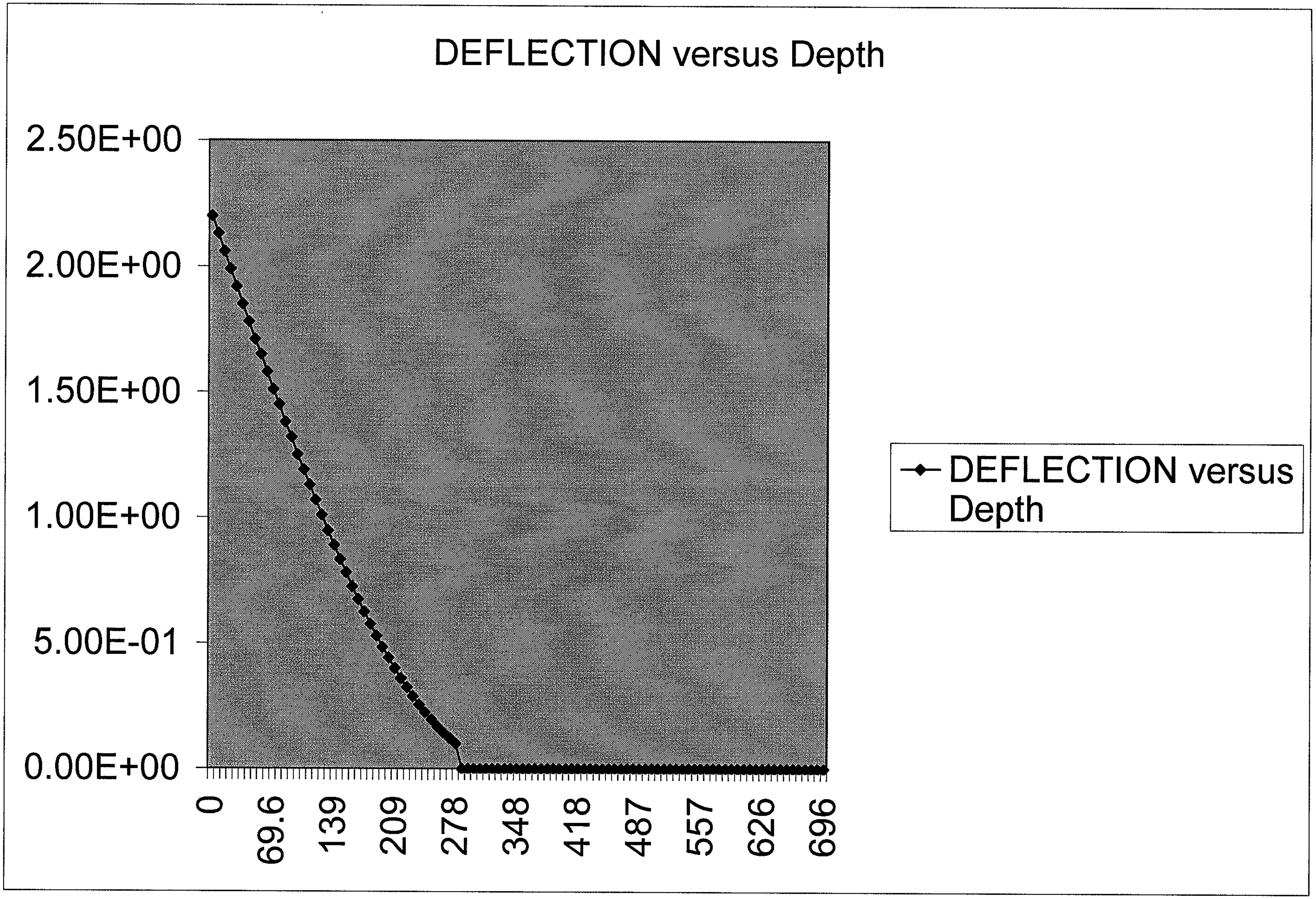


Moment versus Depth
7.00E+06
6.00E+06
5.00E+06
4.00E+06
3.00E+06
2.00E+06
1.00E+06
0.00E+00
-1.00E+06
0
69.6
139.2
208.8
278.4
348
417.6
487.2
556.8
626.4
696
Moment versus Depth

467



SHEAR versus Depth
4.00E+04
3.00E+04
2.00E+04
1.00E+04
0.00E+00
-1.00E+04
-2.00E+04
-3.00E+04
-4.00E+04
-5.00E+04
0
69.6
139
209
278
348
418
487
557
626
696
SHEAR versus Depth


DEFLECTION versus Depth
2.50E+00
2.00E+00
1.50E+00
1.00E+00
5.00E-01
0.00E+00
0
69.6
139
209
278
348
418
487
557
626
696
DEFLECTION versus Depth

```
*******************************************************
*    PROGRAM LPILE1                                   *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                  *
*    ALL RIGHTS RESERVED                              *
*   -----------------------------------------------   *
*                                                     *
*              PREPARED ESPECIALLY FOR                *
*                                                     *
*           STATE DEPARTMENT OF HIGHWAYS              *
*                                                     *
*              DENVER, COLORADO 80222                 *
*                                                     *
*                 LICENSE NO. 138                     *
*                                                     *
*******************************************************
```

PROGRAM LPILE1

CAISSON DESIGN ABUTMENT, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 696.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	30.000	.397D+05	.707D+03	.364D+07
180.00	30.000	.397D+05	.707D+03	.364D+07
180.00	30.000	.397D+05	.707D+03	.364D+07
696.00	30.000	.397D+05	.707D+03	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 180.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER            =     180.00 IN
X AT THE BOTTOM OF THE LAYER         =     624.00 IN
MODULUS OF SUBGRADE REACTION         =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER            =     624.00 IN
X AT THE BOTTOM OF THE LAYER         =    1000.00 IN
MODULUS OF SUBGRADE REACTION         =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                   8 POINTS
            X,IN     WEIGHT,LBS/IN**3
            1.00         .00D+00
          180.00         .00D+00
          180.00         .67D-01
          348.00         .67D-01
          348.00         .69D-01
          624.00         .69D-01
          624.00         .78D-01
         1000.00         .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                   8 POINTS
            X,IN       C,LBS/IN**2      PHI,DEGREES      E50
            1.00        .000D+00        .000D+00        -----
          180.00        .000D+00        .000D+00        -----
          180.00        .000D+00        .320D+02        -----
          348.00        .000D+00        .320D+02        -----
          348.00        .000D+00        .320D+02        -----
          624.00        .000D+00        .320D+02        -----
          624.00        .694D+02            .000        .500D-02
         1000.00        .694D+02            .000        .500D-02


BOUNDARY AND LOADING CONDITIONS
------------------------------


DISTRIBUTED LOAD CURVE              5 POINTS
                            X,IN     LOAD,LBS/IN
                            1.00        .000D+00
                          120.00        .000D+00
                          120.00        .000D+00
                          144.00        .000D+00
                          144.00        .000D+00


LOADING NUMBER    1

BOUNDARY CONDITION CODE                 =          4
DEFLECTION AT THE PILE HEAD             =   .220D+01 IN
MOMENT AT THE PILE HEAD                 =   .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD             =   .525D+06 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                  =        100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =   .100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =        100
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MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
120.00	.000D+00
120.00	.000D+00
144.00	.000D+00
144.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .220D+01 IN
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .525D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.220D+01	.000D+00	.241D+05	.000D+00	.743D+03	.145D+12
6.96	.213D+01	.205D+06	.241D+05	.000D+00	.820D+03	.145D+12
13.92	.206D+01	.409D+06	.241D+05	.000D+00	.897D+03	.145D+12
20.88	.199D+01	.614D+06	.241D+05	.000D+00	.975D+03	.145D+12
27.84	.192D+01	.819D+06	.241D+05	.000D+00	.105D+04	.145D+12
34.80	.185D+01	.102D+07	.241D+05	.000D+00	.113D+04	.145D+12
41.76	.178D+01	.123D+07	.241D+05	.000D+00	.121D+04	.145D+12
48.72	.171D+01	.143D+07	.241D+05	.000D+00	.128D+04	.145D+12
55.68	.164D+01	.163D+07	.241D+05	.000D+00	.136D+04	.145D+12
62.64	.157D+01	.184D+07	.241D+05	.000D+00	.144D+04	.145D+12
69.60	.151D+01	.204D+07	.241D+05	.000D+00	.151D+04	.145D+12
76.56	.144D+01	.224D+07	.241D+05	.000D+00	.159D+04	.145D+12
83.52	.137D+01	.245D+07	.241D+05	.000D+00	.167D+04	.145D+12
90.48	.131D+01	.265D+07	.241D+05	.000D+00	.174D+04	.145D+12
97.44	.125D+01	.285D+07	.241D+05	.000D+00	.182D+04	.145D+12
104.40	.118D+01	.305D+07	.241D+05	.000D+00	.190D+04	.145D+12
111.36	.112D+01	.325D+07	.241D+05	.000D+00	.197D+04	.145D+12
118.32	.106D+01	.345D+07	.241D+05	.000D+00	.205D+04	.145D+12
125.28	.999D+00	.365D+07	.241D+05	.000D+00	.212D+04	.145D+12
132.24	.940D+00	.385D+07	.241D+05	.000D+00	.220D+04	.145D+12
139.20	.882D+00	.405D+07	.241D+05	.000D+00	.227D+04	.145D+12
146.16	.826D+00	.424D+07	.241D+05	.000D+00	.235D+04	.145D+12
153.12	.772D+00	.444D+07	.241D+05	.000D+00	.242D+04	.145D+12
160.08	.718D+00	.464D+07	.241D+05	.000D+00	.249D+04	.145D+12

167.04	.667D+00	.483D+07	.241D+05	.000D+00	.257D+04	.145D+12
174.00	.617D+00	.503D+07	.241D+05	.000D+00	.264D+04	.145D+12
180.96	.569D+00	.522D+07	.241D+05	-.127D+02	.271D+04	.145D+12
187.92	.522D+00	.541D+07	.236D+05	-.112D+03	.279D+04	.145D+12
194.88	.477D+00	.560D+07	.225D+05	-.219D+03	.286D+04	.145D+12
201.84	.434D+00	.577D+07	.206D+05	-.328D+03	.292D+04	.145D+12
208.80	.393D+00	.593D+07	.179D+05	-.431D+03	.298D+04	.145D+12
215.76	.354D+00	.606D+07	.146D+05	-.524D+03	.303D+04	.145D+12
222.72	.317D+00	.617D+07	.107D+05	-.603D+03	.307D+04	.145D+12
229.68	.283D+00	.625D+07	.625D+04	-.670D+03	.310D+04	.145D+12
236.64	.250D+00	.629D+07	.141D+04	-.721D+03	.312D+04	.145D+12
243.60	.219D+00	.630D+07	-.373D+04	-.756D+03	.312D+04	.145D+12
250.56	.191D+00	.627D+07	-.906D+04	-.775D+03	.311D+04	.145D+12
257.52	.164D+00	.620D+07	-.144D+05	-.773D+03	.309D+04	.145D+12
264.48	.140D+00	.610D+07	-.198D+05	-.752D+03	.305D+04	.145D+12
271.44	.117D+00	.595D+07	-.248D+05	-.687D+03	.299D+04	.145D+12
278.40	.970D-01	.577D+07	-.293D+05	-.611D+03	.292D+04	.145D+12
285.36	.786D-01	.557D+07	-.332D+05	-.530D+03	.285D+04	.145D+12
292.32	.621D-01	.533D+07	-.366D+05	-.446D+03	.276D+04	.145D+12
299.28	.473D-01	.507D+07	-.395D+05	-.361D+03	.266D+04	.145D+12
306.24	.343D-01	.480D+07	-.417D+05	-.277D+03	.255D+04	.145D+12
313.20	.229D-01	.450D+07	-.433D+05	-.195D+03	.244D+04	.145D+12
320.16	.129D-01	.420D+07	-.444D+05	-.116D+03	.233D+04	.145D+12
327.12	.441D-02	.390D+07	-.450D+05	-.415D+02	.221D+04	.145D+12
334.08	-.281D-02	.359D+07	-.450D+05	.277D+02	.210D+04	.145D+12
341.04	-.882D-02	.328D+07	-.446D+05	.909D+02	.198D+04	.145D+12
348.00	-.137D-01	.297D+07	-.438D+05	.148D+03	.187D+04	.145D+12
354.96	-.177D-01	.267D+07	-.426D+05	.198D+03	.175D+04	.145D+12
361.92	-.207D-01	.238D+07	-.410D+05	.241D+03	.164D+04	.145D+12
368.88	-.229D-01	.210D+07	-.392D+05	.277D+03	.154D+04	.145D+12
375.84	-.244D-01	.184D+07	-.372D+05	.306D+03	.144D+04	.145D+12
382.80	-.253D-01	.159D+07	-.350D+05	.329D+03	.134D+04	.145D+12
389.76	-.257D-01	.135D+07	-.326D+05	.345D+03	.125D+04	.145D+12
396.72	-.256D-01	.113D+07	-.302D+05	.355D+03	.117D+04	.145D+12
403.68	-.252D-01	.931D+06	-.277D+05	.360D+03	.109D+04	.145D+12
410.64	-.244D-01	.746D+06	-.252D+05	.360D+03	.102D+04	.145D+12
417.60	-.234D-01	.579D+06	-.227D+05	.355D+03	.961D+03	.145D+12
424.56	-.222D-01	.429D+06	-.203D+05	.347D+03	.905D+03	.145D+12
431.52	-.208D-01	.296D+06	-.179D+05	.335D+03	.854D+03	.145D+12
438.48	-.193D-01	.178D+06	-.156D+05	.320D+03	.810D+03	.145D+12
445.44	-.178D-01	.765D+05	-.135D+05	.303D+03	.771D+03	.145D+12
452.40	-.163D-01	-.106D+05	-.114D+05	.284D+03	.747D+03	.145D+12
459.36	-.147D-01	-.841D+05	-.952D+04	.263D+03	.774D+03	.145D+12
466.32	-.132D-01	-.145D+06	-.776D+04	.242D+03	.797D+03	.145D+12
473.28	-.117D-01	-.194D+06	-.615D+04	.220D+03	.816D+03	.145D+12
480.24	-.103D-01	-.232D+06	-.469D+04	.199D+03	.830D+03	.145D+12
487.20	-.902D-02	-.260D+06	-.338D+04	.177D+03	.841D+03	.145D+12
494.16	-.779D-02	-.280D+06	-.222D+04	.157D+03	.848D+03	.145D+12
501.12	-.665D-02	-.292D+06	-.120D+04	.137D+03	.853D+03	.145D+12
508.08	-.560D-02	-.298D+06	-.314D+03	.118D+03	.855D+03	.145D+12
515.04	-.466D-02	-.298D+06	.443D+03	.999D+02	.855D+03	.145D+12
522.00	-.382D-02	-.293D+06	.108D+04	.835D+02	.853D+03	.145D+12
528.96	-.307D-02	-.284D+06	.161D+04	.686D+02	.850D+03	.145D+12
535.92	-.242D-02	-.271D+06	.204D+04	.551D+02	.845D+03	.145D+12
542.88	-.186D-02	-.256D+06	.238D+04	.432D+02	.839D+03	.145D+12
549.84	-.139D-02	-.239D+06	.265D+04	.328D+02	.833D+03	.145D+12
556.80	-.994D-03	-.220D+06	.285D+04	.240D+02	.826D+03	.145D+12
563.76	-.674D-03	-.199D+06	.299D+04	.166D+02	.818D+03	.145D+12
570.72	-.421D-03	-.178D+06	.308D+04	.105D+02	.810D+03	.145D+12
577.68	-.228D-03	-.157D+06	.314D+04	.579D+01	.802D+03	.145D+12
584.64	-.867D-04	-.135D+06	.317D+04	.224D+01	.793D+03	.145D+12
591.60	.900D-05	-.113D+06	.317D+04	-.237D+00	.785D+03	.145D+12
598.56	.669D-04	-.907D+05	.317D+04	-.179D+01	.777D+03	.145D+12

605.52	.944D-04	-.687D+05	.315D+04	-.257D+01	.769D+03	.145D+12
612.48	.988D-04	-.469D+05	.313D+04	-.273D+01	.760D+03	.145D+12
619.44	.875D-04	-.251D+05	.311D+04	-.246D+01	.752D+03	.145D+12
626.40	.679D-04	-.350D+04	.239D+04	-.206D+03	.744D+03	.145D+12
633.36	.470D-04	.815D+04	.117D+04	-.145D+03	.746D+03	.145D+12
640.32	.289D-04	.128D+05	.351D+03	-.901D+02	.747D+03	.145D+12
647.28	.151D-04	.131D+05	-.126D+03	-.470D+02	.748D+03	.145D+12
654.24	.559D-05	.110D+05	-.351D+03	-.175D+02	.747D+03	.145D+12
661.20	-.167D-06	.818D+04	-.410D+03	.522D+00	.746D+03	.145D+12
668.16	-.319D-05	.534D+04	-.373D+03	.996D+01	.745D+03	.145D+12
675.12	-.442D-05	.299D+04	-.291D+03	.138D+02	.744D+03	.145D+12
682.08	-.465D-05	.130D+04	-.192D+03	.145D+02	.743D+03	.145D+12
689.04	-.444D-05	.313D+03	-.932D+02	.139D+02	.743D+03	.145D+12
696.00	-.413D-05	.000D+00	.000D+00	.129D+02	.743D+03	.145D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.474D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .874D-07 LBS

OUTPUT SUMMARY

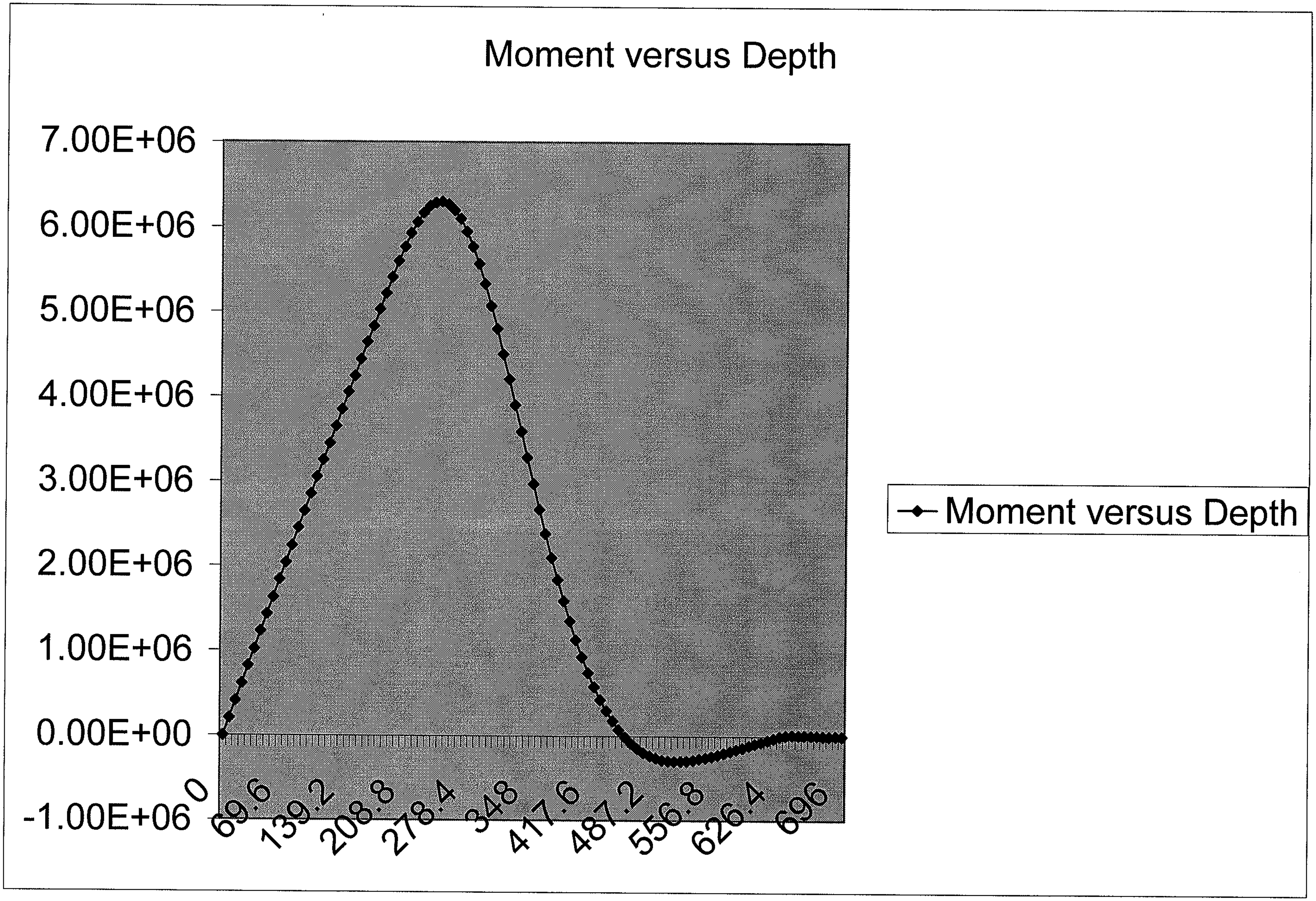
PILE-HEAD DEFLECTION = .220D+01 IN
MAXIMUM BENDING MOMENT = .630D+07 LBS-IN
MAXIMUM SHEAR FORCE = -.450D+05 LBS
NO. OF ITERATIONS = 10
NO. OF ZERO DEFLECTION POINTS = 3

S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.2200D+01	.0000D+00	.5250D+06	.2200D+01	.6300D+07	-.4500D+05

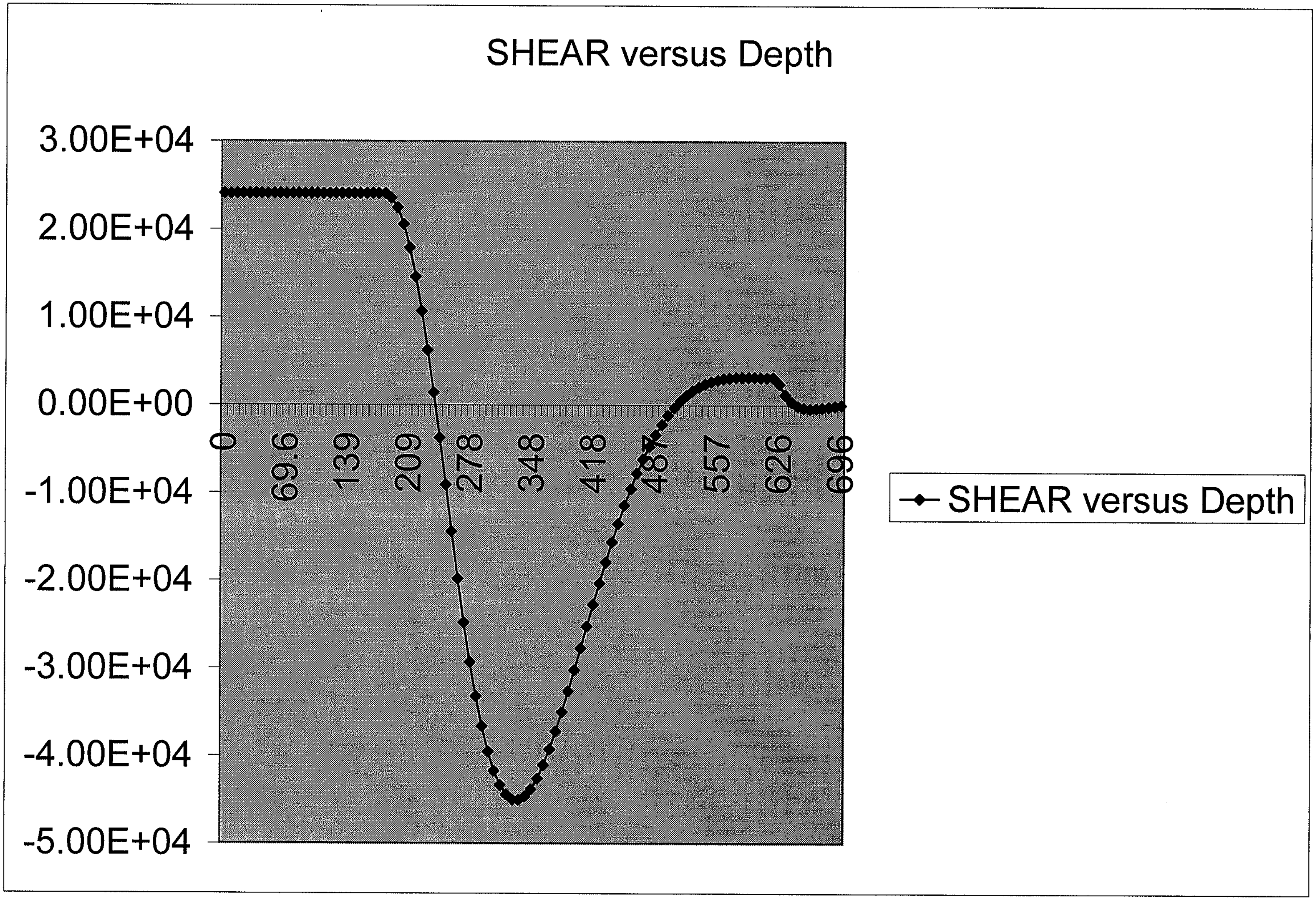
525 ft K

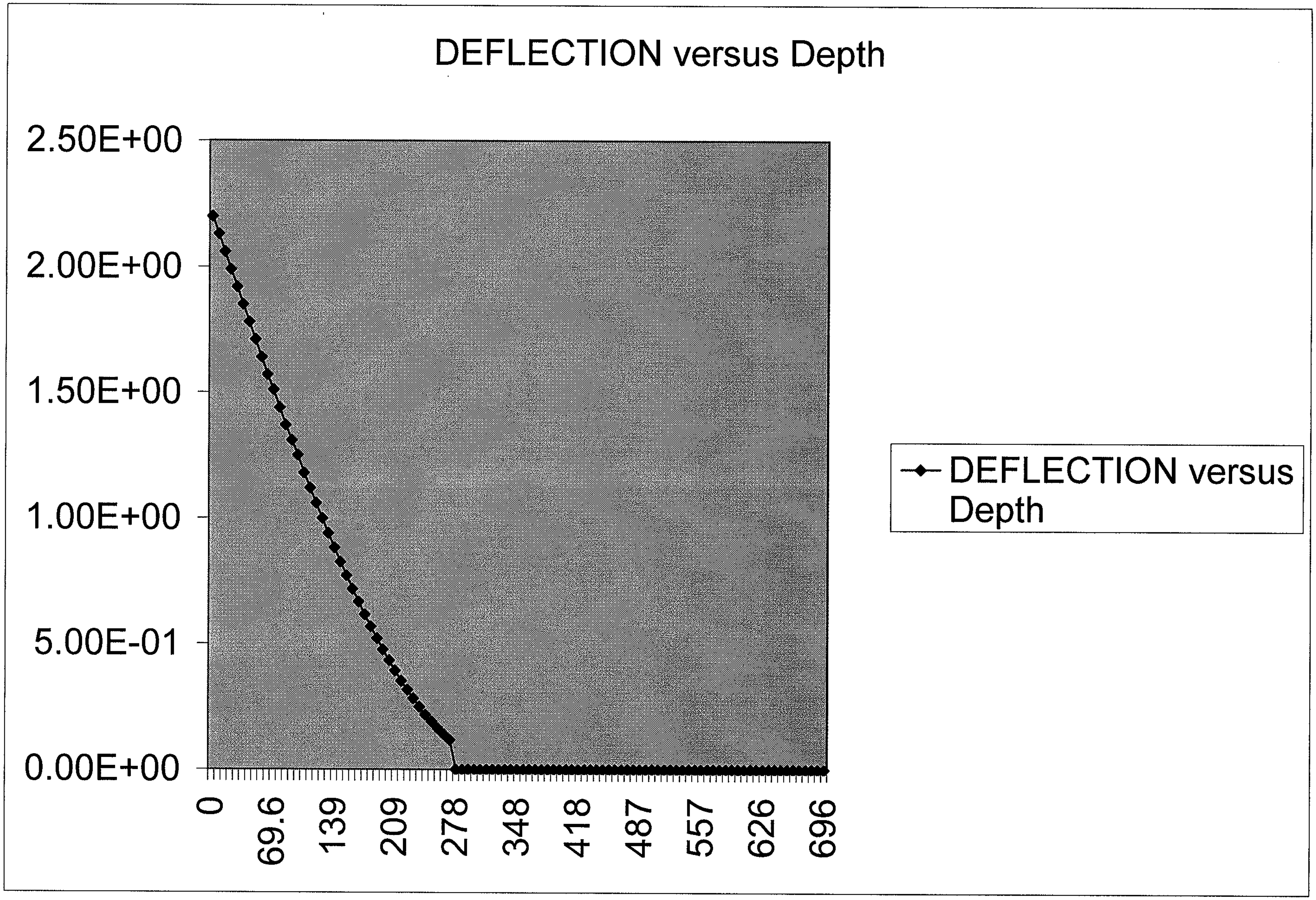
474



Moment versus Depth
7.00E+06
6.00E+06
5.00E+06
4.00E+06
3.00E+06
2.00E+06
1.00E+06
0.00E+00
-1.00E+06
0
69.6
139.2
208.8
278.4
348
417.6
487.2
556.8
626.4
696
Moment versus Depth

475


SHEAR versus Depth
3.00E+04
2.00E+04
1.00E+04
0.00E+00
-1.00E+04
-2.00E+04
-3.00E+04
-4.00E+04
-5.00E+04
0
69.6
139
209
278
348
418
487
557
626
696
SHEAR versus Depth

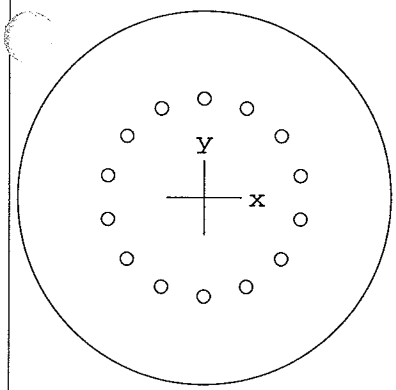

DEFLECTION versus Depth
2.50E+00
2.00E+00
1.50E+00
1.00E+00
5.00E-01
0.00E+00
0
69.6
139
209
278
348
418
487
557
626
696
DEFLECTION versus Depth

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

$V_u = 67.5\ k$
$M_u \approx 916.7\ ft\,k$
} 36" ϕ

$V_u = 44.2\ k$
$M_u = 520.7\ ft\,k$
} 30" ϕ

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 479 of



30.0 inch diam.

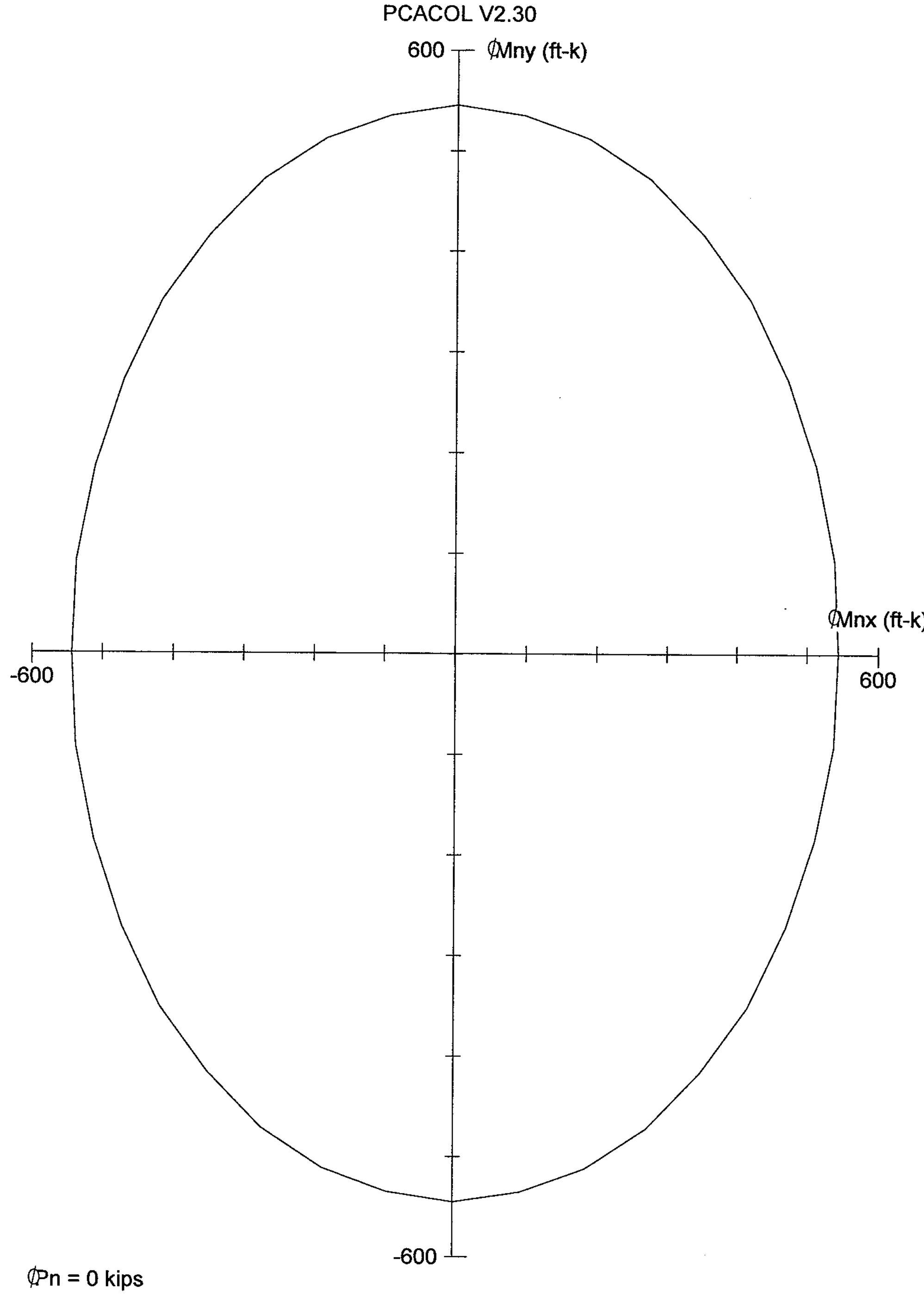
f'c = 4.0 ksi

fy = 60.0 ksi

Confinement: Tied
clr cover = 6.50 in
spacing = 2.40 in
14-#9 at 1.98%
As = 14 in^2

Ix = 39761 in^4
Iy = 39761 in^4

Xo = 0.00 in
Yo = 0.00 in



File name: P:\0224\16212_~1\CALCS\PCACOL\ABUT.COL

Project:	Material Properties:	
Column Id:	Ec = 3834 ksi	eu = 0.003 in/in
Engineer:	fc = 3.40 ksi	Es = 29000 ksi
Date: 03/09/09 Time: 10:52:34	Beta1 = 0.85	
Code: ACI 318-89	Stress Profile: Block	
Units: in-lb	phi(c) = 0.70, phi(b) = 0.90	

X-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

Y-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

479

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0000000    00000    00000    00000    00000   00
00    00  00    00  00    00  00    00  00    00  00
00    00  00    00  00    00  00    00  00    00  00
00    00  00        00    00  00        00    00  00
00    00  00        0000000   00        00    00  00
0000000   00    00  00    00  00    00  00    00  00
00        00    00  00    00  00    00  00    00  00
00         00000    00    00   00000    00000   00000  (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
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```
General Information:
====================
  File Name:  P:\0224\16212_~1\CALCS\PCACOL\ABUT.COL
  Project:                            Code: ACI 318-89
  Column:                             Units: US in-lbs
  Engineer:                           Date: 03/09/09  Time: 10:52:34

  Run Option: Design                  Slender column
  Run Axis:   Biaxial                 Column Type: Structural

Material Properties:
====================
  f'c    = 4 ksi                          fy    = 60 ksi
  Ec     = 3834.25 ksi                    Es    = 29000 ksi
  fc     = 3.4 ksi                        erup = 0 in/in
  eu     = 0.003 in/in
  Stress Profile: Block                   Beta1 = 0.85

Geometry:
=========
  Circular:    Diameter = 30 in

  Gross section area, Ag =  706.858 in^2
  Ix =  39760.8 in^4                      Xo =  0 in
  Iy =  39760.8 in^4                      Yo =  0 in

Reinforcement:
==============
  Rebar Database: ASTM
  Size     Diam    Area  Size     Diam    Area  Size     Diam    Area
  ---------------------------------------------------------------------
     3     0.38    0.11     4     0.50    0.20     5     0.63   0.31
     6     0.75    0.44     7     0.88    0.60     8     1.00   0.79
     9     1.13    1.00    10     1.27    1.27    11     1.41   1.56
    14     1.69    2.25    18     2.26    4.00

  Confinement: Tied; phi(c) = 0.7,  phi(b) = 0.9,  a = 0.8
  #4 ties with #10 bars, #5 with larger bars.

  Layout: Circular
  Pattern: All Sides Equal  [Cover to transverse reinforcement (ties)]

  Total steel area, As = 14.00 in^2 at 1.98%

  14-#9   Cover = 6 in
```

4181

Slenderness:
============

X-axis: Braced against sidesway -- Not hinged at either end.
Y-axis: Braced against sidesway -- Not hinged at either end.

Columns:

Col.	Axis	Height (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Design	X	60	30	30	39760.8	4	3834.25
	Y	60			39760.8		
Above	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					
Below	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					

Beams:

X-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Y-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	N/A	96.0
Y	0.000	0.000	1.000	N/A	96.0

Moment Magnification Factors:

Beta(d) load case factors: Dead = 1.4, Live = 1.7
Strength reduction factor = 0.7

Load Comb	Braced (X-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (X-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	412	1.000	2.16e+007	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.000	* * Not Applicable * *		
U3				0.400	1.000	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Comb	Braced (Y-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (Y-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	412	1.000	2.16e+007	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.000	* * Not Applicable * *		
U3				0.400	1.000	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Combinations:

U1 = 1.000*Dead + 1.700*Live + 0.000*Lateral
U2 = 1.050*Dead + 1.275*Live + 1.275*Lateral
U3 = 1.050*Dead + 0.000*Live + 1.275*Lateral
U4 = 0.900*Dead + 0.000*Live + 1.300*Lateral

Service Loads:

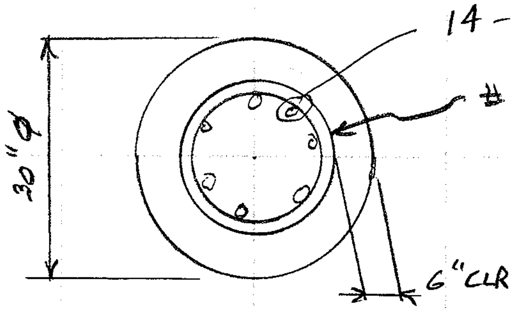
No.	Load Case	Axial Load (kip)	Moments about X-axis @ Top (ft-k)	@ Bot (ft-k)	Moments about Y-axis @ Top (ft-k)	@ Bot (ft-k)
1	Dead	90	521	521	100	100
	Live	0	0	0	0	0
	Latl	0	0	0	0	0

NOTE: Each loading combination includes the following cases:
First line - moment at column top.
Second line - moment at column bottom.
Third line - moment due to minimum X-Eccentricity.
Fourth line - moment due to minimum Y-Eccentricity.

		Applied Loads			Computed Strength			Computed/
Pt.	Load Comb	P (kips)	Mx (ft-k)	My (ft-k)	P (kips)	Mx (ft-k)	My (ft-k)	Applied Ray length
1	1 U1	90	521	100	93	543	104	1.041
2			-521	-100	93	-543	-104	1.041
3			11	0	1790	224	-0	19.903
4			0	11	1790	0	223	19.902
5	1 U2	94	547	105	93	543	104	0.991
6			-547	-105	93	-543	-104	0.991
7			12	0	1790	224	-0	18.956
8			0	12	1790	0	223	18.955
9	1 U3	94	547	105	93	543	104	0.991
10			-547	-105	93	-543	-104	0.991
11			12	0	1790	224	-0	18.956
12			0	12	1790	0	223	18.955
13	1 U4	81	469	90	93	543	104	1.157
14			-469	-90	93	-543	-104	1.157
15			10	0	1790	224	-0	22.115
16			0	10	1790	0	223	22.114

Program completed as requested!

CAISSON



$d_e = D/2 + \frac{D_r}{\pi}$ (5.8.2.9-2)

$D = 30"$

$D_r = 30" - 6"(2) - 1" - 1" \approx 16"$

$d_e \approx \frac{30}{2} + \frac{16}{\pi} \approx 20.09"$

Max Shear ≈ 45 K * 1.2 = 54 K

$d_v = .9 d_e = .9(20.09) = 18.08"$

$S \leq 12"$ per 5.10.6.3

$d_v = .72h = .72(30) = 21.6"$

assume $\beta = 2.0$
$\theta = 45°$

$V_c = .0316 \beta \sqrt{f'_c}\, b_v d_v$ (5.8.3.3-3)

$V_c = .0316 (2.0) \sqrt{4.0} (30")(21.6")$

$V_c = 81.9$ K

$V_u = 54 \geq .5 \phi V_c$ (5.8.2.4-1)

∴ Transverse Reinforcement Required

$A_v \geq .0316 \sqrt{f'_c} \frac{b_v s}{f_y}$ (5.8.2.5-1)

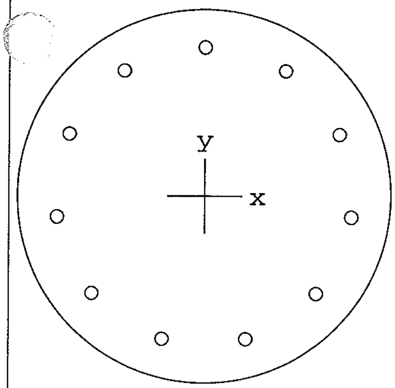
$A_v \geq .0316 \sqrt{4.0}\ 30"\ 6"/60\text{ ksi}$

$A_v \geq .19 \text{ in}^2$

spiral or tie = 2 legs
$\#4 = .2(2) = .40 \text{ in}^2$

use #4 spiral @ 6" pitch

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 485 of



30.0 inch diam.

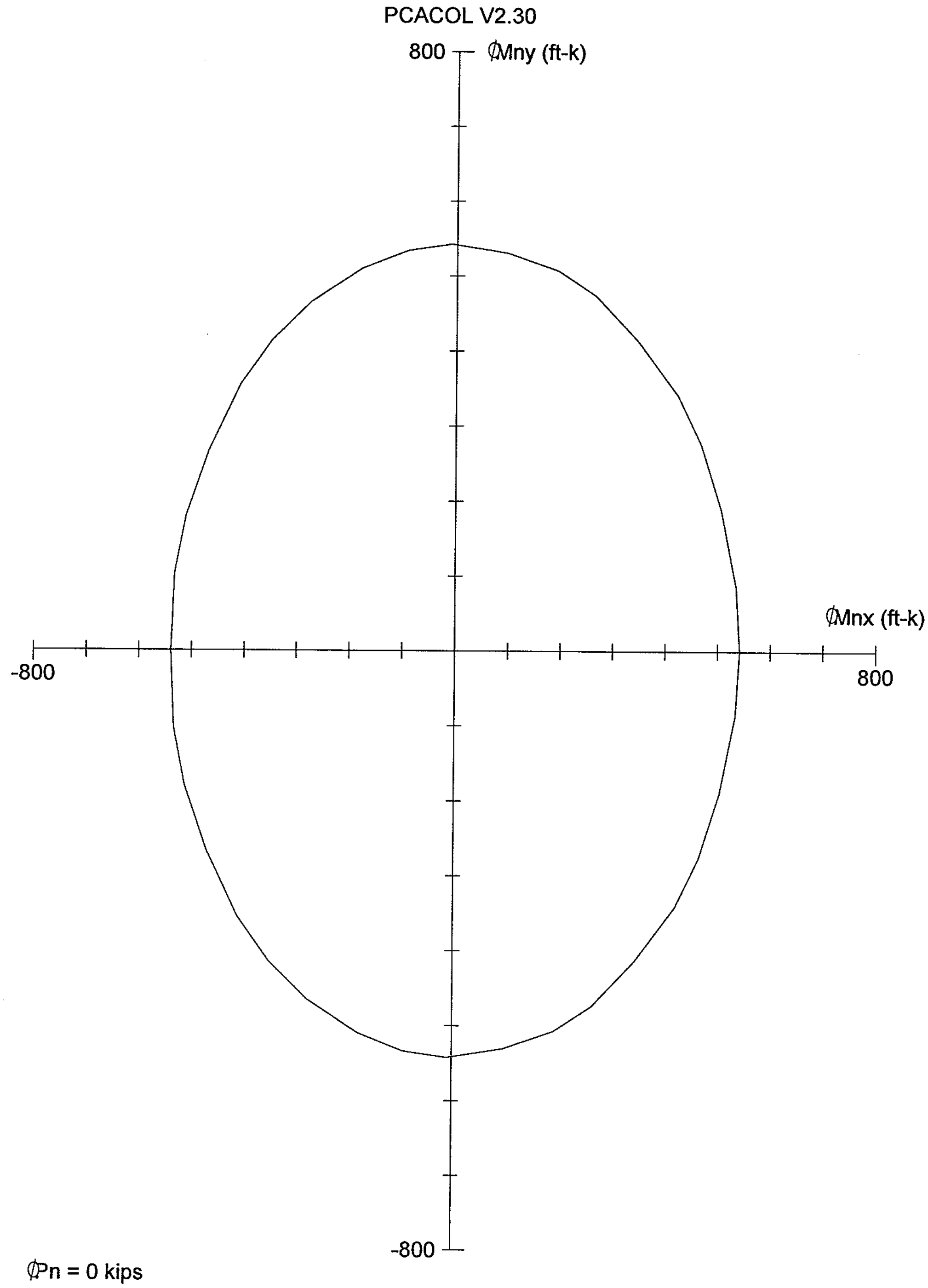
f'c = 4.5 ksi

fy = 60.0 ksi

Confinement: Tied
clr cover = 2.50 in
spacing = 5.60 in
11-#9 at 1.56%
As = 11 in^2

Ix = 39761 in^4
Iy = 39761 in^4

Xo = 0.00 in
Yo = 0.00 in



File name: P:\0224\16212_~1\CALCS\PCACOL\ABUTCOL.COL

Project:

Column Id:

Engineer:

Date: 03/09/09 Time: 10:52:34

Code: ACI 318-89

Units: in-lb

Material Properties:

Ec = 4067 ksi eu = 0.003 in/in

fc = 3.83 ksi Es = 29000 ksi

Beta1 = 0.82

Stress Profile: Block

phi(c) = 0.70, phi(b) = 0.90

X-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

Y-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

```
0000000   00000   00000   00000   00000   00
00    00 00   00 00   00 00   00 00   00 00
00    00 00   00 00   00 00   00 00   00 00
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===
Computer program for the Strength Design of Reinforced Concrete Sections
===

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General Information:
====================

File Name: P:\0224\16212_~1\CALCS\PCACOL\ABUTCOL.COL
Project:
Column:
Engineer:

Code: ACI 318-89
Units: US in-lbs
Date: 03/09/09 Time: 10:52:34

Run Option: Design
Run Axis: Biaxial

Slender column
Column Type: Structural

Material Properties:
====================

f'c = 4.5 ksi
Ec = 4066.84 ksi
fc = 3.825 ksi
eu = 0.003 in/in
Stress Profile: Block

fy = 60 ksi
Es = 29000 ksi
erup = 0 in/in
Beta1 = 0.825

Geometry:
=========

Circular: Diameter = 30 in

Gross section area, Ag = 706.858 in^2
Ix = 39760.8 in^4
Iy = 39760.8 in^4

Xo = 0 in
Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: Tied; phi(c) = 0.7, phi(b) = 0.9, a = 0.8
#4 ties with #10 bars, #5 with larger bars.

Layout: Circular
Pattern: All Sides Equal [Cover to transverse reinforcement (ties)]

Total steel area, As = 10.80 in^2 at 1.53%

18-#7 Cover = 2 in

488

Slenderness:

X-axis: Braced against sidesway -- Not hinged at either end.
Y-axis: Braced against sidesway -- Not hinged at either end.

Columns:

Col.	Axis	Height (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Design	X	60	30	30	39760.8	4.5	4066.84
	Y	60			39760.8		
Above	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					
Below	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					

Beams:

X-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Y-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	N/A	96.0
Y	0.000	0.000	1.000	N/A	96.0

Moment Magnification Factors:

Beta(d) load case factors: Dead = 1.4, Live = 1.7
Strength reduction factor = 0.7

Load Comb	Braced (X-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (X-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	525	1.000	2.76e+007	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.000	* * Not Applicable * *		
U3				0.400	1.000	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Comb	Braced (Y-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (Y-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	525	1.000	2.76e+007	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.000	* * Not Applicable * *		
U3				0.400	1.000	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Combinations:

U1 = 1.000*Dead + 1.700*Live + 0.000*Lateral
U2 = 1.050*Dead + 1.275*Live + 1.275*Lateral
U3 = 1.050*Dead + 0.000*Live + 1.275*Lateral
U4 = 0.900*Dead + 0.000*Live + 1.300*Lateral

Service Loads:

No.	Load Case	Axial Load (kip)	Moments about X-axis @ Top (ft-k)	@ Bot (ft-k)	Moments about Y-axis @ Top (ft-k)	@ Bot (ft-k)
1	Dead	90	521	521	100	100
	Live	0	0	0	0	0
	Latl	0	0	0	0	0

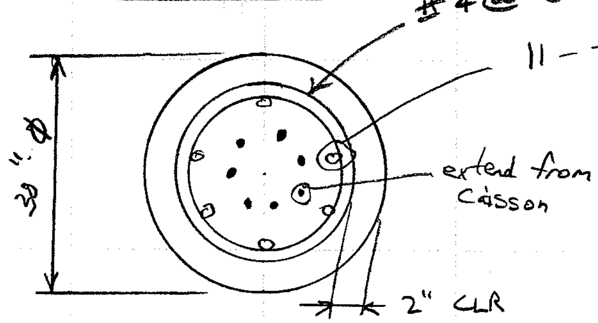
NOTE: Each loading combination includes the following cases:
First line - moment at column top.
Second line - moment at column bottom.
Third line - moment due to minimum X-Eccentricity.
Fourth line - moment due to minimum Y-Eccentricity.

Pt.	Load Comb	Applied Loads P (kips)	Applied Loads Mx (ft-k)	Applied Loads My (ft-k)	Computed Strength P (kips)	Computed Strength Mx (ft-k)	Computed Strength My (ft-k)	Computed/ Applied Ray length
1	1 U1	90	521	100	93	562	107	1.076
2			-521	-100	93	-562	-107	1.076
3			11	0	1854	236	-0	20.635
4			0	11	1854	0	236	20.636
5	1 U2	94	547	105	93	562	107	1.025
6			-547	-105	93	-562	-107	1.025
7			12	0	1854	236	-0	19.653
8			0	12	1854	0	236	19.653
9	1 U3	94	547	105	93	562	107	1.025
10			-547	-105	93	-562	-107	1.025
11			12	0	1854	236	-0	19.653
12			0	12	1854	0	236	19.653
13	1 U4	81	469	90	93	562	107	1.196
14			-469	-90	93	-562	-107	1.196
15			10	0	1854	236	-0	22.928
16			0	10	1854	0	236	22.929

Program completed as requested!

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

COLUMN



Evenly Spaced (or match caisson)

$d_e = D/2 + \frac{D_r}{\pi}$ (5.8.2.9-2)

$D = 30"$

$D_r = 30" - 2"(2) - 1" - 1" \approx 24"$

$d_e = \frac{30}{2} + \frac{24}{\pi} \approx 22.64"$

$d_v = .9 d_e = .9(22.64) \approx 20.37"$

$d_v = .72h = .72(30) \approx 21.6"$ ←

Max Shear ≈ 45 K * 1.2 ≈ 54 K

$S \leq 12"$ per 5.10.6.3

$V_c = 81.9$

use #4 spiral @ 6" pitch

(see caisson calcs.)

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 492 of ______

CAISSON LOADS

Φ (End Bearing) = 0.5

Φ (Side Shear) = 0.55

1480 sap

Pier Load = 2707 kips   Abutment Load = 525 kips

dia=	30	inches	dia=	36	inches
	2.5	feet		3	feet
A=	4.91	sf	A=	7.07	sf
P=	7.85	feet	P=	9.42	feet
Q1dia=	381.11	kips	Q1dia=	561.25	kips
Q2dia=	568.63	kips	Q2dia=	831.27	kips
Q3dia=	731.60	kips	Q3dia=	1065.94	kips
Q4dia=	796.39	kips	Q4dia=	1159.25	kips
Q5dia=	861.19	kips	Q5dia=	1252.55	kips
Q10dia=	1185.17	kips	Q10dia=	1719.08	kips

dia=	54	inches	dia=	60	inches
	4.5	feet		5	feet
A=	15.90	sf	A=	19.63	sf
P=	14.14	feet	P=	15.71	feet
Q1dia=	1309.46	kips	Q1dia=	1628.13	kips
Q2dia=	1917.00	kips	Q2dia=	2378.19	kips
Q3dia=	2445.02	kips	Q3dia=	3030.07	kips
Q4dia=	2654.96	kips	Q4dia=	3289.25	kips
Q5dia=	2864.90	kips	Q5dia=	3548.43	kips
Q10dia=	3914.58	kips	Q10dia=	4844.34	kips

Use 2½ Φ embed at abutment ≈ 6'

Use 5 Φ embed at piers ≈ 22.5'

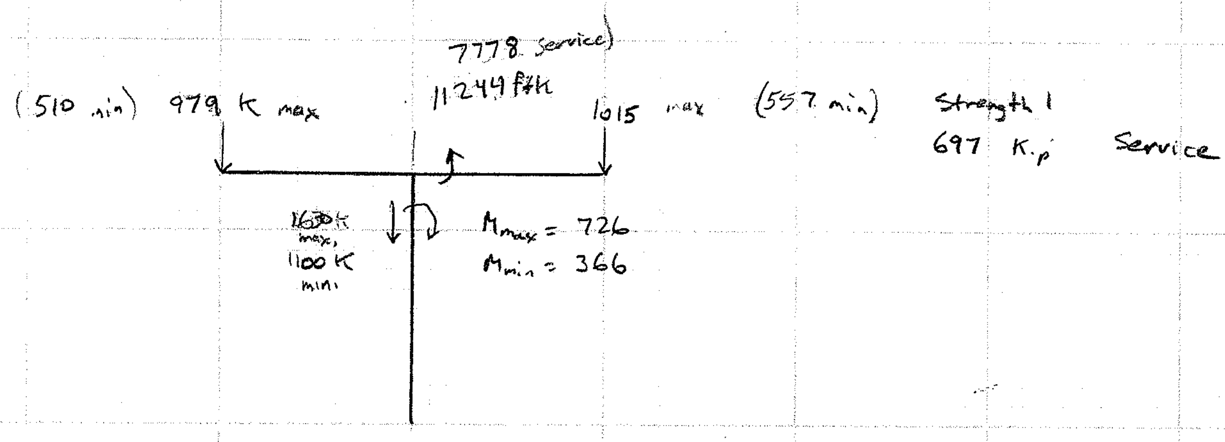
dia=	66	inches	dia=	72	inches
	5.5	feet		6	feet
A=	23.76	sf	A=	28.27	sf
P=	17.28	feet	P=	18.85	feet
Q1dia=	1981.44	kips	Q1dia=	2369.39	kips
Q2dia=	2889.01	kips	Q2dia=	3449.47	kips
Q3dia=	3677.78	kips	Q3dia=	4388.18	kips
Q4dia=	3991.39	kips	Q4dia=	4761.40	kips
Q5dia=	4305.00	kips	Q5dia=	5134.62	kips
Q9.5dia=	5716.25	kips	Q7dia=	5881.06	kips

dia=	78	inches	dia=	84	inches
	6.5	feet		7	feet
A=	33.18	sf	A=	38.48	sf
P=	20.42	feet	P=	21.99	feet
Q1dia=	2791.97	kips	Q1dia=	3249.19	kips
Q2dia=	4059.57	kips	Q2dia=	4719.30	kips
Q3dia=	5161.24	kips	Q3dia=	5996.99	kips
Q4dia=	5599.26	kips	Q4dia=	6504.98	kips
Q5dia=	6037.28	kips	Q5dia=	7012.98	kips
Q10dia=	8227.36	kips	Q10dia=	9552.95	kips

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

PIER DESIGN

FROM SAP 2000



$$\frac{A_s f_y}{A_g f'_c} + \cancel{\frac{A_{ps} f_{pu}}{A_g f'_c}}^{\,0} \geq .135 \qquad 5.7.4.2-3$$

$f_y = 60$ ksi

$f'_c = 4$ ksi

$A_g = \frac{\pi\, 54^2}{4} = 2290.22$

$$\frac{A_s\, 60}{2290.22\,(4)} + 0 \geq .135$$

Caisson: $A_{s_{min}} \geq 20.61\ in^2 \geq 1\% A_g$

$= .9\% A_g$

Column:

$A_g = 54 * 54 = 2916\ in^2$ min.

$f'_c = 4.5$ ksi

$A_{s_{min}} \geq 29.52\ in^2 \geq 1\% A_g$

$= 1\% A_g$

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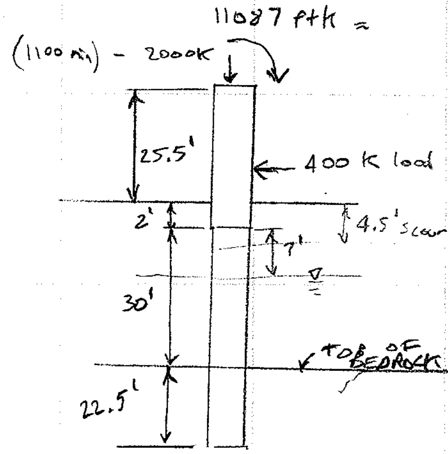
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)



$\gamma_t = 120$ lb/ft^3 = .0694 lb/in^3

$\phi = 32^0$

$c = 0$

$K_n = 64$ lb/in^3

$\gamma_t = 135$ lb/ft^3 = .0781 lb/in^2

$\phi = 0$

$c = 10000$ lb/ft^2 = 69.44 lb/in^2

$K_s = 2000$ lb/in^3

$\varepsilon_{50} = .005$

LPILE INPUT

LINE 1: TITLE

LINE 2: UNITS 1 = ENGLISH

LINE 3: VARIABLES; NI, NDIAM, XGS, XLN

NI = # of Increments = 100

NDIAM = # of segment with different ϕ diam or E ≈ 2

XGS ≈ 27.5' = 330"

XLN ≈ 80' ≈ 960"

LINE 4: XDIAM(I), DIAM(I), MINERT(I) AREA(I), EPILE(I)

XDIAM = x coordindate = 0 TOP, 330" BOTTOM } COLUMN; 330" TOP, 960 BOTTOM } CAISSON

DIAM = DIAMETER OF PILE = 54" - COLUMN; 54" - CAISSON

MINERT = MOMENT of INERTIA = $\frac{\pi d^4}{64}$ = 417393

AREA = $\pi d^2/4$ = 2290.2

EPILE ≈ 3640 Ksi -

By:	Date	Project no.	Project code (SA#):
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LINE 5: NL, NGI, NSTR, NPY

NL = # of soil layers = 2
NGI = # of points on plot = 8
NSTR = # of points in input = 8
NPY = # of input p-y curves = 0

LINE 6: KSOIL (I), XTOP(I), XBOT(I), XK(I)

KSOIL - PY curve code
4 - sand
3 - stiff clay

XTOP I = 306" → 690 sand   360" for scour condition
XBOT I = 690 → 10000 stiff clay

XK = KN = 64 lb/in³ sand
K_n = 2000 lb/in³ clay

LINE 7: XGI (I), GAM(I)

XGI = location
GAM = unit weight = .0694 lb/in³ dry   .0723 lb/in³ wt
.0781 lb/in³ bedrock

LINE 8: XSTR (I), C(I), PHI(I), EE50(I)

SOIL 1   XSTR = 306 - 690   360" for scour

C = 0
PH = 32
EE50 = 0

SOIL 2   XSTR = 690 - 1000

C = 10000 lb/ft³ = 69.44 lb/in²
PHI = 0   EE50 = .005

LINE 12: NW = 5
KYCL = 1
RCYL = 0

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Chk'd:	Date	Structure no.	Sheet 496 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

LINE 13: XW(I), WW(I)

0 0

no load for scour?

SOIL @ 306"

4' above 258"

246, 0
246, 16666.7 lb/in
270, 16666.7 lb/in
270, 0

} 400 kip load distributed over 2'

LINE 14: NLDS = 1

LINE 15: KBC1(I), BC1(I), BC2(I), Q(I)

Q = axial load ≈ 1630 max

factored
no live load

BC1 = displacement = 0 assumed

BC2 = moment ≈ 1480 ft kips = 17760 in kips

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 497 of

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```
CAISSON DESIGN PIER, Structure F-16-XP,

   1
 100     4   .330D+03   .960D+03
 .000D+00   .540D+02   .417D+06   .229D+04   .364D+07
 .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .960D+03   .540D+02   .417D+06   .229D+04   .364D+07
   2     8     8     0
   4   .306D+03   .690D+03   .640D+02
   3   .690D+03   .100D+04   .200D+04
 .100D+01   .000D+00
 .306D+03   .000D+00
 .306D+03   .666D-01
 .414D+03   .666D-01
 .414D+03   .694D-01
 .690D+03   .694D-01
 .690D+03   .780D-01
 .100D+04   .780D-01
 .100D+01   .000D+00   .000D+00   .000D+00
 .306D+03   .000D+00   .000D+00   .000D+00
 .306D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .694D+02   .000D+00   .500D-02
 .100D+04   .694D+02   .000D+00   .500D-02
    5    1   .000D+00
 .100D+01   .000D+00
 .246D+03   .000D+00
 .246D+03   .167D+05
 .270D+03   .167D+05
 .270D+03   .000D+00
   1
   4   .000D+00   .178D+08   .163D+07
   0                .475 worst case
   1     1     0
 100   .100D-05   .360D+03
```


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```
*******************************************************
*    PROGRAM LPILE1                                   *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                  *
*    ALL RIGHTS RESERVED                              *
*   ----------------------------------------------    *
*                                                     *
*              PREPARED ESPECIALLY FOR                *
*                                                     *
*           STATE DEPARTMENT OF HIGHWAYS              *
*                                                     *
*             DENVER, COLORADO 80222                  *
*                                                     *
*                LICENSE NO. 138                      *
*                                                     *
*******************************************************
```

PROGRAM LPILE1

CAISSON DESIGN PIER, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 960.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
960.00	54.000	.417D+06	.229D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 330.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER           =     330.00 IN
X AT THE BOTTOM OF THE LAYER        =     690.00 IN
MODULUS OF SUBGRADE REACTION        =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER           =     690.00 IN
X AT THE BOTTOM OF THE LAYER        =    1000.00 IN
MODULUS OF SUBGRADE REACTION        =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                    8 POINTS
            X,IN      WEIGHT,LBS/IN**3
            1.00         .00D+00
          306.00         .00D+00
          306.00         .67D-01
          414.00         .67D-01
          414.00         .69D-01
          690.00         .69D-01
          690.00         .78D-01
         1000.00         .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                    8 POINTS
            X,IN        C,LBS/IN**2        PHI,DEGREES       E50
            1.00        .000D+00           .000D+00          -----
          306.00        .000D+00           .000D+00          -----
          306.00        .000D+00           .320D+02          -----
          414.00        .000D+00           .320D+02          -----
          414.00        .000D+00           .320D+02          -----
          690.00        .000D+00           .320D+02          -----
          690.00        .694D+02               .000          .500D-02
         1000.00        .694D+02               .000          .500D-02


BOUNDARY AND LOADING CONDITIONS
------------------------------


DISTRIBUTED LOAD CURVE              5 POINTS
                          X,IN      LOAD,LBS/IN
                          1.00        .000D+00
                        246.00        .000D+00
                        246.00        .167D+05
                        270.00        .167D+05
                        270.00        .000D+00

LOADING NUMBER    1

BOUNDARY CONDITION CODE                  =          4
DEFLECTION AT THE PILE HEAD              =   .000D+00 IN
MOMENT AT THE PILE HEAD                  =   .178D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =   .163D+07 LBS

FINITE-DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS                                  =        100
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =   .100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =        100
```

MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
246.00	.000D+00
246.00	.167D+05
270.00	.167D+05
270.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .000D+00 IN
MOMENT AT THE PILE HEAD = .178D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .163D+07 LBS

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
.00	.000D+00	.178D+08	-.215D+06	.000D+00	.186D+04	.152D+13
9.60	.186D-01	.157D+08	-.215D+06	.000D+00	.173D+04	.152D+13
19.20	.381D-01	.136D+08	-.215D+06	.000D+00	.159D+04	.152D+13
28.80	.584D-01	.115D+08	-.215D+06	.000D+00	.146D+04	.152D+13
38.40	.795D-01	.940D+07	-.215D+06	.000D+00	.132D+04	.152D+13
48.00	.101D+00	.729D+07	-.215D+06	.000D+00	.118D+04	.152D+13
57.60	.123D+00	.519D+07	-.215D+06	.000D+00	.105D+04	.152D+13
67.20	.146D+00	.308D+07	-.215D+06	.000D+00	.911D+03	.152D+13
76.80	.168D+00	.976D+06	-.215D+06	.000D+00	.775D+03	.152D+13
86.40	.191D+00	-.113D+07	-.215D+06	.000D+00	.785D+03	.152D+13
96.00	.213D+00	-.323D+07	-.215D+06	.000D+00	.921D+03	.152D+13
105.60	.236D+00	-.534D+07	-.215D+06	.000D+00	.106D+04	.152D+13
115.20	.258D+00	-.744D+07	-.215D+06	.000D+00	.119D+04	.152D+13
124.80	.279D+00	-.955D+07	-.215D+06	.000D+00	.133D+04	.152D+13
134.40	.300D+00	-.117D+08	-.215D+06	.000D+00	.147D+04	.152D+13
144.00	.321D+00	-.138D+08	-.215D+06	.000D+00	.160D+04	.152D+13
153.60	.340D+00	-.159D+08	-.215D+06	.000D+00	.174D+04	.152D+13
163.20	.358D+00	-.180D+08	-.215D+06	.000D+00	.187D+04	.152D+13
172.80	.376D+00	-.200D+08	-.215D+06	.000D+00	.201D+04	.152D+13
182.40	.392D+00	-.221D+08	-.215D+06	.000D+00	.215D+04	.152D+13
192.00	.407D+00	-.242D+08	-.215D+06	.000D+00	.228D+04	.152D+13
201.60	.420D+00	-.263D+08	-.215D+06	.000D+00	.242D+04	.152D+13
211.20	.432D+00	-.284D+08	-.215D+06	.000D+00	.255D+04	.152D+13
220.80	.442D+00	-.305D+08	-.215D+06	.000D+00	.269D+04	.152D+13

501

22'EMBED.OUT

COLUMN / CAISSON

	DEFLECT.	MOMENT	SHEAR			
230.40	.450D+00	-.326D+08	-.215D+06	.000D+00	.282D+04	.152D+13
240.00	.457D+00	-.347D+08	-.215D+06	.000D+00	.296D+04	.152D+13
249.60	.461D+00	-.367D+08	-.135D+06	.000D+00	.309D+04	.152D+13
259.20	.463D+00	-.373D+08	.250D+05	.000D+00	.312D+04	.152D+13
268.80	.462D+00	-.363D+08	.185D+06	.000D+00	.306D+04	.152D+13
278.40	.460D+00	-.337D+08	.265D+06	.000D+00	.289D+04	.152D+13
288.00	.455D+00	-.312D+08	.265D+06	.000D+00	.273D+04	.152D+13
297.60	.448D+00	-.286D+08	.265D+06	.000D+00	.256D+04	.152D+13
307.20	.440D+00	-.260D+08	.265D+06	.000D+00	.240D+04	.152D+13
316.80	.430D+00	-.235D+08	.265D+06	.000D+00	.223D+04	.152D+13
326.40	.419D+00	-.209D+08	.265D+06	.000D+00	.206D+04	.152D+13
336.00	.407D+00	-.183D+08	.265D+06	-.156D+03	.190D+04	.152D+13
345.60	.393D+00	-.158D+08	.262D+06	-.392D+03	.173D+04	.152D+13
355.20	.378D+00	-.133D+08	.257D+06	-.610D+03	.157D+04	.152D+13
364.80	.363D+00	-.108D+08	.250D+06	-.808D+03	.141D+04	.152D+13
374.40	.347D+00	-.839D+07	.242D+06	-.985D+03	.126D+04	.152D+13
384.00	.330D+00	-.609D+07	.232D+06	-.114D+04	.111D+04	.152D+13
393.60	.313D+00	-.389D+07	.220D+06	-.128D+04	.964D+03	.152D+13
403.20	.296D+00	-.181D+07	.207D+06	-.139D+04	.829D+03	.152D+13
412.80	.279D+00	.146D+06	.194D+06	-.148D+04	.721D+03	.152D+13
422.40	.262D+00	.196D+07	.179D+06	-.155D+04	.839D+03	.152D+13
432.00	.244D+00	.364D+07	.164D+06	-.160D+04	.947D+03	.152D+13
441.60	.227D+00	.517D+07	.148D+06	-.162D+04	.105D+04	.152D+13
451.20	.211D+00	.654D+07	.133D+06	-.164D+04	.114D+04	.152D+13
460.80	.195D+00	.777D+07	.117D+06	-.163D+04	.121D+04	.152D+13
470.40	.179D+00	.885D+07	.102D+06	-.161D+04	.128D+04	.152D+13
480.00	.164D+00	.977D+07	.864D+05	-.157D+04	.134D+04	.152D+13
489.60	.149D+00	.106D+08	.715D+05	-.152D+04	.140D+04	.152D+13
499.20	.135D+00	.112D+08	.572D+05	-.146D+04	.144D+04	.152D+13
508.80	.122D+00	.117D+08	.435D+05	-.139D+04	.147D+04	.152D+13
518.40	.109D+00	.121D+08	.305D+05	-.132D+04	.149D+04	.152D+13
528.00	.972D-01	.123D+08	.183D+05	-.123D+04	.151D+04	.152D+13
537.60	.861D-01	.125D+08	.687D+04	-.114D+04	.152D+04	.152D+13
547.20	.758D-01	.125D+08	-.368D+04	-.105D+04	.152D+04	.152D+13
556.80	.662D-01	.124D+08	-.134D+05	-.961D+03	.152D+04	.152D+13
566.40	.573D-01	.123D+08	-.221D+05	-.867D+03	.151D+04	.152D+13
576.00	.492D-01	.120D+08	-.300D+05	-.775D+03	.149D+04	.152D+13
585.60	.419D-01	.117D+08	-.370D+05	-.685D+03	.147D+04	.152D+13
595.20	.352D-01	.113D+08	-.432D+05	-.598D+03	.145D+04	.152D+13
604.80	.292D-01	.109D+08	-.485D+05	-.514D+03	.142D+04	.152D+13
614.40	.239D-01	.104D+08	-.531D+05	-.436D+03	.139D+04	.152D+13
624.00	.193D-01	.990D+07	-.569D+05	-.362D+03	.135D+04	.152D+13
633.60	.152D-01	.934D+07	-.601D+05	-.295D+03	.132D+04	.152D+13
643.20	.117D-01	.876D+07	-.626D+05	-.234D+03	.128D+04	.152D+13
652.80	.869D-02	.815D+07	-.646D+05	-.180D+03	.124D+04	.152D+13
662.40	.621D-02	.753D+07	-.661D+05	-.132D+03	.120D+04	.152D+13
672.00	.419D-02	.689D+07	-.671D+05	-.917D+02	.116D+04	.152D+13
681.60	.258D-02	.625D+07	-.679D+05	-.581D+02	.112D+04	.152D+13
691.20	.136D-02	.559D+07	-.781D+05	-.207D+04	.107D+04	.152D+13
700.80	.471D-03	.475D+07	-.921D+05	-.868D+03	.102D+04	.152D+13
710.40	-.126D-03	.383D+07	-.952D+05	.237D+03	.960D+03	.152D+13
720.00	-.492D-03	.292D+07	-.895D+05	.940D+03	.901D+03	.152D+13
729.60	-.680D-03	.211D+07	-.787D+05	.132D+04	.848D+03	.152D+13
739.20	-.740D-03	.141D+07	-.653D+05	.147D+04	.803D+03	.152D+13
748.80	-.715D-03	.855D+06	-.513D+05	.144D+04	.767D+03	.152D+13
758.40	-.637D-03	.429D+06	-.382D+05	.130D+04	.740D+03	.152D+13
768.00	-.533D-03	.122D+06	-.266D+05	.111D+04	.720D+03	.152D+13
777.60	-.422D-03	-.818D+05	-.169D+05	.894D+03	.717D+03	.152D+13
787.20	-.316D-03	-.203D+06	-.939D+04	.680D+03	.725D+03	.152D+13
796.80	-.222D-03	-.262D+06	-.379D+04	.486D+03	.729D+03	.152D+13
806.40	-.144D-03	-.277D+06	.749D+02	.320D+03	.730D+03	.152D+13
816.00	-.831D-04	-.261D+06	.251D+04	.187D+03	.729D+03	.152D+13
825.60	-.378D-04	-.228D+06	.383D+04	.867D+02	.727D+03	.152D+13

835.20	-.649D-05	-.188D+06	.432D+04	.151D+02	.724D+03	.152D+13
844.80	.135D-04	-.146D+06	.424D+04	-.318D+02	.721D+03	.152D+13
854.40	.246D-04	-.107D+06	.380D+04	-.588D+02	.719D+03	.152D+13
864.00	.292D-04	-.727D+05	.318D+04	-.709D+02	.717D+03	.152D+13
873.60	.294D-04	-.455D+05	.249D+04	-.725D+02	.715D+03	.152D+13
883.20	.269D-04	-.249D+05	.182D+04	-.671D+02	.713D+03	.152D+13
892.80	.228D-04	-.105D+05	.122D+04	-.578D+02	.712D+03	.152D+13
902.40	.181D-04	-.146D+04	.720D+03	-.465D+02	.712D+03	.152D+13
912.00	.133D-04	.332D+04	.330D+03	-.347D+02	.712D+03	.152D+13
921.60	.877D-05	.489D+04	.526D+02	-.231D+02	.712D+03	.152D+13
931.20	.448D-05	.434D+04	-.116D+03	-.120D+02	.712D+03	.152D+13
940.80	.461D-06	.269D+04	-.179D+03	-.125D+01	.712D+03	.152D+13
950.40	-.340D-05	.914D+03	-.141D+03	.930D+01	.712D+03	.152D+13
960.00	-.720D-05	.000D+00	.000D+00	.200D+02	.712D+03	.152D+13

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.942D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.168D-06 LBS

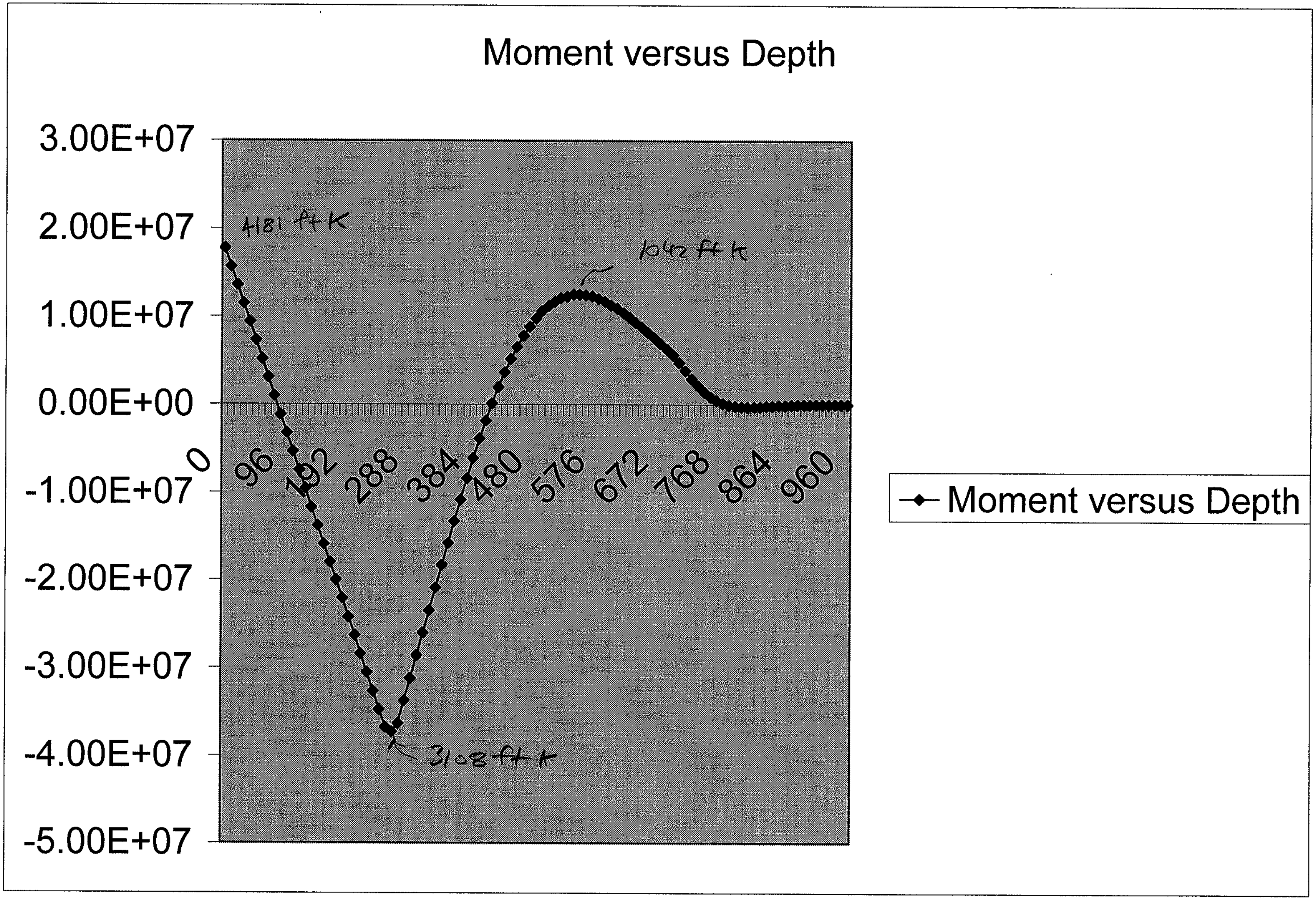
OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .000D+00 IN
MAXIMUM BENDING MOMENT = -.373D+08 LBS-IN
MAXIMUM SHEAR FORCE = .265D+06 LBS
NO. OF ITERATIONS = 8
NO. OF ZERO DEFLECTION POINTS = 3

S U M M A R Y   T A B L E
****

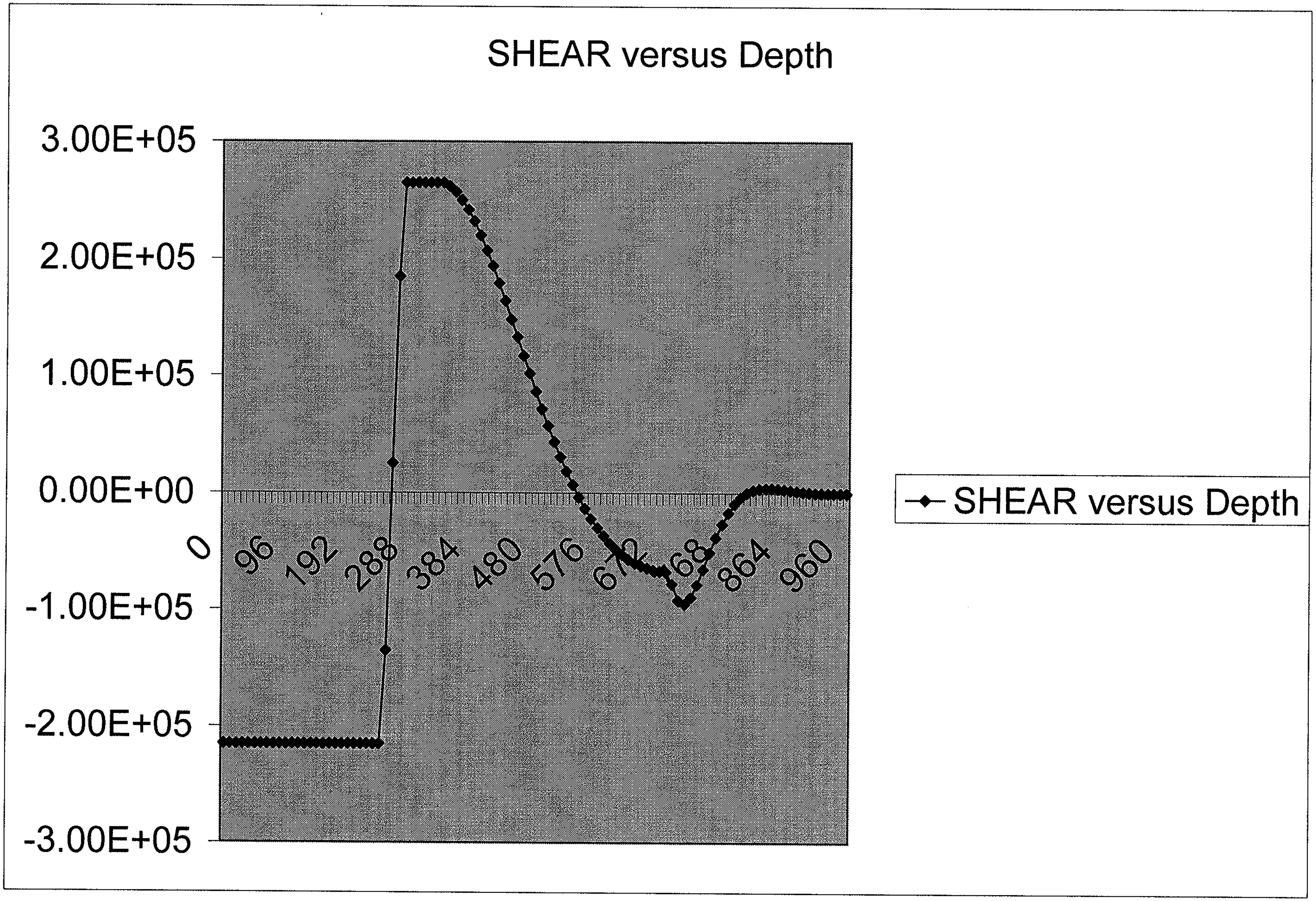
BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.0000D+00	.1780D+08	.1630D+07	.0000D+00	-.3727D+08	.2655D+06

503

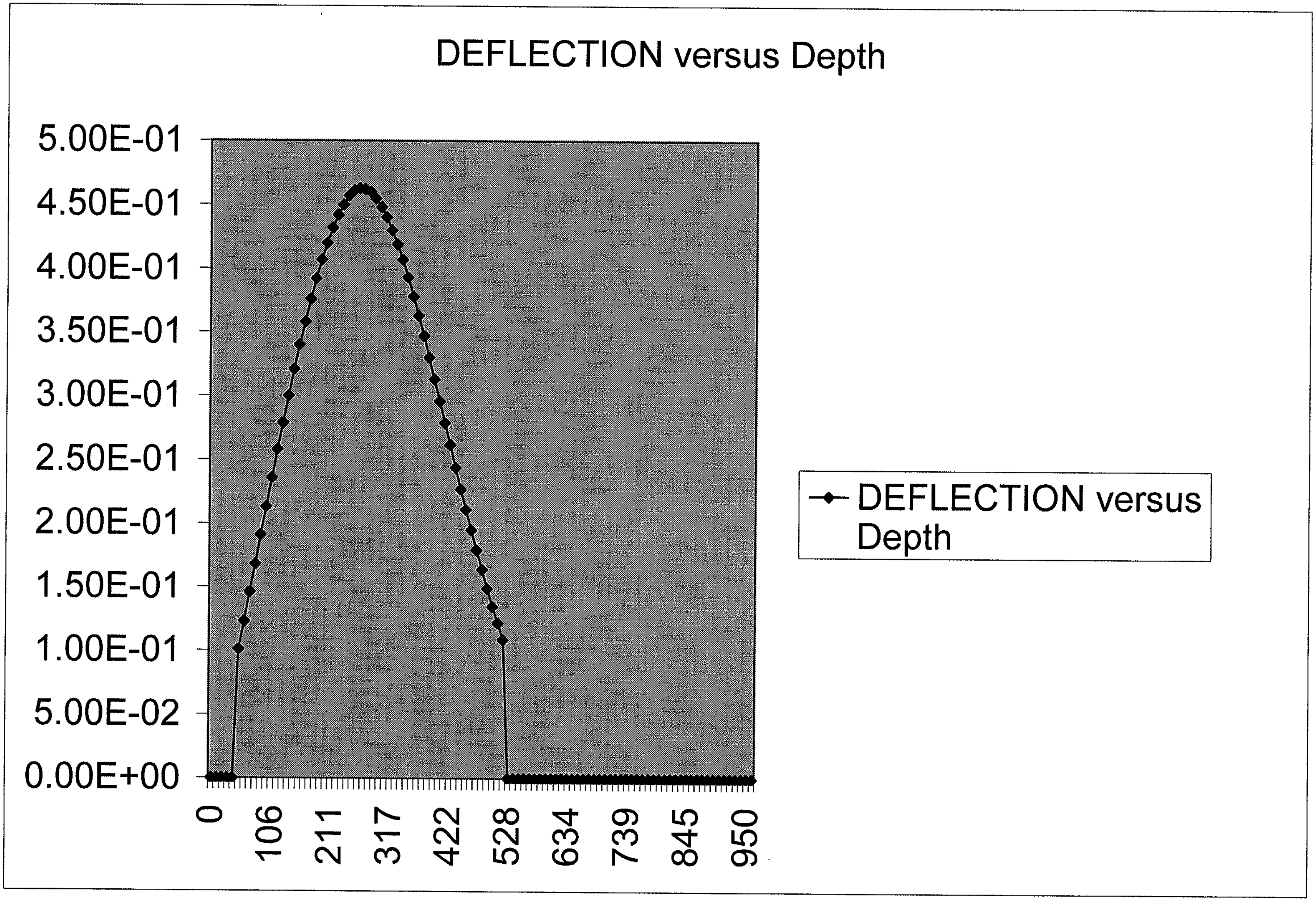


Moment versus Depth
3.00E+07
2.00E+07
1.00E+07
0.00E+00
-1.00E+07
-2.00E+07
-3.00E+07
-4.00E+07
-5.00E+07
0
96
192
288
384
480
576
672
768
864
960
4181 ft k
1042 ft k
3108 ft k
Moment versus Depth

504


SHEAR versus Depth
3.00E+05
2.00E+05
1.00E+05
0.00E+00
-1.00E+05
-2.00E+05
-3.00E+05
0
96
192
288
384
480
576
960
864
SHEAR versus Depth

505

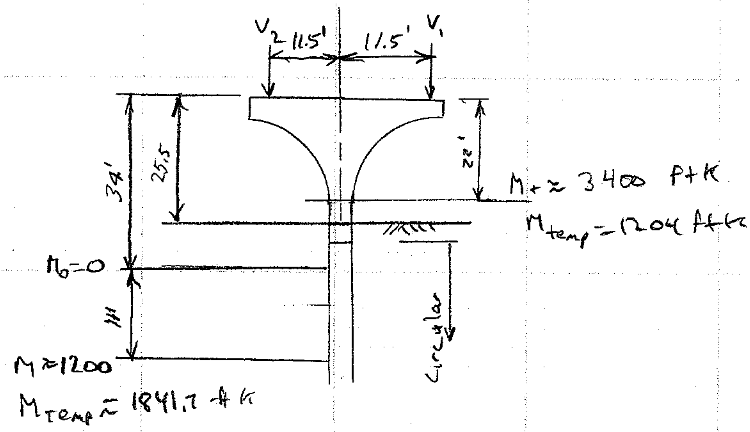

DEFLECTION versus Depth
5.00E-01
4.50E-01
4.00E-01
3.50E-01
3.00E-01
2.50E-01
2.00E-01
1.50E-01
1.00E-01
5.00E-02
0.00E+00
0
106
211
317
422
528
634
739
845
950
DEFLECTION versus Depth

506

10

$M_{Temp} \approx 1204$ ft k

$M_{strength} = 1.2\,(1204) \approx 1445$



$M_{22'} = 3400$ ft k

$M_{34'} \approx 0$

$M_{45'} \approx 1200$ ft k

$M_n \approx 3400$ ft k $+ 1.2\,(1204) \approx 4845$

$\phi = .75$

$A_{g_{caisson}} = \dfrac{\pi\, 54^2}{4} = 2290.22$

A_g column $= 54 \times 54 \approx 2916$

$h = 54''$

COLUMN

$\dfrac{\phi M_n}{A_g h} = \dfrac{.75\,(4845)(12)}{54 \cdot 2916} \approx .28$

$P \approx 2000\,K_{max}$

$\dfrac{\phi P_n}{A_g} = \dfrac{.9\,(2000)}{2916} = .62$

$\Rightarrow$ use $\rho_g \approx .01$ min. for Column

CAISSON

$\dfrac{\phi M_n}{A_g h} \approx \dfrac{.75\,[1200 + 1.2\,(1842)]\,12}{(2290.22)\,54} \approx .25$

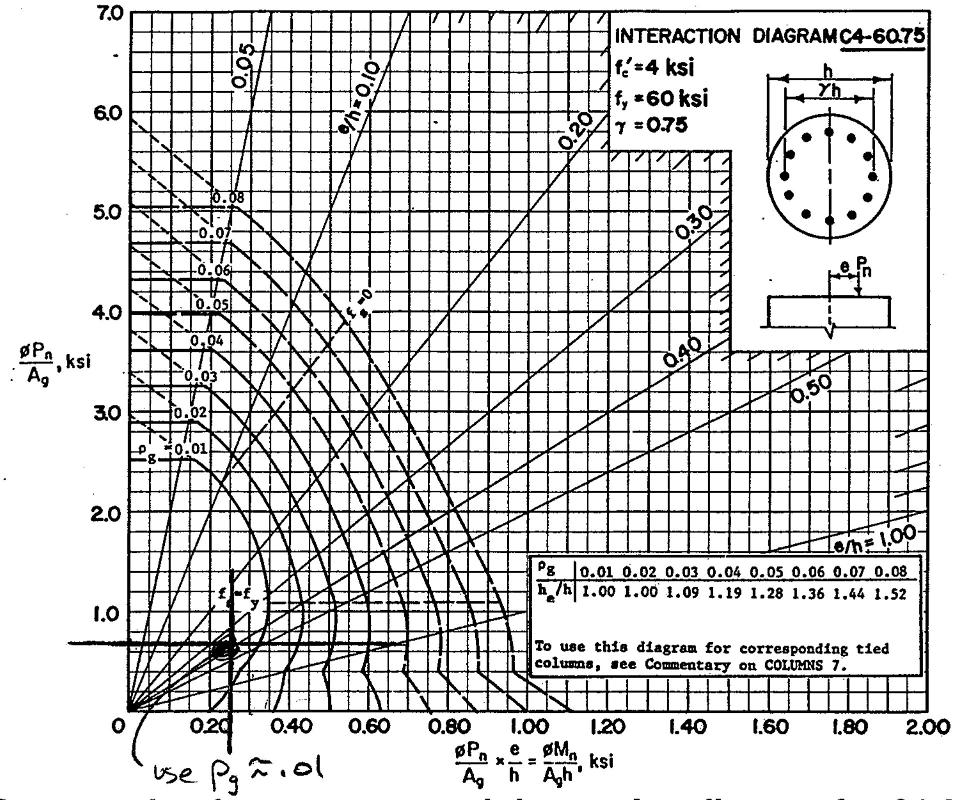
$\dfrac{\phi P_n}{A_g} = \dfrac{.75\,(2000)}{2290.22} \approx .66$

$\Rightarrow$ Use $\rho_g \approx .01$ min. for Caisson

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 507 of

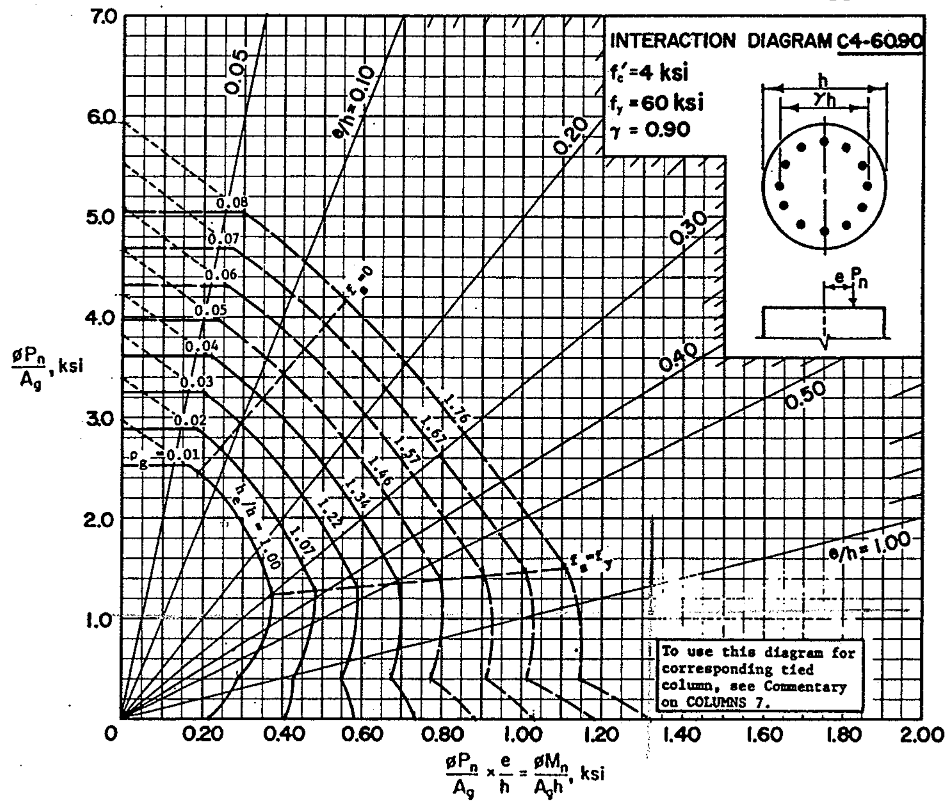
COLUMNS 7.22.3—Load-moment strength interaction diagram for C4-60.75 spirally reinforced columns

References: ACI 318-89 Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



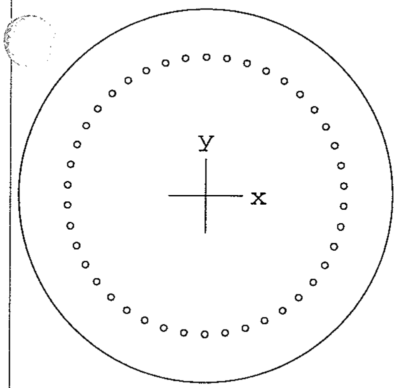
COLUMNS 7.22.4—Load-moment strength interaction diagram for C4-60.90 spirally reinforced columns

References: ACI 318-89 Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



For use of these Design Aids, see Columns Examples 1-8 and 11-16.

508



54.0 inch diam.

f'c = 4.0 ksi

fy = 60.0 ksi

Confinement: Tied

clr cover = 6.50 in

spacing = 1.99 in

42-#8 at 1.45%

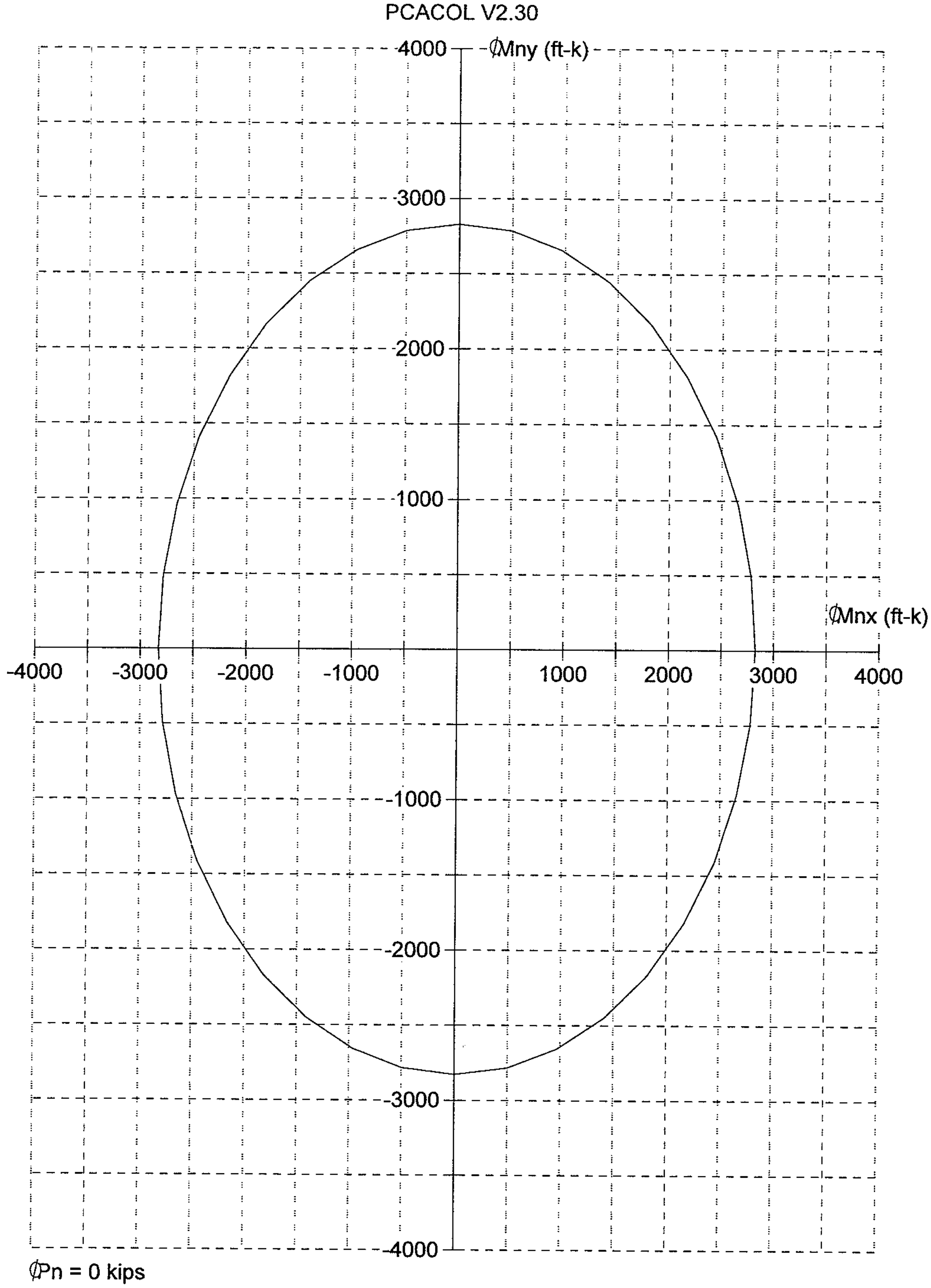
As = 33 in^2

Ix = 417393 in^4

Iy = 417393 in^4

Xo = 0.00 in

Yo = 0.00 in



Licensed To: Licensee name not yet specified.

File name: P:\0224\16212_~1\CALCS\PCACOL\CAISSON.COL

Project:	Material Properties:	
Column Id:	Ec = 3834 ksi	eu = 0.003 in/in
Engineer:	fc = 3.40 ksi	Es = 29000 ksi
Date: 03/09/09 Time: 10:52:34	Beta1 = 0.85	
Code: ACI 318-89	Stress Profile: Block	
Units: in-lb	phi(c) = 0.70, phi(b) = 0.90	

X-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

Y-axis slenderness is considered; k(b) = 1.00 k(s) = 1.20

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```
0000000   00000    00000    00000    00000   00
00    00 00    00 00    00 00    00 00    00 00
00    00 00    00 00    00 00    00 00    00 00
00    00 00       00    00 00       00    00 00
00    00 00       0000000  00       00    00 00
0000000  00    00 00    00 00    00 00    00 00
00       00    00 00    00 00    00 00    00 00
00        00000   00    00  00000    00000   00000  (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
===

Licensee stated above acknowledges that Portland Cement Association (PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the PCACOL(tm) computer program. Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the PCACOL(tm) program. Although PCA has endeavored to produce PCACOL(tm) error free, the program is not and can't be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensees. Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the PCACOL(tm) program.

General Information:
====================

File Name: P:\0224\16212_~1\CALCS\PCACOL\CAISSON.COL
Project: Code: ACI 318-89
Column: Units: US in-lbs
Engineer: Date: 03/09/09 Time: 10:52:34

Run Option: Design Slender column
Run Axis: Biaxial Column Type: Structural

Material Properties:
====================

f'c = 4 ksi fy = 60 ksi
Ec = 3834.25 ksi Es = 29000 ksi
fc = 3.4 ksi erup = 0 in/in
eu = 0.003 in/in
Stress Profile: Block Beta1 = 0.85

Geometry:
=========

Circular: Diameter = 54 in

Gross section area, Ag = 2290.22 in^2
Ix = 417393 in^4 Xo = 0 in
Iy = 417393 in^4 Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: Tied; phi(c) = 0.7, phi(b) = 0.9, a = 0.8
#4 ties with #10 bars, #5 with larger bars.

Layout: Circular
Pattern: All Sides Equal [Cover to transverse reinforcement (ties)]

Total steel area, As = 33.18 in^2 at 1.45%

42-#8 Cover = 6 in

Slenderness:
============

X-axis: Braced against sidesway -- Not hinged at either end.
Y-axis: Braced against sidesway -- Not hinged at either end.

Columns:

Col.	Axis	Height (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Design	X	60	54	54	417393	4	3834.25
	Y	60			417393		
Above	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					
Below	X	(NO COLUMN SPECIFIED...)					
	Y	(NO COLUMN SPECIFIED...)					

Beams:

X-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Y-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	(NO BEAM SPECIFIED...)					
Above Right	(NO BEAM SPECIFIED...)					
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	N/A	53.3
Y	0.000	0.000	1.000	N/A	53.3

Moment Magnification Factors:

Beta(d) load case factors: Dead = 1.4, Live = 1.7
Strength reduction factor = 0.7

Load Comb	Braced (X-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (X-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	4879	1.000	2.56e+008	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.039	* * Not Applicable * *		
U3				0.400	1.039	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Comb	Braced (Y-axis) Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Sway (Y-axis) Pc (kip)	EI (k-in^2)	Delta
1 U1	4879	1.000	2.56e+008	0.400	1.000	* * Not Applicable * *		
U2				0.400	1.039	* * Not Applicable * *		
U3				0.400	1.039	* * Not Applicable * *		
U4				0.400	1.000	* * Not Applicable * *		

Load Combinations:

U1 = 1.000*Dead + 1.700*Live + 0.000*Lateral
U2 = 1.050*Dead + 1.275*Live + 1.275*Lateral
U3 = 1.050*Dead + 0.000*Live + 1.275*Lateral
U4 = 0.900*Dead + 0.000*Live + 1.300*Lateral

Service Loads:

No.	Load Case	Axial Load (kip)	Moments about X-axis @ Top (ft-k)	@ Bot (ft-k)	Moments about Y-axis @ Top (ft-k)	@ Bot (ft-k)
1	Dead	2000	3410	3410	100	100
	Live	0	0	0	0	0
	Latl	0	0	0	0	0

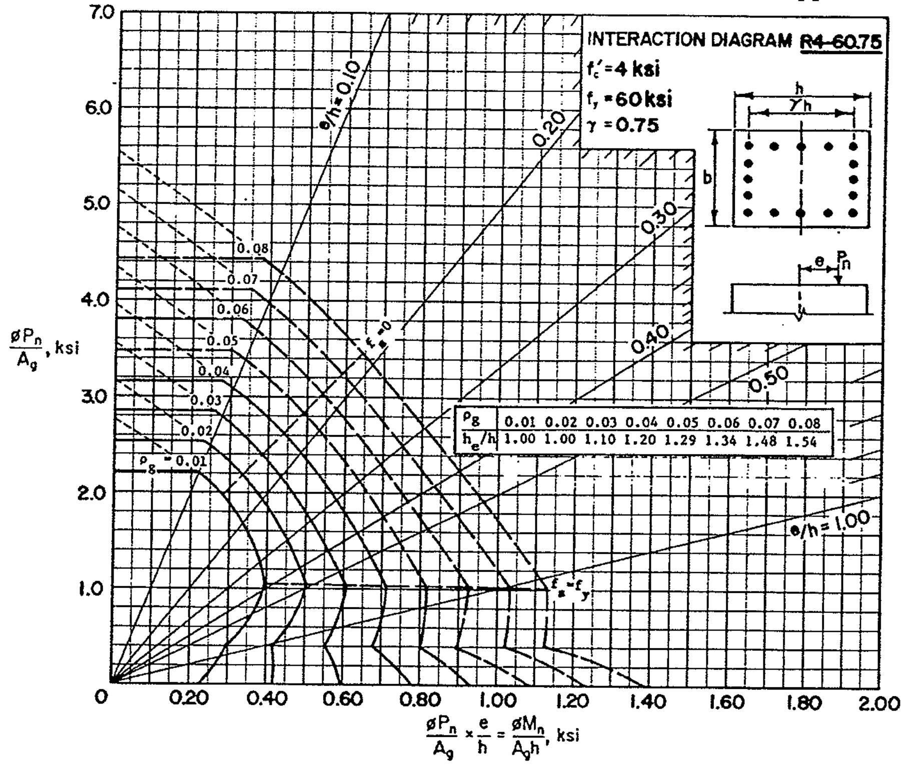
NOTE: Each loading combination includes the following cases:
First line - moment at column top.
Second line - moment at column bottom.
Third line - moment due to minimum X-Eccentricity.
Fourth line - moment due to minimum Y-Eccentricity.

		Applied Loads			Computed Strength			Computed/
Pt.	Load Comb	P (kips)	Mx (ft-k)	My (ft-k)	P (kips)	Mx (ft-k)	My (ft-k)	Applied Ray length
1	1 U1	2000	3410	100	2117	3688	109	1.076
2			-3410	-100	2117	-3688	-109	1.076
3			370	0	5412	989	-0	2.715
4			0	370	5412	0	989	2.715
5	1 U2	2100	3719	109	2027	3675	109	0.983
6			-3719	-109	2027	-3675	-109	0.983
7			404	0	5412	1015	-0	2.585
8			0	404	5412	0	1015	2.585
9	1 U3	2100	3719	109	2027	3675	109	0.983
10			-3719	-109	2027	-3675	-109	0.983
11			404	0	5412	1015	-0	2.585
12			0	404	5412	0	1015	2.585
13	1 U4	1800	3069	90	2117	3688	109	1.195
14			-3069	-90	2117	-3688	-109	1.195
15			333	0	5412	989	-0	3.017
16			0	333	5412	0	989	3.017

Program completed as requested!

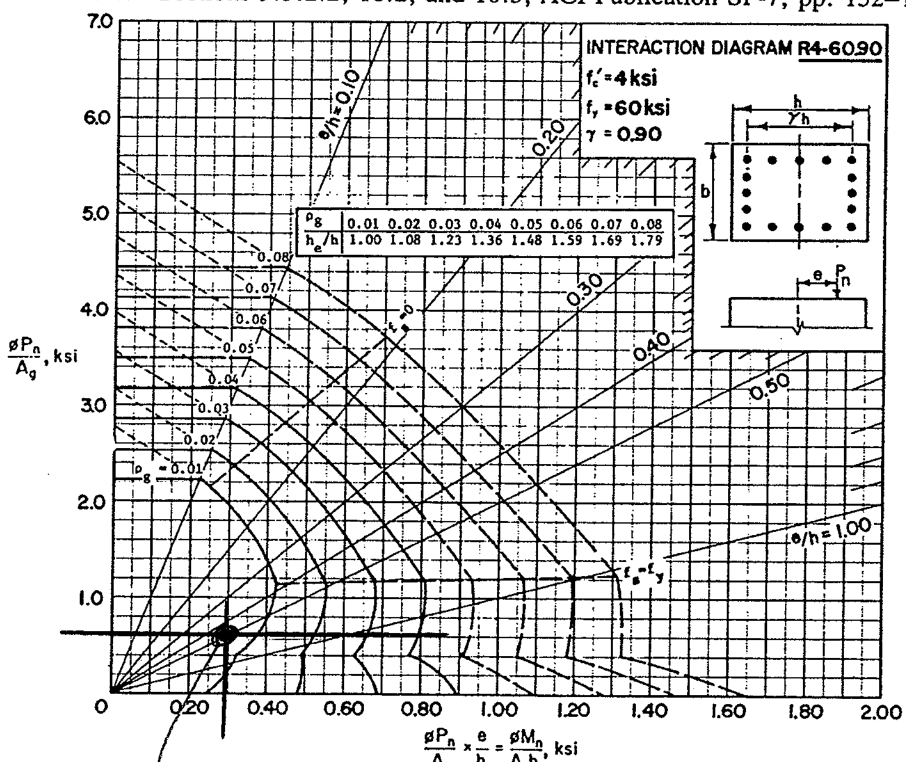
COLUMNS 7.4.3—Load-moment strength interaction diagram for R4-60.75 columns

References: ACI 318-89 Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



COLUMNS 7.4.4—Load-moment strength interaction diagram for R4-60.90 columns

References: ACI 318-89 Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182

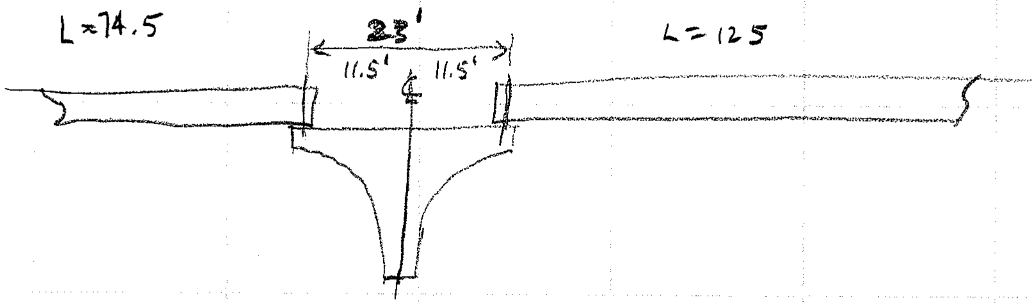


For use of these Design Aids, see Columns Examples 1–8 and 11–16.

use $\rho \approx .01$

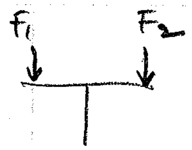
515

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)



deck wt $\approx \left(\frac{8''}{12''}\right)(25')\ .15 \approx 2.5$ k/ft

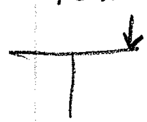
girder wt $\approx (12.26\text{ SF})\ .15 \approx 1.84$ k/ft



$F_1 \approx \frac{74.5}{2}(2.5+1.84) \approx 161.67$ k

$F_2 = \frac{125}{2}(2.5+1.84) \approx 271.25$ k

$\Delta \approx 109.6$ k



$M \approx (109.6\text{k})(11.5) \approx 1260.4$ ft k

FROM SAP 2000:

$M_{Live} \approx 1196$ ft k

$M_{DW} \approx 192$ ft k

Strength I $\approx 1.75(1196) + 1.5(192) + 1.25(1260.4)$

≈ 3956.5 ft k = 47478 in kips

for fixed pier ↗

.475 E + 08

for pinned pier from SAP2000

$M_{max} = 1480$ ~~ft k~~

Strength I $\approx 1480 + 1260.4(1.25)$

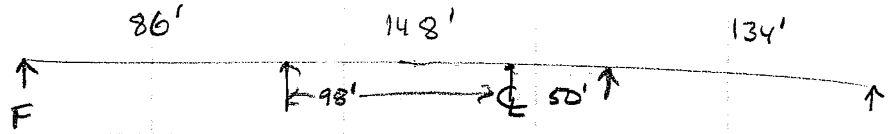
≈ 3055.5 ft k = 36666 in kips

.367 E 08

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 576 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

Temperature Movement



$L_{TOT} = 368'$

$L_{1/2} = 184'$

assume bridge expands about center

$l \approx 98'$

$HM = (98')\ 90^{\circ}(.000006)\ (12''/ft)$

$HM \approx 0.64''$

for pinned pier

max column moment ≈ 2225 ft k

max caisson moment ≈ 1150 ft k

for fixed moment pier

max column moment ≈ 1950

max caisson moment ≈ 983.3 ft k

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 57 of

```
CAISSON DESIGN PIER, Structure F-16-XP,

    1
  100     4   .330D+03   .960D+03
  .000D+00   .540D+02   .417D+06   .229D+04   .364D+07
  .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
  .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
  .960D+03   .540D+02   .417D+06   .229D+04   .364D+07
    2     8     8     0
    4   .306D+03   .690D+03   .640D+02
    3   .690D+03   .100D+04   .200D+04
  .100D+01   .000D+00
  .306D+03   .000D+00
  .306D+03   .666D-01
  .414D+03   .666D-01
  .414D+03   .694D-01
  .690D+03   .694D-01
  .690D+03   .780D-01
  .100D+04   .780D-01
  .100D+01   .000D+00   .000D+00   .000D+00
  .306D+03   .000D+00   .000D+00   .000D+00
  .306D+03   .000D+00   .320D+02   .000D+00
  .414D+03   .000D+00   .320D+02   .000D+00
  .414D+03   .000D+00   .320D+02   .000D+00
  .690D+03   .000D+00   .320D+02   .000D+00
  .690D+03   .694D+02   .000D+00   .500D-02
  .100D+04   .694D+02   .000D+00   .500D-02
     5     1   .000D+00
  .100D+01   .000D+00
  .246D+03   .000D+00
  .246D+03   .167D+05
  .270D+03   .167D+05
  .270D+03   .000D+00
    1
    4   .640D+00   .475D+08   .163D+07
    0
    1     1     0
  100   .100D-05   .360D+03
```


5/8

PIERWRST.OUT

```
*****************************************************
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*   ---------------------------------------------   *
*                                                   *
*              PREPARED ESPECIALLY FOR              *
*                                                   *
*           STATE DEPARTMENT OF HIGHWAYS            *
*                                                   *
*             DENVER, COLORADO 80222                *
*                                                   *
*                 LICENSE NO. 138                   *
*                                                   *
*****************************************************
```

PROGRAM LPILE1

CAISSON DESIGN PIER, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 960.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
960.00	54.000	.417D+06	.229D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 330.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER          =     330.00 IN
X AT THE BOTTOM OF THE LAYER       =     690.00 IN
MODULUS OF SUBGRADE REACTION       =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER          =     690.00 IN
X AT THE BOTTOM OF THE LAYER       =    1000.00 IN
MODULUS OF SUBGRADE REACTION       =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                   8 POINTS
            X,IN     WEIGHT,LBS/IN**3
            1.00        .00D+00
          306.00        .00D+00
          306.00        .67D-01
          414.00        .67D-01
          414.00        .69D-01
          690.00        .69D-01
          690.00        .78D-01
         1000.00        .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                   8 POINTS
            X,IN       C,LBS/IN**2      PHI,DEGREES     E50
            1.00       .000D+00         .000D+00        -----
          306.00       .000D+00         .000D+00        -----
          306.00       .000D+00         .320D+02        -----
          414.00       .000D+00         .320D+02        -----
          414.00       .000D+00         .320D+02        -----
          690.00       .000D+00         .320D+02        -----
          690.00       .694D+02             .000        .500D-02
         1000.00       .694D+02             .000        .500D-02
```

BOUNDARY AND LOADING CONDITIONS

```
DISTRIBUTED LOAD CURVE          5 POINTS
                      X,IN     LOAD,LBS/IN
                      1.00       .000D+00
                    246.00       .000D+00
                    246.00       .167D+05
                    270.00       .167D+05
                    270.00       .000D+00


LOADING NUMBER   1

BOUNDARY CONDITION CODE               =         4
DEFLECTION AT THE PILE HEAD           =   .640D+00 IN
MOMENT AT THE PILE HEAD               =   .475D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD           =   .163D+07 LBS
```

FINITE-DIFFERENCE PARAMETERS

```
NUMBER OF PILE INCREMENTS                                =        100
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE         =   .100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =          100
```

MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
246.00	.000D+00
246.00	.167D+05
270.00	.167D+05
270.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .640D+00 IN
MOMENT AT THE PILE HEAD = .475D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .163D+07 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.640D+00	.475D+08	-.281D+06	.000D+00	.379D+04	.152D+13
9.60	.616D+00	.448D+08	-.281D+06	.000D+00	.362D+04	.152D+13
19.20	.594D+00	.422D+08	-.281D+06	.000D+00	.344D+04	.152D+13
28.80	.575D+00	.395D+08	-.281D+06	.000D+00	.327D+04	.152D+13
38.40	.559D+00	.369D+08	-.281D+06	.000D+00	.310D+04	.152D+13
48.00	.544D+00	.342D+08	-.281D+06	.000D+00	.293D+04	.152D+13
57.60	.532D+00	.315D+08	-.281D+06	.000D+00	.275D+04	.152D+13
67.20	.521D+00	.288D+08	-.281D+06	.000D+00	.258D+04	.152D+13
76.80	.513D+00	.261D+08	-.281D+06	.000D+00	.240D+04	.152D+13
86.40	.506D+00	.235D+08	-.281D+06	.000D+00	.223D+04	.152D+13
96.00	.500D+00	.208D+08	-.281D+06	.000D+00	.206D+04	.152D+13
105.60	.496D+00	.181D+08	-.281D+06	.000D+00	.188D+04	.152D+13
115.20	.493D+00	.154D+08	-.281D+06	.000D+00	.171D+04	.152D+13
124.80	.490D+00	.127D+08	-.281D+06	.000D+00	.153D+04	.152D+13
134.40	.489D+00	.100D+08	-.281D+06	.000D+00	.136D+04	.152D+13
144.00	.488D+00	.733D+07	-.281D+06	.000D+00	.119D+04	.152D+13
153.60	.487D+00	.463D+07	-.281D+06	.000D+00	.101D+04	.152D+13
163.20	.487D+00	.194D+07	-.281D+06	.000D+00	.837D+03	.152D+13
172.80	.487D+00	-.755D+06	-.281D+06	.000D+00	.761D+03	.152D+13
182.40	.487D+00	-.345D+07	-.281D+06	.000D+00	.935D+03	.152D+13
192.00	.486D+00	-.614D+07	-.281D+06	.000D+00	.111D+04	.152D+13
201.60	.486D+00	-.884D+07	-.281D+06	.000D+00	.128D+04	.152D+13
211.20	.484D+00	-.115D+08	-.281D+06	.000D+00	.146D+04	.152D+13
220.80	.482D+00	-.142D+08	-.281D+06	.000D+00	.163D+04	.152D+13

230.40	.480D+00	-.169D+08	-.281D+06	.000D+00	.181D+04	.152D+13
240.00	.476D+00	-.196D+08	-.281D+06	.000D+00	.198D+04	.152D+13
249.60	.471D+00	-.223D+08	-.201D+06	.000D+00	.215D+04	.152D+13
259.20	.464D+00	-.234D+08	-.402D+05	.000D+00	.223D+04	.152D+13
268.80	.456D+00	-.230D+08	.120D+06	.000D+00	.220D+04	.152D+13
278.40	.447D+00	-.211D+08	.200D+06	.000D+00	.208D+04	.152D+13
288.00	.437D+00	-.192D+08	.200D+06	.000D+00	.195D+04	.152D+13
297.60	.425D+00	-.172D+08	.200D+06	.000D+00	.183D+04	.152D+13
307.20	.412D+00	-.153D+08	.200D+06	.000D+00	.170D+04	.152D+13
316.80	.399D+00	-.133D+08	.200D+06	.000D+00	.157D+04	.152D+13
326.40	.384D+00	-.114D+08	.200D+06	.000D+00	.145D+04	.152D+13
336.00	.369D+00	-.943D+07	.200D+06	-.142D+03	.132D+04	.152D+13
345.60	.353D+00	-.750D+07	.197D+06	-.353D+03	.120D+04	.152D+13
355.20	.337D+00	-.560D+07	.193D+06	-.544D+03	.107D+04	.152D+13
364.80	.321D+00	-.374D+07	.187D+06	-.714D+03	.954D+03	.152D+13
374.40	.304D+00	-.195D+07	.179D+06	-.864D+03	.838D+03	.152D+13
384.00	.287D+00	-.246D+06	.170D+06	-.992D+03	.728D+03	.152D+13
393.60	.270D+00	.137D+07	.160D+06	-.110D+04	.801D+03	.152D+13
403.20	.254D+00	.289D+07	.149D+06	-.119D+04	.899D+03	.152D+13
412.80	.237D+00	.429D+07	.138D+06	-.126D+04	.990D+03	.152D+13
422.40	.221D+00	.558D+07	.125D+06	-.130D+04	.107D+04	.152D+13
432.00	.205D+00	.675D+07	.113D+06	-.134D+04	.115D+04	.152D+13
441.60	.189D+00	.780D+07	.998D+05	-.135D+04	.122D+04	.152D+13
451.20	.174D+00	.872D+07	.868D+05	-.135D+04	.128D+04	.152D+13
460.80	.159D+00	.951D+07	.740D+05	-.133D+04	.133D+04	.152D+13
470.40	.145D+00	.102D+08	.613D+05	-.130D+04	.137D+04	.152D+13
480.00	.132D+00	.107D+08	.490D+05	-.127D+04	.141D+04	.152D+13
489.60	.119D+00	.112D+08	.370D+05	-.122D+04	.143D+04	.152D+13
499.20	.107D+00	.115D+08	.256D+05	-.116D+04	.146D+04	.152D+13
508.80	.956D-01	.117D+08	.148D+05	-.109D+04	.147D+04	.152D+13
518.40	.850D-01	.118D+08	.466D+04	-.102D+04	.148D+04	.152D+13
528.00	.750D-01	.118D+08	-.482D+04	-.951D+03	.148D+04	.152D+13
537.60	.658D-01	.117D+08	-.136D+05	-.874D+03	.147D+04	.152D+13
547.20	.573D-01	.116D+08	-.216D+05	-.796D+03	.146D+04	.152D+13
556.80	.495D-01	.114D+08	-.289D+05	-.718D+03	.145D+04	.152D+13
566.40	.424D-01	.111D+08	-.354D+05	-.641D+03	.143D+04	.152D+13
576.00	.359D-01	.107D+08	-.412D+05	-.565D+03	.140D+04	.152D+13
585.60	.301D-01	.103D+08	-.463D+05	-.493D+03	.138D+04	.152D+13
595.20	.249D-01	.983D+07	-.507D+05	-.423D+03	.135D+04	.152D+13
604.80	.204D-01	.933D+07	-.544D+05	-.358D+03	.132D+04	.152D+13
614.40	.163D-01	.880D+07	-.576D+05	-.297D+03	.128D+04	.152D+13
624.00	.129D-01	.824D+07	-.601D+05	-.242D+03	.125D+04	.152D+13
633.60	.989D-02	.766D+07	-.622D+05	-.192D+03	.121D+04	.152D+13
643.20	.738D-02	.705D+07	-.639D+05	-.148D+03	.117D+04	.152D+13
652.80	.530D-02	.644D+07	-.651D+05	-.109D+03	.113D+04	.152D+13
662.40	.360D-02	.581D+07	-.660D+05	-.766D+02	.109D+04	.152D+13
672.00	.226D-02	.517D+07	-.666D+05	-.495D+02	.105D+04	.152D+13
681.60	.124D-02	.453D+07	-.670D+05	-.278D+02	.101D+04	.152D+13
691.20	.486D-03	.389D+07	-.713D+05	-.879D+03	.964D+03	.152D+13
700.80	-.277D-04	.317D+07	-.753D+05	.510D+02	.917D+03	.152D+13
710.40	-.350D-03	.245D+07	-.719D+05	.656D+03	.870D+03	.152D+13
720.00	-.523D-03	.179D+07	-.640D+05	.999D+03	.828D+03	.152D+13
729.60	-.587D-03	.122D+07	-.537D+05	.114D+04	.791D+03	.152D+13
739.20	-.578D-03	.757D+06	-.427D+05	.114D+04	.761D+03	.152D+13
748.80	-.523D-03	.400D+06	-.322D+05	.105D+04	.738D+03	.152D+13
758.40	-.443D-03	.140D+06	-.227D+05	.908D+03	.721D+03	.152D+13
768.00	-.355D-03	-.371D+05	-.148D+05	.740D+03	.714D+03	.152D+13
777.60	-.269D-03	-.146D+06	-.855D+04	.570D+03	.721D+03	.152D+13
787.20	-.192D-03	-.201D+06	-.383D+04	.414D+03	.725D+03	.152D+13
796.80	-.127D-03	-.219D+06	-.502D+03	.279D+03	.726D+03	.152D+13
806.40	-.759D-04	-.211D+06	.165D+04	.169D+03	.725D+03	.152D+13
816.00	-.373D-04	-.188D+06	.286D+04	.842D+02	.724D+03	.152D+13
825.60	-.101D-04	-.157D+06	.337D+04	.231D+02	.722D+03	.152D+13

835.20	.765D-05	-.123D+06	.340D+04	-.178D+02	.720D+03	.152D+13
844.80	.179D-04	-.913D+05	.311D+04	-.423D+02	.718D+03	.152D+13
854.40	.226D-04	-.634D+05	.265D+04	-.542D+02	.716D+03	.152D+13
864.00	.235D-04	-.405D+05	.211D+04	-.571D+02	.714D+03	.152D+13
873.60	.219D-04	-.228D+05	.158D+04	-.540D+02	.713D+03	.152D+13
883.20	.189D-04	-.101D+05	.110D+04	-.473D+02	.712D+03	.152D+13
892.80	.154D-04	-.175D+04	.681D+03	-.389D+02	.712D+03	.152D+13
902.40	.117D-04	.300D+04	.351D+03	-.299D+02	.712D+03	.152D+13
912.00	.815D-05	.500D+04	.106D+03	-.212D+02	.712D+03	.152D+13
921.60	.495D-05	.505D+04	-.586D+02	-.130D+02	.712D+03	.152D+13
931.20	.205D-05	.389D+04	-.147D+03	-.547D+01	.712D+03	.152D+13
940.80	-.616D-06	.222D+04	-.166D+03	.167D+01	.712D+03	.152D+13
950.40	-.314D-05	.715D+03	-.116D+03	.861D+01	.712D+03	.152D+13
960.00	-.563D-05	.000D+00	.000D+00	.156D+02	.712D+03	.152D+13

OUTPUT VERIFICATION

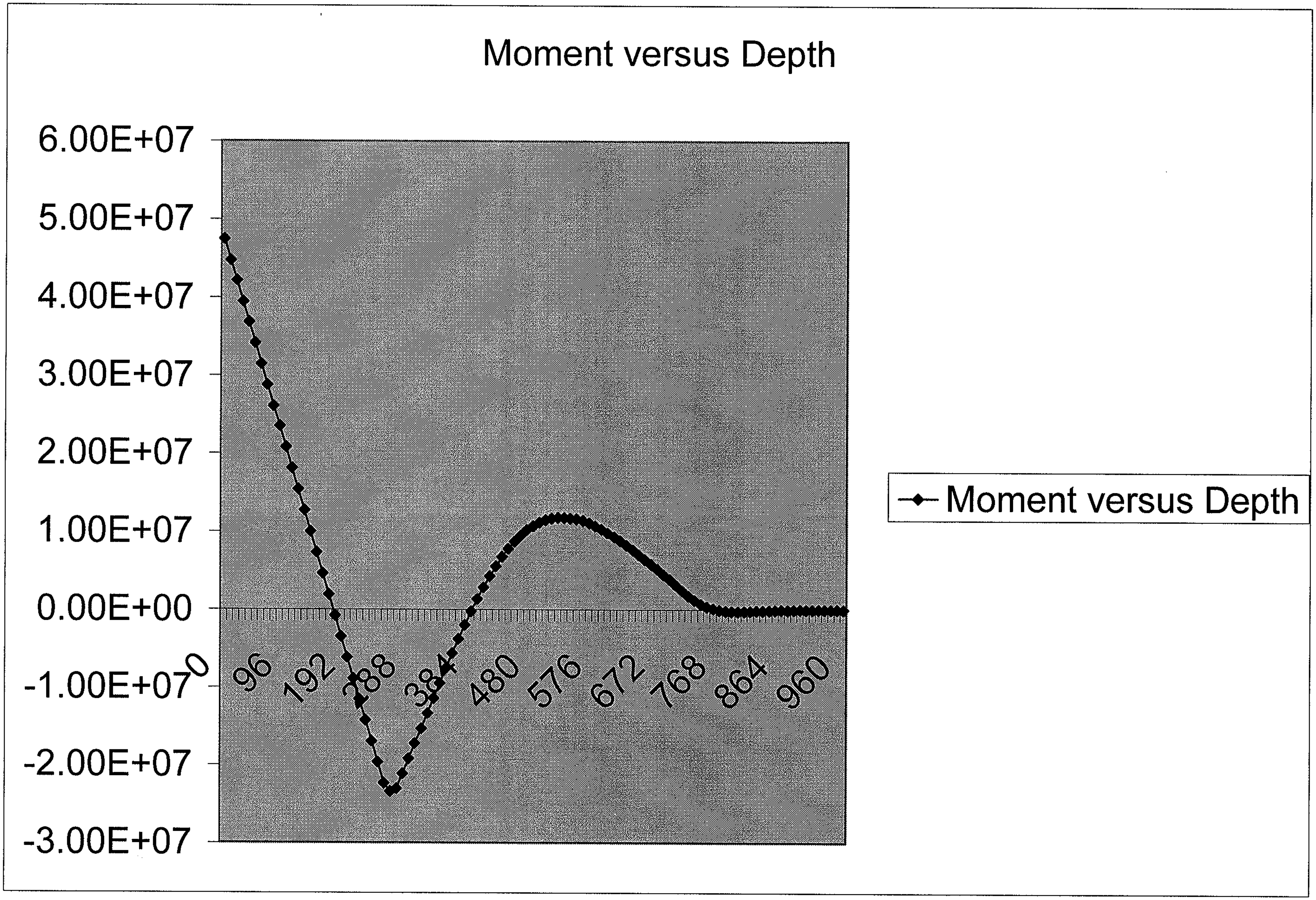
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.153D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.167D-06 LBS

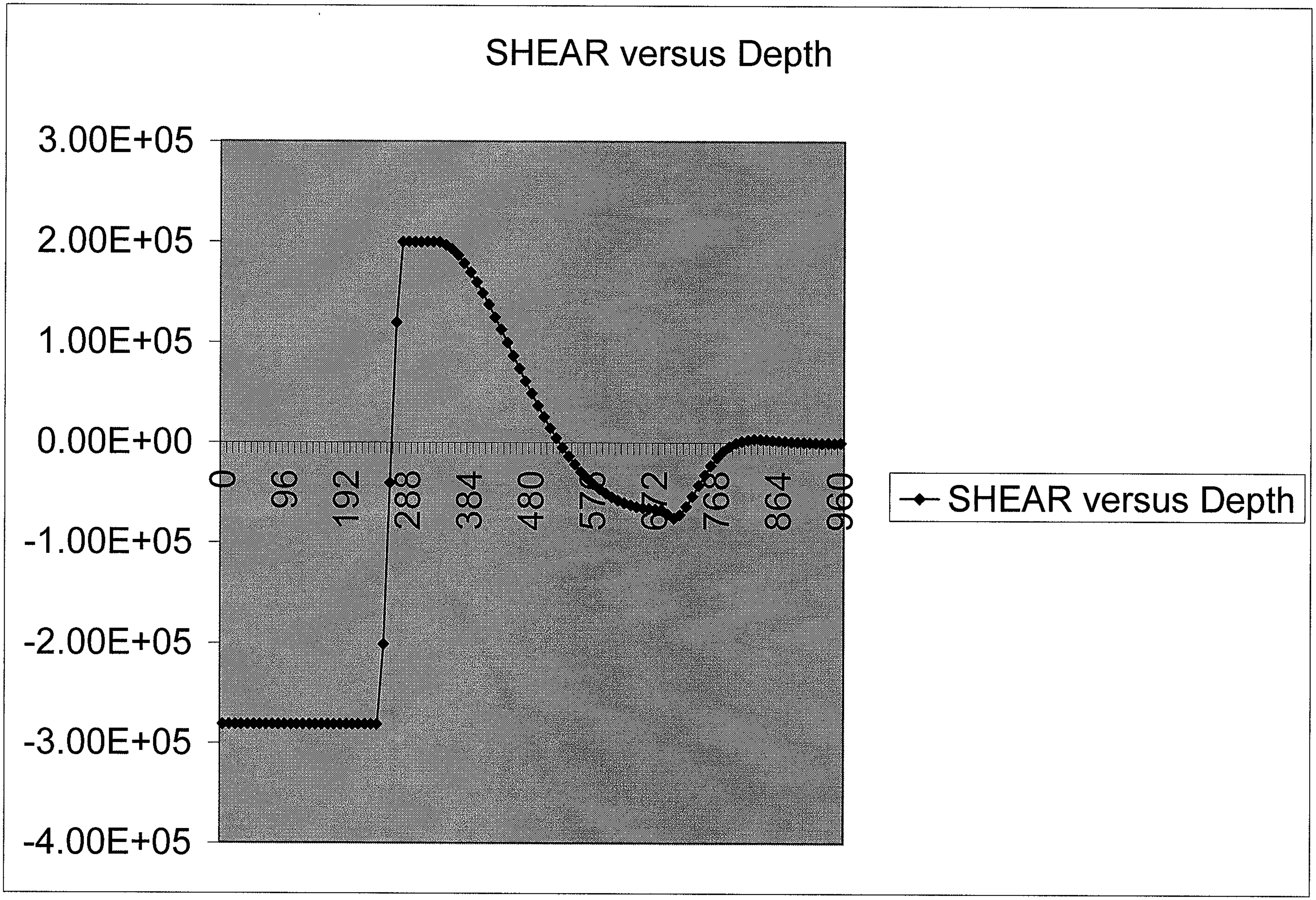
OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .640D+00 IN
MAXIMUM BENDING MOMENT = .475D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.281D+06 LBS
NO. OF ITERATIONS = 5
NO. OF ZERO DEFLECTION POINTS = 3

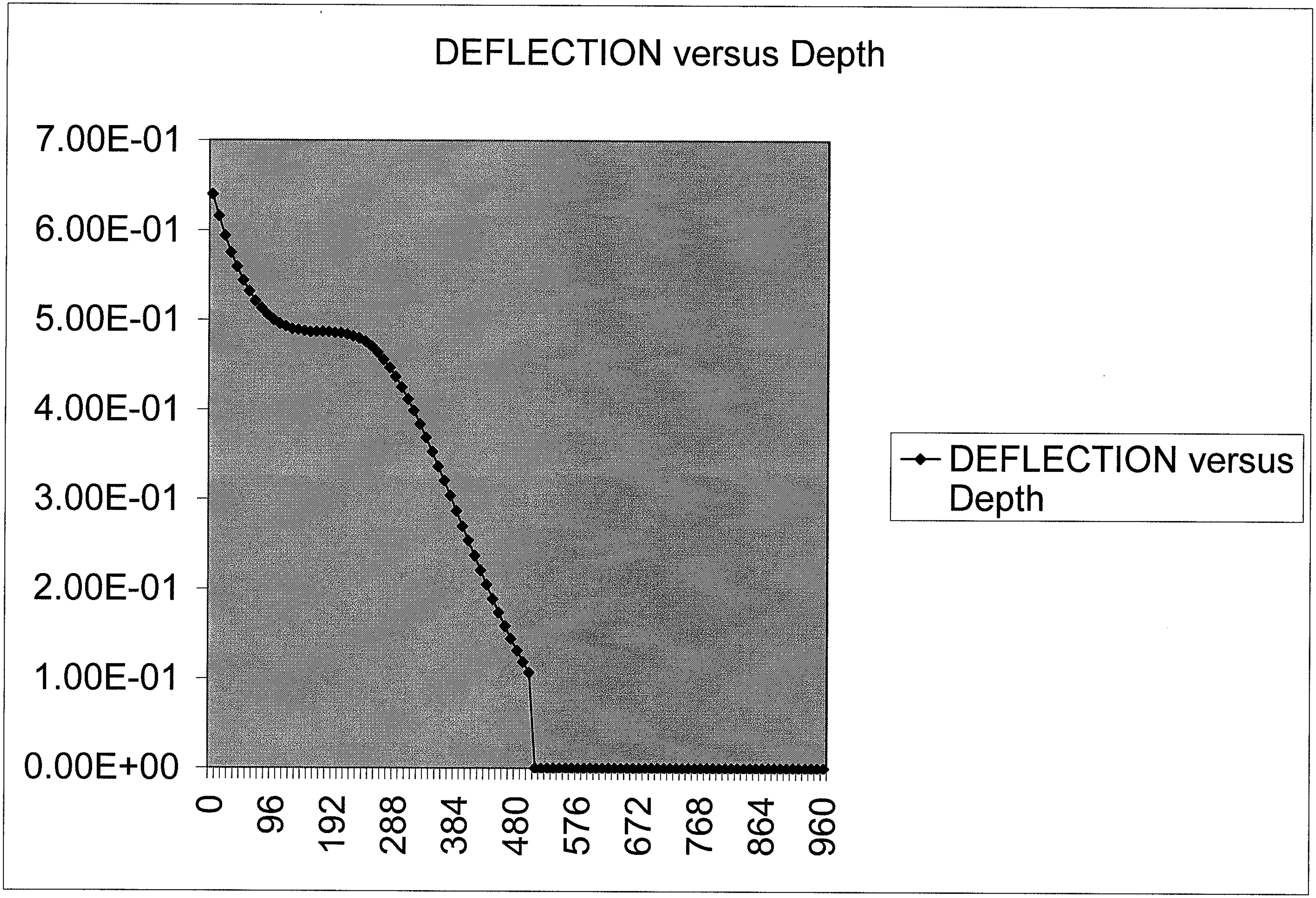
S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.6400D+00	.4750D+08	.1630D+07	.6400D+00	.4750D+08	-.2807D+06


Moment versus Depth
6.00E+07
5.00E+07
4.00E+07
3.00E+07
2.00E+07
1.00E+07
0.00E+00
-1.00E+07
-2.00E+07
-3.00E+07
0
96
192
288
384
480
576
672
768
864
960
Moment versus Depth


SHEAR versus Depth
3.00E+05
2.00E+05
1.00E+05
0.00E+00
-1.00E+05
-2.00E+05
-3.00E+05
-4.00E+05
0
96
192
288
384
480
576
672
768
864
960
SHEAR versus Depth

525


DEFLECTION versus Depth
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
1.00E-01
0.00E+00
0
96
192
288
384
480
576
672
768
864
960
DEFLECTION versus Depth

```
CAISSON DESIGN PIER, Structure F-16-XP,

   1
 100     4   .330D+03   .960D+03
 .000D+00   .540D+02   .417D+06   .229D+04   .364D+07
 .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .330D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .960D+03   .540D+02   .417D+06   .229D+04   .364D+07
   2     8     8     0
   4  .306D+03   .690D+03   .640D+02
   3  .690D+03   .100D+04   .200D+04
 .100D+01   .000D+00
 .306D+03   .000D+00
 .306D+03   .666D-01
 .414D+03   .666D-01
 .414D+03   .694D-01
 .690D+03   .694D-01
 .690D+03   .780D-01
 .100D+04   .780D-01
 .100D+01   .000D+00   .000D+00   .000D+00
 .306D+03   .000D+00   .000D+00   .000D+00
 .306D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .694D+02   .000D+00   .500D-02
 .100D+04   .694D+02   .000D+00   .500D-02
    5    1   .000D+00
 .100D+01   .000D+00
 .246D+03   .000D+00
 .246D+03   .167D+05
 .270D+03   .167D+05
 .270D+03   .000D+00
   1
   4  .640D+00   .367D+08   .163D+07
   0
   1     1     0
 100  .100D-05   .360D+03
```


527

```
*****************************************************
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*   ---------------------------------------------   *
*                                                   *
*             PREPARED ESPECIALLY FOR               *
*                                                   *
*          STATE DEPARTMENT OF HIGHWAYS             *
*                                                   *
*            DENVER, COLORADO 80222                 *
*                                                   *
*               LICENSE NO. 138                     *
*                                                   *
*****************************************************
```

PROGRAM LPILE1

CAISSON DESIGN PIER, Structure F-16-XP,

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 960.00 IN
4 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
330.00	54.000	.417D+06	.229D+04	.364D+07
960.00	54.000	.417D+06	.229D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 330.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER          =     330.00 IN
X AT THE BOTTOM OF THE LAYER       =     690.00 IN
MODULUS OF SUBGRADE REACTION       =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER          =     690.00 IN
X AT THE BOTTOM OF THE LAYER       =    1000.00 IN
MODULUS OF SUBGRADE REACTION       =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                   8 POINTS
            X,IN     WEIGHT,LBS/IN**3
            1.00       .00D+00
          306.00       .00D+00
          306.00       .67D-01
          414.00       .67D-01
          414.00       .69D-01
          690.00       .69D-01
          690.00       .78D-01
         1000.00       .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                   8 POINTS
            X,IN       C,LBS/IN**2      PHI,DEGREES      E50
            1.00       .000D+00         .000D+00         -----
          306.00       .000D+00         .000D+00         -----
          306.00       .000D+00         .320D+02         -----
          414.00       .000D+00         .320D+02         -----
          414.00       .000D+00         .320D+02         -----
          690.00       .000D+00         .320D+02         -----
          690.00       .694D+02             .000         .500D-02
         1000.00       .694D+02             .000         .500D-02


BOUNDARY AND LOADING CONDITIONS
------------------------------


DISTRIBUTED LOAD CURVE            5 POINTS
                       X,IN      LOAD,LBS/IN
                       1.00       .000D+00
                     246.00       .000D+00
                     246.00       .167D+05
                     270.00       .167D+05
                     270.00       .000D+00


LOADING NUMBER    1

BOUNDARY CONDITION CODE                  =        4
DEFLECTION AT THE PILE HEAD              =   .640D+00 IN
MOMENT AT THE PILE HEAD                  =   .367D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =   .163D+07 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                             =       100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE      =   .100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =      100
```

230.40	.564D+00	-.210D+08	-.251D+06	.000D+00	.207D+04	.152D+13
240.00	.559D+00	-.234D+08	-.251D+06	.000D+00	.223D+04	.152D+13
249.60	.552D+00	-.258D+08	-.171D+06	.000D+00	.238D+04	.152D+13
259.20	.544D+00	-.267D+08	-.106D+05	.000D+00	.244D+04	.152D+13
268.80	.535D+00	-.260D+08	.150D+06	.000D+00	.240D+04	.152D+13
278.40	.523D+00	-.238D+08	.230D+06	.000D+00	.225D+04	.152D+13
288.00	.511D+00	-.216D+08	.230D+06	.000D+00	.211D+04	.152D+13
297.60	.496D+00	-.193D+08	.230D+06	.000D+00	.196D+04	.152D+13
307.20	.481D+00	-.171D+08	.230D+06	.000D+00	.182D+04	.152D+13
316.80	.465D+00	-.149D+08	.230D+06	.000D+00	.167D+04	.152D+13
326.40	.448D+00	-.126D+08	.230D+06	.000D+00	.153D+04	.152D+13
336.00	.430D+00	-.104D+08	.229D+06	-.165D+03	.138D+04	.152D+13
345.60	.411D+00	-.817D+07	.226D+06	-.411D+03	.124D+04	.152D+13
355.20	.392D+00	-.599D+07	.221D+06	-.633D+03	.110D+04	.152D+13
364.80	.373D+00	-.386D+07	.214D+06	-.830D+03	.962D+03	.152D+13
374.40	.353D+00	-.181D+07	.205D+06	-.100D+04	.829D+03	.152D+13
384.00	.333D+00	.148D+06	.195D+06	-.115D+04	.721D+03	.152D+13
393.60	.314D+00	.200D+07	.183D+06	-.128D+04	.841D+03	.152D+13
403.20	.294D+00	.373D+07	.171D+06	-.138D+04	.954D+03	.152D+13
412.80	.275D+00	.534D+07	.157D+06	-.145D+04	.106D+04	.152D+13
422.40	.255D+00	.681D+07	.143D+06	-.151D+04	.115D+04	.152D+13
432.00	.237D+00	.815D+07	.128D+06	-.155D+04	.124D+04	.152D+13
441.60	.219D+00	.934D+07	.113D+06	-.156D+04	.132D+04	.152D+13
451.20	.201D+00	.104D+08	.983D+05	-.156D+04	.138D+04	.152D+13
460.80	.184D+00	.113D+08	.835D+05	-.154D+04	.144D+04	.152D+13
470.40	.168D+00	.120D+08	.688D+05	-.151D+04	.149D+04	.152D+13
480.00	.152D+00	.127D+08	.546D+05	-.146D+04	.153D+04	.152D+13
489.60	.137D+00	.131D+08	.408D+05	-.140D+04	.156D+04	.152D+13
499.20	.123D+00	.135D+08	.277D+05	-.134D+04	.158D+04	.152D+13
508.80	.110D+00	.137D+08	.152D+05	-.126D+04	.160D+04	.152D+13
518.40	.978D-01	.138D+08	.353D+04	-.118D+04	.161D+04	.152D+13
528.00	.863D-01	.138D+08	-.738D+04	-.109D+04	.161D+04	.152D+13
537.60	.756D-01	.137D+08	-.175D+05	-.100D+04	.160D+04	.152D+13
547.20	.658D-01	.135D+08	-.267D+05	-.914D+03	.159D+04	.152D+13
556.80	.567D-01	.132D+08	-.350D+05	-.824D+03	.157D+04	.152D+13
566.40	.485D-01	.129D+08	-.425D+05	-.734D+03	.155D+04	.152D+13
576.00	.411D-01	.124D+08	-.491D+05	-.647D+03	.152D+04	.152D+13
585.60	.344D-01	.120D+08	-.549D+05	-.563D+03	.149D+04	.152D+13
595.20	.285D-01	.114D+08	-.599D+05	-.483D+03	.145D+04	.152D+13
604.80	.232D-01	.108D+08	-.642D+05	-.408D+03	.141D+04	.152D+13
614.40	.186D-01	.102D+08	-.678D+05	-.338D+03	.137D+04	.152D+13
624.00	.146D-01	.953D+07	-.707D+05	-.275D+03	.133D+04	.152D+13
633.60	.112D-01	.885D+07	-.731D+05	-.218D+03	.128D+04	.152D+13
643.20	.833D-02	.814D+07	-.750D+05	-.167D+03	.124D+04	.152D+13
652.80	.596D-02	.741D+07	-.764D+05	-.123D+03	.119D+04	.152D+13
662.40	.403D-02	.668D+07	-.774D+05	-.858D+02	.114D+04	.152D+13
672.00	.251D-02	.594D+07	-.780D+05	-.550D+02	.110D+04	.152D+13
681.60	.135D-02	.519D+07	-.784D+05	-.305D+02	.105D+04	.152D+13
691.20	.509D-03	.443D+07	-.830D+05	-.921D+03	.999D+03	.152D+13
700.80	-.661D-04	.359D+07	-.868D+05	.122D+03	.944D+03	.152D+13
710.40	-.424D-03	.277D+07	-.824D+05	.795D+03	.891D+03	.152D+13
720.00	-.613D-03	.201D+07	-.730D+05	.117D+04	.842D+03	.152D+13
729.60	-.680D-03	.137D+07	-.610D+05	.132D+04	.800D+03	.152D+13
739.20	-.664D-03	.840D+06	-.484D+05	.132D+04	.766D+03	.152D+13
748.80	-.598D-03	.436D+06	-.363D+05	.120D+04	.740D+03	.152D+13
758.40	-.505D-03	.144D+06	-.255D+05	.103D+04	.721D+03	.152D+13
768.00	-.403D-03	-.541D+05	-.165D+05	.839D+03	.715D+03	.152D+13
777.60	-.304D-03	-.174D+06	-.942D+04	.644D+03	.723D+03	.152D+13
787.20	-.216D-03	-.235D+06	-.409D+04	.465D+03	.727D+03	.152D+13
796.80	-.142D-03	-.253D+06	-.364D+03	.312D+03	.728D+03	.152D+13
806.40	-.841D-04	-.242D+06	.203D+04	.187D+03	.727D+03	.152D+13
816.00	-.405D-04	-.214D+06	.336D+04	.914D+02	.726D+03	.152D+13
825.60	-.989D-05	-.178D+06	.391D+04	.227D+02	.723D+03	.152D+13

530

MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
246.00	.000D+00
246.00	.167D+05
270.00	.167D+05
270.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 4
DEFLECTION AT THE PILE HEAD = .640D+00 IN
MOMENT AT THE PILE HEAD = .367D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .163D+07 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
.00	.640D+00	.367D+08	-.251D+06	.000D+00	.309D+04	.152D+13
9.60	.625D+00	.343D+08	-.251D+06	.000D+00	.293D+04	.152D+13
19.20	.612D+00	.319D+08	-.251D+06	.000D+00	.278D+04	.152D+13
28.80	.602D+00	.295D+08	-.251D+06	.000D+00	.262D+04	.152D+13
38.40	.593D+00	.271D+08	-.251D+06	.000D+00	.247D+04	.152D+13
48.00	.585D+00	.247D+08	-.251D+06	.000D+00	.231D+04	.152D+13
57.60	.579D+00	.223D+08	-.251D+06	.000D+00	.216D+04	.152D+13
67.20	.575D+00	.199D+08	-.251D+06	.000D+00	.200D+04	.152D+13
76.80	.572D+00	.175D+08	-.251D+06	.000D+00	.185D+04	.152D+13
86.40	.569D+00	.151D+08	-.251D+06	.000D+00	.169D+04	.152D+13
96.00	.568D+00	.127D+08	-.251D+06	.000D+00	.153D+04	.152D+13
105.60	.567D+00	.103D+08	-.251D+06	.000D+00	.138D+04	.152D+13
115.20	.568D+00	.789D+07	-.251D+06	.000D+00	.122D+04	.152D+13
124.80	.568D+00	.548D+07	-.251D+06	.000D+00	.107D+04	.152D+13
134.40	.569D+00	.307D+07	-.251D+06	.000D+00	.910D+03	.152D+13
144.00	.570D+00	.654D+06	-.251D+06	.000D+00	.754D+03	.152D+13
153.60	.571D+00	-.176D+07	-.251D+06	.000D+00	.826D+03	.152D+13
163.20	.572D+00	-.417D+07	-.251D+06	.000D+00	.982D+03	.152D+13
172.80	.573D+00	-.658D+07	-.251D+06	.000D+00	.114D+04	.152D+13
182.40	.573D+00	-.899D+07	-.251D+06	.000D+00	.129D+04	.152D+13
192.00	.573D+00	-.114D+08	-.251D+06	.000D+00	.145D+04	.152D+13
201.60	.572D+00	-.138D+08	-.251D+06	.000D+00	.161D+04	.152D+13
211.20	.571D+00	-.162D+08	-.251D+06	.000D+00	.176D+04	.152D+13
220.80	.568D+00	-.186D+08	-.251D+06	.000D+00	.192D+04	.152D+13

835.20	.990D-05	-.139D+06	.391D+04	-.230D+02	.721D+03	.152D+13
844.80	.212D-04	-.103D+06	.356D+04	-.501D+02	.718D+03	.152D+13
854.40	.263D-04	-.711D+05	.302D+04	-.630D+02	.716D+03	.152D+13
864.00	.271D-04	-.451D+05	.240D+04	-.657D+02	.715D+03	.152D+13
873.60	.251D-04	-.251D+05	.179D+04	-.618D+02	.713D+03	.152D+13
883.20	.216D-04	-.108D+05	.123D+04	-.539D+02	.712D+03	.152D+13
892.80	.174D-04	-.145D+04	.760D+03	-.441D+02	.712D+03	.152D+13
902.40	.132D-04	.381D+04	.385D+03	-.338D+02	.712D+03	.152D+13
912.00	.917D-05	.596D+04	.108D+03	-.239D+02	.712D+03	.152D+13
921.60	.552D-05	.591D+04	-.760D+02	-.145D+02	.712D+03	.152D+13
931.20	.223D-05	.451D+04	-.174D+03	-.595D+01	.712D+03	.152D+13
940.80	-.793D-06	.257D+04	-.193D+03	.215D+01	.712D+03	.152D+13
950.40	-.366D-05	.822D+03	-.134D+03	.100D+02	.712D+03	.152D+13
960.00	-.647D-05	.000D+00	.000D+00	.179D+02	.712D+03	.152D+13

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.176D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.220D-06 LBS

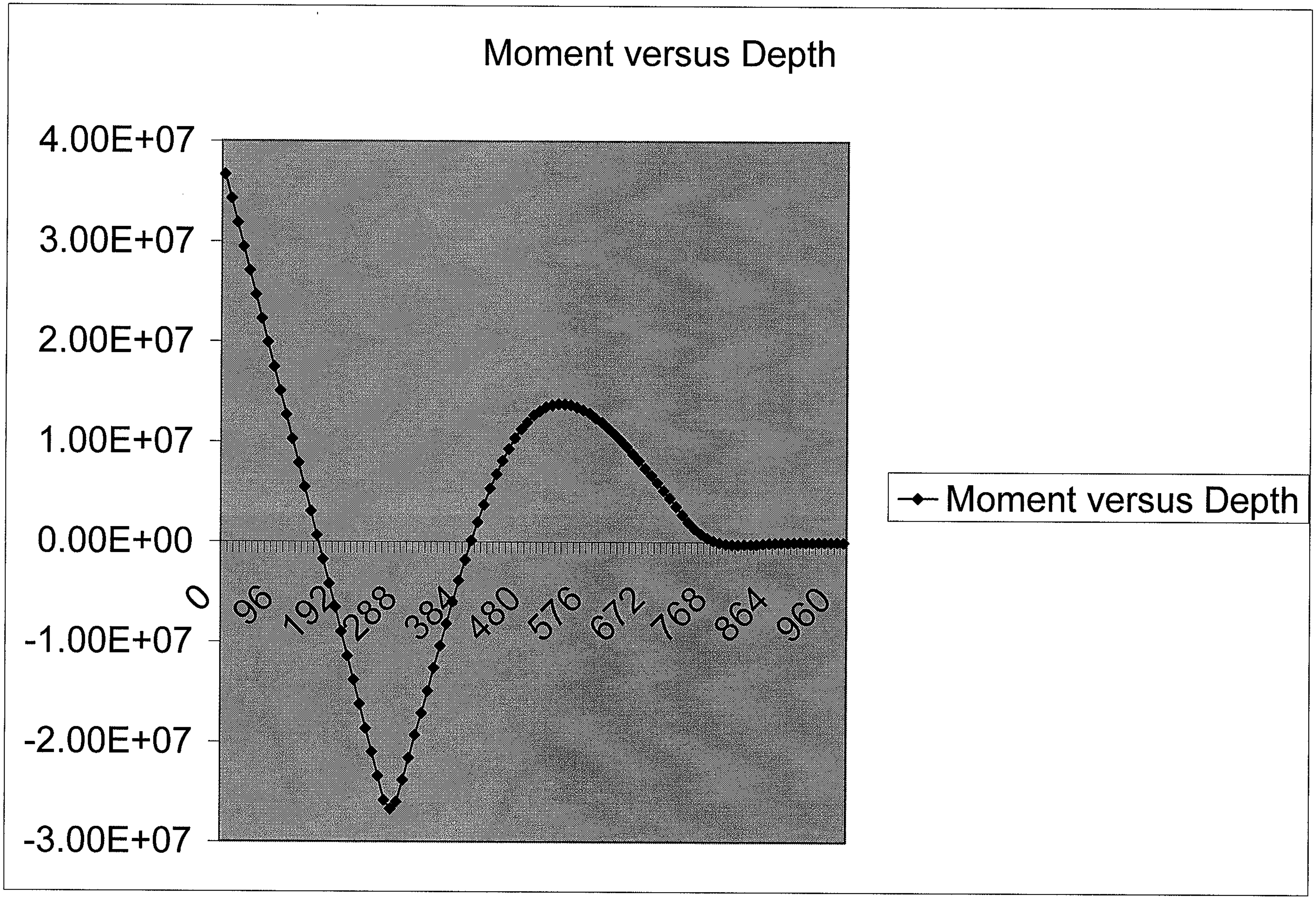
OUTPUT SUMMARY

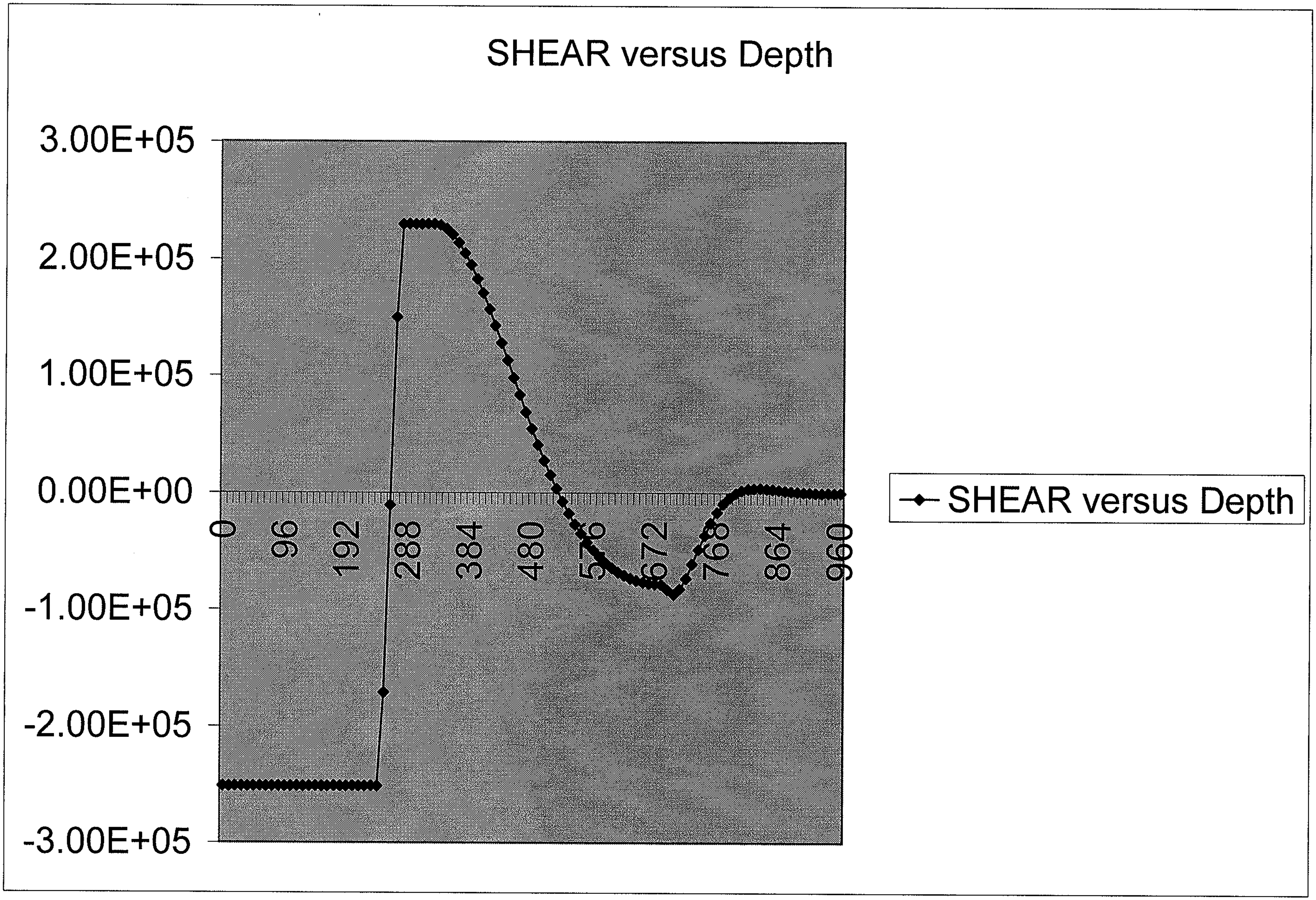
PILE-HEAD DEFLECTION = .640D+00 IN
MAXIMUM BENDING MOMENT = .367D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.251D+06 LBS
NO. OF ITERATIONS = 5
NO. OF ZERO DEFLECTION POINTS = 3

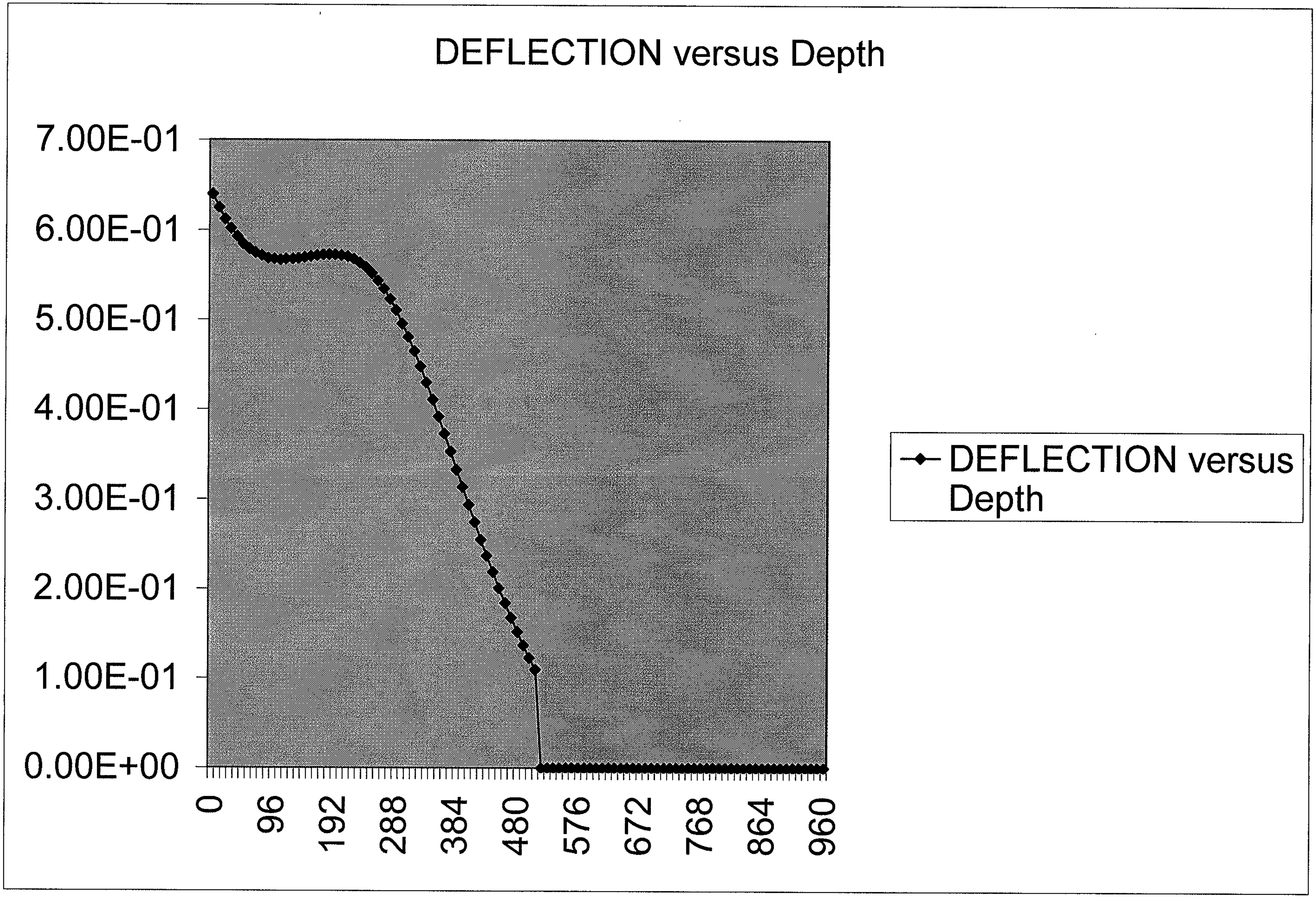
S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.6400D+00	.3670D+08	.1630D+07	.6400D+00	.3670D+08	-.2511D+06

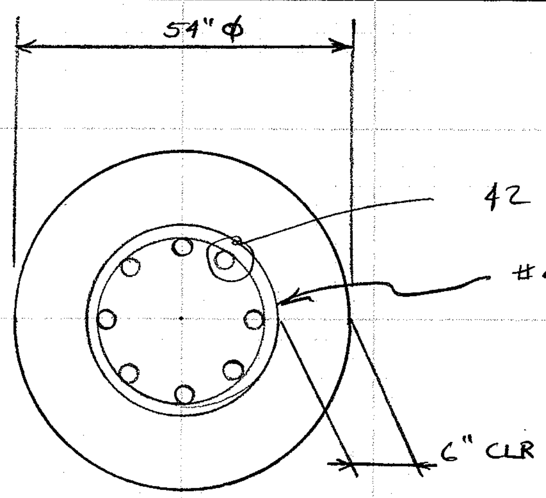
532


Moment versus Depth
4.00E+07
3.00E+07
2.00E+07
1.00E+07
0.00E+00
-1.00E+07
-2.00E+07
-3.00E+07
0
96
192
288
384
480
576
672
768
864
960
Moment versus Depth


SHEAR versus Depth
3.00E+05
2.00E+05
1.00E+05
0.00E+00
-1.00E+05
-2.00E+05
-3.00E+05
0
96
192
288
384
480
576
672
768
864
960
SHEAR versus Depth



DEFLECTION versus Depth
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
1.00E-01
0.00E+00
0
96
192
288
384
480
576
672
768
864
960
DEFLECTION versus Depth



(bundle all?)

42 - #8 Evenly spaced (bundle at quadrants to miss tie bars)

#4 @ 6" pitch

$d_e = \frac{D}{2} + \frac{D_r}{\pi}$ (C5.8.2.9-2)

$D = 54"$

$D_r = 54 - 6"(2) - 1" - 1" \approx 40"$

$d_e \approx \frac{54}{2} + \frac{40}{\pi} \approx 39.73"$

Max Shear ≈ 265 K

$S \leq 12"$ per 5.10.6.3

$d_v = .9\,d_e = .9(39.73) = 35.76"$

$.72h = .72(54) = 38.88"$

(5.8.2.9)

$\beta = 2.0$

$\theta = 45°$

$V_c = .0316\,\beta\,\sqrt{f'_c}\,b_v\,d_v$ (5.8.3.3-3)

$V_c = .0316\,(2.0)\,\sqrt{4.0}\,(54")(38.88")$

$V_c = 265.4$ K

$V_u = 265\text{k} \geq .5\,\phi(V_c + V_p)$ (5.8.2.4-1)

∴ Transverse Reinforcement Required

$A_v \geq .0316\,\sqrt{f'_c}\,\frac{b_v\,s}{f_y}$ (5.8.2.5-1)

$\geq .0316\,\sqrt{4.0}\,\frac{54"(6")}{60\text{ksi}}$

$A_v \geq .342\ in^2$

spiral or tie = 2 legs

$\#4 = (.2)\,2 = .40\ in^2$

$v_u = \frac{265\text{ksi}}{(.9)(54")(38.88)} \approx .14 < .125\,f'_c$ (5.8.2.7-1)

$S_{max} = .8\,d_v = .8(38.88) = 31.1 \leq 24"$

Use #4 spiral @ 6" pitch

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 536 of

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

42" ϕ effective steel $d_e = \frac{42}{2} + \frac{42 - 2(3") - 1 - 1}{\pi} \approx 31.82$

$d_v = .9\, d_e = .9(31.82) = 28.64"$

$d_v = .72H = .72(42) \approx 30.24"$ ← controls

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{S} \qquad (5.8.3.3\text{-}4)$$

for #4 @ 6"

$$V_s = \frac{(.4)(60)\,30.24(\cot 45 + \cot 90)\sin 90}{6"}$$

$$V_s \approx 120.96\ K$$

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3\text{-}1)$$

$$= 264.4 + 120.96 + 0$$

$V_n \approx 385.4\ K$ ← controls

$$V_n = .25\, f'_c\, b_v\, d_v + V_p \qquad (5.8.3.3\text{-}2)$$

$$= .25(40)(54)(38.88) = 2099.5\ K$$

$$V_r = .9 V_n = (.9)(385.4) \approx 346.8\ K$$

$V_R > V_u$ ✓ okay

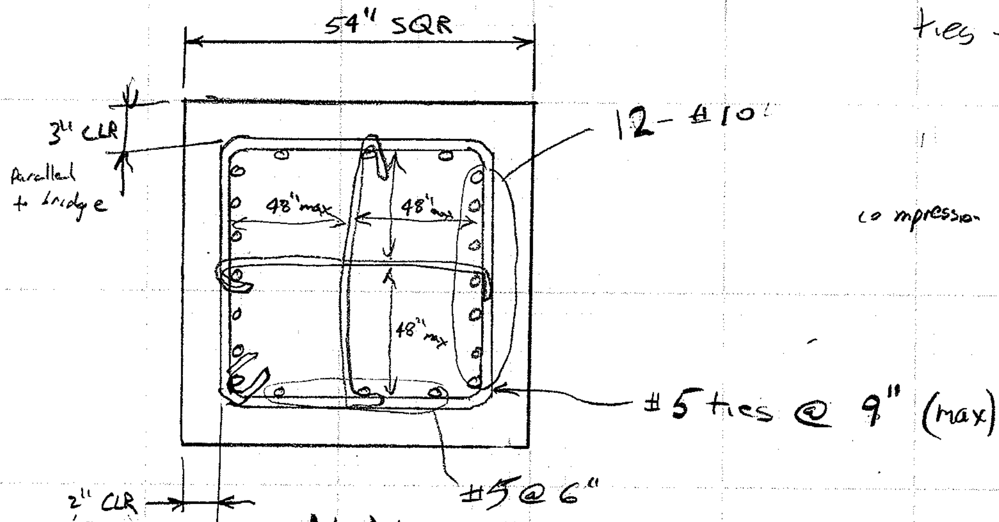
By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 537 of

13 #9

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

ties – 5.10.6.3



compression splice length $\geq .3 d_b f_y$ (5.11.2.2.1-2)

$\approx 23"$

A1-A1

Max Shear ≈ 265 K $= V_u$

$V_c = .0316\, \beta \sqrt{f'_c}\, b_v d_v$ (5.8.3.3-3)

$\beta = 2.0$

$\theta = 45°$ (5.8.3.4.1)

$V_c = .0316(2.0)\sqrt{4.5}\,(54")\, d_v$

$d_e = \dfrac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$ (5.8.2.9-2)

$d_e = d_s \approx 50.94"$

$d_v = .9 d_e = (50.94).9 \approx 45.84"$ ← controls

$d_v = .72 h = .72(54) = 38.88"$

$V_c = .0316(2.0)\sqrt{4.5}\,(54")(45.84")$

$V_c \approx 331.9$ K

$V_u \geq .5 \phi (V_c + V_p)$ (5.8.2.4-1)

$265 \geq .5(.9)(331.9) = 149.3$ K ✓ yes

∴ Transverse Reinforcement Required

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 538 of ___

$$A_v \geq .0316 \sqrt{f'_c} \frac{b_v s}{f_y} \qquad (5.8.2.5-1)$$

$$A_v \geq .0316 \sqrt{4.5} \frac{(54'')(6'')}{60 \text{ ksi}}$$

$$A_v \geq .36 \text{ in}^2 @ 6'' \qquad = .54 \text{ in}^2 @ 9''$$

#4 bar = .20 in²

2 legs (.20) = .40 in² > .36 in² ✓ ok

#4 ties @ 6" ~~@~~ (#5 ties @ 9")

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha) \sin\alpha}{s} \qquad (5.8.3.3-4)$$

$$V_s = \frac{.40 (60 \text{ ksi}) 45.84'' (\cot 45 + \cot 90) \sin 90}{6''}$$

$$V_s \approx 183.3 \text{ K}$$

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3-1)$$

$$V_n = 331.9 + 183.3 \approx 515.2 \text{ K}$$

$$V_R = \phi V_n = .9 (515.2) \approx 463.7 \text{ K}$$

$V_R > V_u$ ✓ okay

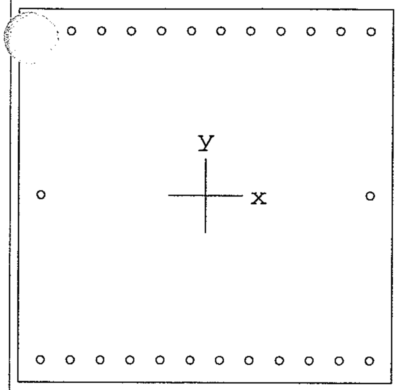
When $V_c \geq 588.9$K transverse reinforcement is not required

$$V_c = .0316 (2.0) \sqrt{4.5} (54'') d_v = 588.9$$

$$d_v \approx 81.34''$$

$$H = 81.34 + \frac{1.128''}{2} + \tfrac{1}{2}'' + 2'' \approx 84.4''$$

≈ 8' above start of flare



54.0 x 54.0 inch

f'c = 4.5 ksi

fy = 60.0 ksi

Confinement: Tied

clr cover = 2.63 in

spacing = 3.05 in

26-#10 at 1.13%

As = 33 in^2

Ix = 708588 in^4

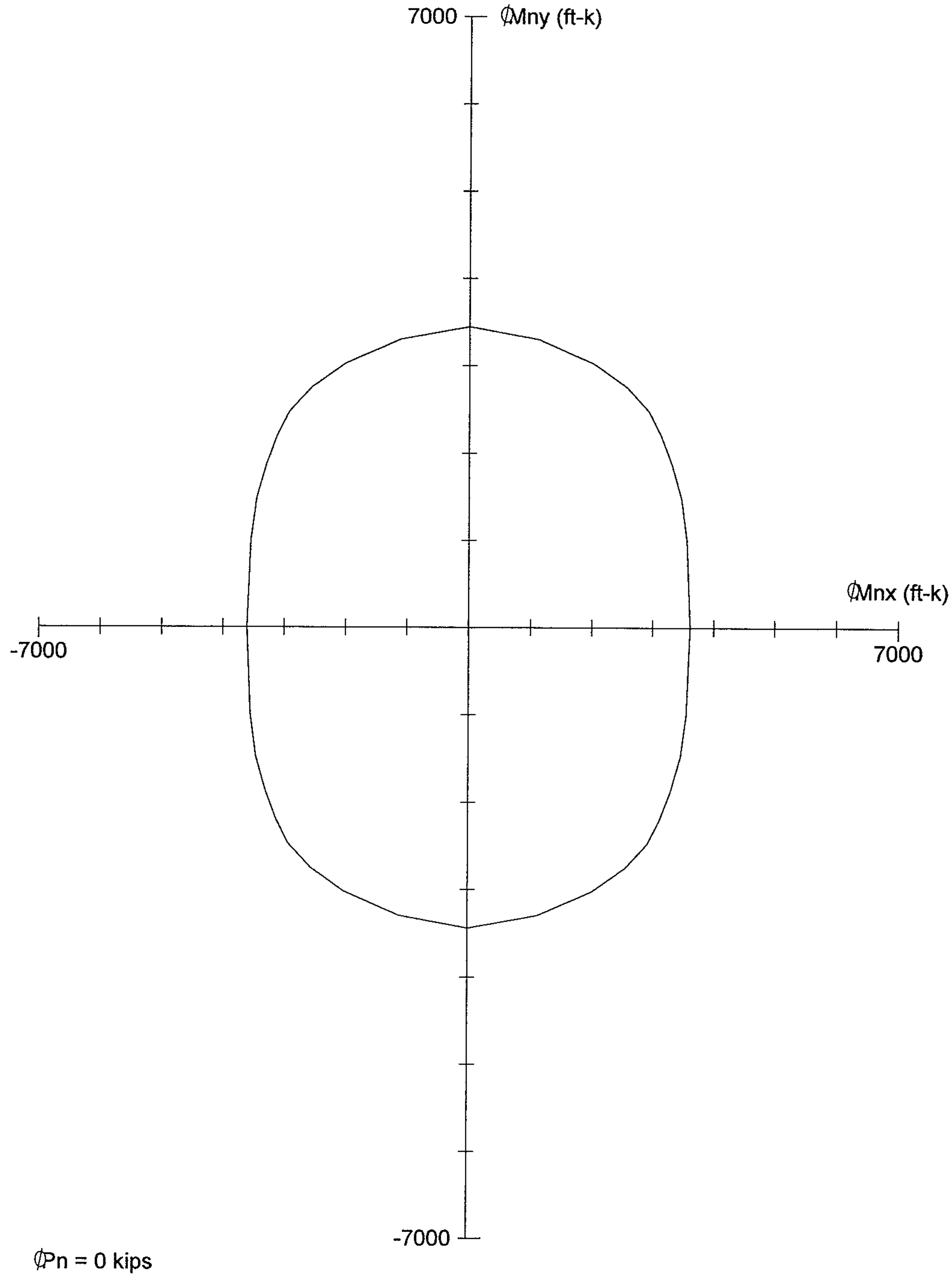
Iy = 708588 in^4

Xo = 0.00 in

Yo = 0.00 in

© 1993 PCA

PCACOL V2.30



Licensed To: Licensee name not yet specified.

File name: P:\0224\16212_~1\CALCS\PCACOL\PIERCOL.COL

Project:

Column Id:

Engineer:

Date: 03/09/09 Time: 10:52:34

Code: ACI 318-89

Units: in-lb

X-axis slenderness is not considered.

Y-axis slenderness is not considered.

Material Properties:

Ec = 4067 ksi eu = 0.003 in/in

fc = 3.83 ksi Es = 29000 ksi

Beta1 = 0.82

Stress Profile: Block

phi(c) = 0.70, phi(b) = 0.90

```
General Information:
====================
  File Name:  P:\0224\16212_~1\CALCS\PCACOL\PIERCOL.COL
  Project:                             Code: ACI 318-89
  Column:                              Units: US in-lbs
  Engineer:                            Date: 03/09/09  Time: 10:52:34

  Run Option: Design                   Short (nonslender) column
  Run Axis:   Biaxial                  Column Type: Structural

Material Properties:
====================
  f'c    = 4.5 ksi                         fy    = 60 ksi
  Ec     = 4066.84 ksi                     Es    = 29000 ksi
  fc     = 3.825 ksi                       erup = 0 in/in
  eu     = 0.003 in/in
  Stress Profile: Block                    Beta1 = 0.825

Geometry:
=========
  Rectangular: Width = 54 in               Depth = 54 in

  Gross section area, Ag =  2916 in^2
  Ix =  708588 in^4                        Xo =  0 in
  Iy =  708588 in^4                        Yo =  0 in

Reinforcement:
==============
  Rebar Database: ASTM
  Size     Diam    Area  Size     Diam    Area  Size     Diam    Area
  -------------------------------------------------------------------
     3     0.38    0.11    4      0.50    0.20    5      0.63    0.31
     6     0.75    0.44    7      0.88    0.60    8      1.00    0.79
     9     1.13    1.00   10      1.27    1.27   11      1.41    1.56
    14     1.69    2.25   18      2.26    4.00

  Confinement: Tied; phi(c) = 0.7,  phi(b) = 0.9,  a = 0.8
  #5 ties with #10 bars, #5 with larger bars.

  Layout: Rectangular
  Pattern: Sides Different  [Cover to transverse reinforcement (ties)]

  Total steel area, As = 33.02 in^2 at 1.13%

                 Top        Bottom          Left          Right
              -----------------------------------------------
  Bars        12 -#10     12 -#10        1 -#10        1 -#10
  Cover(in)         2           2             2             2
```

541

```
0000000    00000    00000    00000    00000   00
00    00  00   00  00   00  00   00  00   00  00
00    00  00   00  00   00  00   00  00   00  00
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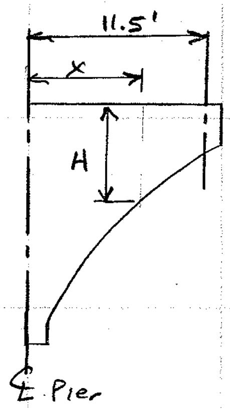
00         00000   00   00   00000    00000   00000  (TM)
```

==
Computer program for the Strength Design of Reinforced Concrete Sections
==

	Applied Loads			Computed Strength			Computed/
	P	Mx	My	P	Mx	My	Applied
Pt.	(kips)	(ft-k)	(ft-k)	(kips)	(ft-k)	(ft-k)	Ray length
1	1100	4845	500	1100	4914	509	1.014

Program completed as requested!



X	M_{STI}	$M_{serv.}$	V_{max}	H	Rebar
0	11244	7778	985	>22'-3"	13-#9
3.5	7807	5398	981	15'	14-#9
3.75	7561	5229	980	14'-3½"	14-#9
4.0	7316	5059	980	13'-8"	14-#9
4.25	7072	4890	980	13'-1"	14-#9
4.5	6827	4721	979	12'-6½"	14-#9
5.0	6338	4382	978.5	11'-9"	14-#9
6	5360	3705	978	9'-10"	13-#9
8.5	2920	2018	975	6'-9"	9-#9
11.5	~0	~0	972	4'-0"	

use 14- #9s or 11 #10s

$\ell_d \approx 50"$ for #10

for hook

$$\ell_{hb} = \frac{38\, d_b}{\sqrt{f'_c}} = \frac{38(1.27)}{\sqrt{4.5}} \qquad (5.11.2.4.1\text{-}1)$$

$$\ell_{hb} = 22.75"$$

side cover > 2.5"

$$\ell_{dh} = .7(22.75) \approx 15.9" \approx 16"$$

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 543 of ____

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (t_s)= 267.00 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 0'

13 - #9

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	7778.00	6.57
STRENGTH I	11244.00	9.52
SERVICE I	7778.00	6.57

d_s= 263.94 inches
per 5.10.8.2 AS_{temp}= 21.63 sq inches

Use # 9 at top face min. spacing = 5.67 inches
use spacing= 4.15 inches
As= 13.000 sq. inches

compressive steel:

Use # 9 at bottom face
As'= 0.00 sq. inches

M_n= 17032.58 ft-K (5.7.3.2.2-1)
M_r= 15329.32 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.58 inches
de=ds= 263.94 inches (for no prestressing)
c/de= 0.02

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 32664.99 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 14954.52 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 8.71 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.24 inches

By: Date Project no. Project code (SA#)
Chk'd: Date Structure no. Sheet of

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 180.00 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 3.5'

14 - #9

TOP STEEL

LOAD TYPE	MhTOT	ASreq'd
	ft-K	in^2
Mh (UNFACTORED)	5398.00	6.82
STRENGTH I	7807.00	9.89
SERVICE I	5398.00	6.82

ds= 176.94 inches
per 5.10.8.2 AStemp= 14.58 sq inches

Use # 9 at top face min. spacing = 5.46 inches
use spacing= 3.86 inches
As= 13.999 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

Mn= 12242.17 ft-K (5.7.3.2.2-1)
Mr= 11017.95 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.93 inches
de=ds= 176.94 inches (for no prestressing)
c/de= 0.03
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 14845.85 ft-K <--- Test 1
1.33MhTOT (max.)= 10383.31 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.38 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 171.50 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 3.75'

14-#9

TOP STEEL

LOAD TYPE	M_{hTOT}
	ft-K
Mh (UNFACTORED)	5229.00
STRENGTH I	7561.00
SERVICE I	5229.00

$AS_{req'd}$
in^2
6.94
10.06
6.94

d_s= 168.44 inches
per 5.10.8.2 AS_{temp}= 13.89 sq inches

Use # 9 at top face min. spacing = 5.36 inches
use spacing= 3.86 inches
As= 14.001 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

M_n= 11648.71 ft-K (5.7.3.2.2-1)
M_r= 10483.84 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.93 inches
de=ds= 168.44 inches (for no prestressing)
c/de= 0.03

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

$1.2*M_{cracking}$= 13476.85 ft-K <--- Test 1
$1.33M_{hTOT}$ (max.)= 10056.13 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.38 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

@ 4.0'
19 – #9

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.50	inches
Φflexure=	0.90	
Beam Thickness (ts)=	164.00	inches
b=	54.00	inches
bar diameter=	1.128	inches

TOP STEEL

LOAD TYPE	MhTOT	ASreq'd
	ft-K	in^2
Mh (UNFACTORED)	5059.00	7.03
STRENGTH I	7316.00	10.20
SERVICE I	5059.00	7.03

ds= 160.94 inches
per 5.10.8.2 ASTemp= 13.28 sq inches

Use # 9 at top face min. spacing = 5.29 inches
use spacing= 3.86 inches
As= 14.001 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

Mn= 11123.69 ft-K (5.7.3.2.2-1)
Mr= 10011.32 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)
c= 4.93 inches
de=ds= 160.94 inches (for no prestressing)
c/de= 0.03
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2
1.2*Mcracking= 12323.89 ft-K <--- Test 1
1.33MhTOT (max.)= 9730.28 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2
Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.38 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
Φflexure= 0.90
Beam Thickness (ts)= 157.00 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 4.25'
14-#9

TOP STEEL

LOAD TYPE	MhTOT	ASreq'd
	ft-K	in^2
Mh (UNFACTORED)	4890.00	7.11
STRENGTH I	7072.00	10.31
SERVICE I	4890.00	7.11

ds= 153.94 inches
per 5.10.8.2 AStemp= 12.72 sq inches

Use # 9 at top face min. spacing = 5.23 inches
use spacing= 3.86 inches
As= 14.001 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

Mn= 10633.68 ft-K (5.7.3.2.2-1)
Mr= 9570.31 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.93 inches
de=ds= 153.94 inches (for no prestressing)
c/de= 0.03
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 11294.30 ft-K <--- Test 1
1.33MhTOT (max.)= 9405.76 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.38 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (t_s)= 150.50 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 4.5'
14 - #9

TOP STEEL

LOAD TYPE	M_{hTOT}
	ft-K
M_h (UNFACTORED)	4721.00
STRENGTH I	6827.00
SERVICE I	4721.00

$AS_{req'd}$
in^2
7.17
10.40
7.17

d_s= 147.44 inches
per 5.10.8.2 AS_{temp}= 12.19 sq inches

Use # 9 at top face
use spacing= 3.86 inches
As= 14.001 sq. inches
min. spacing = 5.19 inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

M_n= 10178.66 ft-K (5.7.3.2.2-1)
M_r= 9160.79 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.93 inches
d_e=d_s= 147.44 inches (for no prestressing)
c/d_e= 0.03

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 10378.46 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 9079.91 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.38 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

By: Date Project no. . Project code (SA#) .
Chk'd: Date Structure no. . Sheet of .

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 141.00 inches
b= 54.00 inches
bar diameter= 1.128 inches

@ 5.0'
14 - #9

TOP STEEL

LOAD TYPE	M_{hTOT}
	ft-K
M_h (UNFACTORED)	4382.00
STRENGTH I	6338.00
SERVICE I	4382.00

$AS_{req'd}$
in^2
7.11
10.32
7.11

d_s= 137.94 inches
per 5.10.8.2 AS_{temp}= 11.42 sq inches

Use # 9 at top face min. spacing = 5.23 inches
use spacing= 3.86 inches
As= 13.990 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

M_n= 9506.35 ft-K (5.7.3.2.2-1)
M_r= 8555.72 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.93 inches
d_e=d_s= 137.94 inches (for no prestressing)
c/d_e= 0.04

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 9109.58 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 8429.54 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.37 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.50	inches
$\Phi_{flexure}$=	0.90	
Beam Thickness (ts)=	118.00	inches
b=	54.00	inches
bar diameter=	1.128	inches

@ 6'
13 - #9

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	3705.00	7.23
STRENGTH I	5360.00	10.50
SERVICE I	3705.00	7.23

ds= 114.94 inches
per 5.10.8.2 AS_{temp}= 9.56 sq inches

Use # 9 at top face min. spacing = 5.14 inches
use spacing= 4.15 inches
As= 13.000 sq. inches

compressive steel:
Use # 9 at bottom face
As'= 0.00 sq. inches

Mn= 7347.94 ft-K (5.7.3.2.2-1)
Mr= 6613.15 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.58 inches
de=ds= 114.94 inches (for no prestressing)
c/de= 0.04

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 6380.05 ft-K <--- Test 1
1.33MhTOT (max.)= 7128.80 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 8.71 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.24 inches

By: Date Project no. . Project code (SA#) .
Chk'd: Date Structure no. . Sheet of .

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (t_s)= 267.00 inches
b= 54.00 inches
bar diameter= 1.270 inches

@ 0'

10 #10

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
M_h (UNFACTORED)	7778.00	6.57
STRENGTH I	11244.00	9.52
SERVICE I	7778.00	6.57

d_s= 263.87 inches
per 5.10.8.2 AS_{temp}= 21.63 sq inches

Use # 10 at top face min. spacing = 7.20 inches
use spacing= 5.40 inches
As= 12.700 sq. inches

compressive steel:
Use # 10 at bottom face
As'= 0.00 sq. inches

M_n= 16638.93 ft-K (5.7.3.2.2-1)
M_r= 14975.04 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.47 inches
d_e=d_s= 263.87 inches (for no prestressing)
c/d_e= 0.02
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 32664.99 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 14954.52 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 8.51 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.26 inches

By: Date Project no. . Project code (SA#) .
Chk'd: Date Structure no. . Sheet of .

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.88 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 180.00 inches
b= 54.00 inches
bar diameter= 1.270 inches

@ 3.5'
11 - #10

TOP STEEL

LOAD TYPE	MhTOT	ASreq'd
	ft-K	in^2
Mh (UNFACTORED)	5398.00	6.84
STRENGTH I	7807.00	9.91
SERVICE I	5398.00	6.84

ds= 176.49 inches
per 5.10.8.2 AStemp= 14.58 sq inches

Use # 10 at top face min. spacing = 6.91 inches
use spacing= 4.91 inches
As= 13.970 sq. inches

compressive steel:
Use # 10 at bottom face
As'= 0.00 sq. inches

Mn= 12187.02 ft-K (5.7.3.2.2-1)
Mr= 10968.32 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.92 inches
de=ds= 176.49 inches (for no prestressing)
c/de= 0.03
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 14845.85 ft-K <--- Test 1
1.33MhTOT (max.)= 10383.31 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.36 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

@ 4.0'
11 - #10

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.88	inches
$\Phi_{flexure}$=	0.90	
Beam Thickness (ts)=	164.00	inches
b=	54.00	inches
bar diameter=	1.270	inches

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
Mh (UNFACTORED)	5059.00	7.05
STRENGTH I	7316.00	10.22
SERVICE I	5059.00	7.05

ds= 160.49 inches
per 5.10.8.2 — AS_{temp}= 13.28 sq inches

Use # 10 at top face — min. spacing = 6.70 inches
use spacing= 4.91 inches
As= 13.970 sq. inches

compressive steel:
Use # 10 at bottom face
As'= 0.00 sq. inches

Mn= 11069.40 ft-K — (5.7.3.2.2-1)
Mr= 9962.46 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.92 inches
de=ds= 160.49 inches (for no prestressing)
c/de= 0.03

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

$1.2 \cdot M_{cracking}$= 12323.89 ft-K <--- Test 1
$1.33 M_{hTOT}$ (max.)= 9730.28 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % — Use **67** % of required main reinforcement
Required As = 9.36 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.88 inches
Φflexure= 0.90
Beam Thickness (ts)= 150.50 inches
b= 54.00 inches
bar diameter= 1.270 inches

@ 4.5'
11 - #10

TOP STEEL

LOAD TYPE	MhTOT	ASreq'd
	ft-K	in^2
Mh (UNFACTORED)	4721.00	7.19
STRENGTH I	6827.00	10.43
SERVICE I	4721.00	7.19

ds= 146.99 inches
per 5.10.8.2 AStemp= 12.19 sq inches

Use # 10 at top face min. spacing = 6.57 inches
use spacing= 4.91 inches
As= 13.970 sq. inches

compressive steel:

Use # 10 at bottom face
As'= 0.00 sq. inches

Mn= 10126.41 ft-K (5.7.3.2.2-1)
Mr= 9113.77 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.92 inches
de=ds= 146.99 inches (for no prestressing)
c/de= 0.03

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 10378.46 ft-K <--- Test 1
1.33MhTOT (max.)= 9079.91 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.36 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.88 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (t_s)= 141.00 inches
b= 54.00 inches
bar diameter= 1.270 inches

@ 5.0'
11 - #10

TOP STEEL

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
M_h (UNFACTORED)	4382.00	7.14
STRENGTH I	6338.00	10.36
SERVICE I	4382.00	7.14

d_s= 137.49 inches
per 5.10.8.2 AS_{temp}= 11.42 sq inches

Use # 10 at top face min. spacing = 6.62 inches
use spacing= 4.91 inches
As= 13.970 sq. inches

compressive steel:
Use # 10 at bottom face
As'= 0.00 sq. inches

M_n= 9462.82 ft-K (5.7.3.2.2-1)
M_r= 8516.54 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.92 inches
d_e=d_s= 137.49 inches (for no prestressing)
c/d_e= 0.04
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 9109.58 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 8429.54 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%
Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 9.36 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.15 inches

REINFORCING DESIGN

GIVEN:

fy= 60.00 ksi
f'c= 4.50 ksi
COVER= 2.50 inches
$\Phi_{flexure}$= 0.90
Beam Thickness (ts)= 118.00 inches
b= 54.00 inches
bar diameter= 1.270 inches

@6'

10 #10

TOP STEEL

LOAD TYPE	M_{hTOT}
	ft-K
Mh (UNFACTORED)	3705.00
STRENGTH I	5360.00
SERVICE I	3705.00

$AS_{req'd}$
in^2
7.23
10.51
7.23

ds= 114.87 inches
per 5.10.8.2 AS_{temp}= 9.56 sq inches

Use # 10 at top face min. spacing = 6.52 inches
use spacing= 5.40 inches
As= 12.700 sq. inches

compressive steel:
Use # 10 at bottom face
As'= 0.00 sq. inches

Mn= 7177.43 ft-K (5.7.3.2.2-1)
Mr= 6459.69 ft-K
Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 4.47 inches
de=ds= 114.87 inches (for no prestressing)
c/de= 0.04
okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 6380.05 ft-K <--- Test 1
1.33MhTOT (max.)= 7128.80 ft-K <--- Test 2
Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 10.94 ft
Flange Width = 43 inches
Flange Overhang = 18 inches
Effective Length S = 8.86 ft
220 / sqrt(S) = 73.92 % Use **67** % of required main reinforcement
Required As = 8.51 sq inches
Use # 4 transverse reinforcement
min. spacing = 1.26 inches

SHEAR

$V_{u_{bearing}} \approx 971.3$ kip

$V_c \approx .0316\, B \sqrt{f'_c}\, b_v d_v$ (5.8.3.3-3)

@ bearing:

$d_e = d_s \approx 48" - 2" - 1/2" - 1.128/2 \approx 44.94"$ (for #9s)

$d_v = .9 d_e = .9(44.94) \approx 40.44"$ ← controls

$d_v = .72h = .72(48) \approx 34.56"$

$V_c \approx .0316(2.0)\sqrt{4.5}\,(54")(40.44")$

$B = 2.0$
$\theta = 45°$ (5.8.3.4.1)

$V_c \approx 292.8$ K

$V_u \geq .5\,\phi(V_c + V_p)$ (5.8.2.4-1)

∴ Transverse Reinforcement Required

$A_{v_{min}} \geq .0316\sqrt{f'_c}\,\frac{b_v s}{f_y}$ (5.8.2.5-1)

$A_v \geq .0316\sqrt{4.5}\,\frac{(54")(6")}{60 \text{ ksi}}$

$A_v \geq .36 \text{ in}^2$ @ 6"

#4 bar $= .20 \text{ in}^2$

2 legs $(.20) = .40 \text{ in}^2 > .36 \text{ in}^2$ ✓ ok

#4 ties @ 6" min max spacing = 12"

$V_n = V_c + V_s + V_p$ (5.8.3.3-1)

$V_n = .25 f'_c\, b_v d_v + V_p$ (5.8.3.3-2)

$V_n = .25(4.5)(54")(40.44) + 0 = 2456.7$ K

$V_s = V_n - V_c$

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DESIGN COMPUTATIONS (Grid)

@ bearing $(x = 11.5')$

$M_u \approx \phi$ $N_u \approx 0$ $V_p = \phi$

$V_u \approx 971.3$ $A_{ps} \approx 0$

$E_s = 29000$ ksi

$H \approx 4'$ $A_s \approx 13.97\ in^2$

$d_e \approx d_s \approx 48" - 2" - 7/8" - 1.27"/2 \approx 44.49"$ (#10s)

$d_v \approx .9 d_e = .9(44.49) \approx 40.04"$ ← controls

$d_v = .72H = .72(48) \approx 34.56"$

$$\varepsilon_s = \frac{\frac{M_u}{d_v} + .5N_u + |V_u - V_p| - A_{ps} f_{po}}{E_s A_s + E_p A_{ps}} \quad (5.8.3.4.2-4)$$

$$\varepsilon_s \approx \frac{0 + 0 + 971.3 - 0}{29000(13.97)} \approx .0024$$

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = \frac{4.8}{(1 + 750(.0024))} \approx 1.71 \quad (5.8.3.4.2-1)$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(.0024) \approx 37.4^\circ \quad (5.8.3.4.2-3)$$

$$V_c = .0316\,(1.71)\sqrt{4.5}\,(54")\,(40.04") \quad (5.8.3.3-3)$$

$V_c \approx 247.8$ K

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \quad (5.8.3.3-4)$$

assume $s = 8"$

#7 @ 8" double leg = $2.4\ in^2$

$$V_s = \frac{2.4\,(60)\,(40.04)\,(\cot 37.4 + \cot 90)\sin 90}{8"}$$

$V_s = 942.7$ K

$\ell_d \approx 21"$

$1.7\,\ell_d \approx 36"$

$V_n = 247.8 + 942.7 = 1190.5$ K

$V_r = \phi V_n = .9(1190.5) \approx 1071.4$ K

$971.3 = V_u < V_r$ ✓ okay

235.4
900

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@ 1.5' from bearing $(x = 10')$

$M_u \approx 1458.5$

$V_u \approx 973$ k

$H \approx 5'-3"$

$d_e = d_s \approx 63" - 2" - \frac{7}{8}" - \frac{1.27"}{2} \approx 59.49"$

$d_v = .9\, d_e = .9(59.49) \approx 53.54"$ ← controls

$d_v = .72\, H = .72(63) \approx 45.36"$

$$\varepsilon_s = \frac{\frac{1458.5(12)}{53.54} + 973}{29000\ (13.97)} \approx .0032$$

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = \frac{4.8}{1 + 750(.0032)} \approx 1.409$$

$\theta = 29 + 3500\varepsilon_s = 29 + 3500(.0032) \approx 40.2°$

$V_c = .0316\ (1.409)\sqrt{4.5}\ \ 54"\ (53.54")$

$V_c \approx 273.1$ K

Assume $s = 8"$

#7 double bar $= 2.4\ in^2$

$$V_s = \frac{2.4\ (60)(53.54")(\cot 40.2 + \cot 90)\sin 90}{8"}$$

$V_s = 1140.4$ K

$V_n = 273.1 + 1140.4 \approx 1143.5$ K

$V_r = \phi V_n = .9\ (1143.5) \approx 1272.1$ K

$V_u < V_r$ ✓ okay

@ 2' from bearing $(x = 9.5')$

$M_u = 1945$

$V_u = 973.5$

$H = 5'-8\frac{1}{2}''$

$d_e = d_s \approx 68.5'' - 2'' - \frac{7}{8}'' - \frac{1.27}{2} \approx 64.99''$

$d_v = .9 d_e = .9(64.99) \approx 58.49''$ ← controls

$d_v = .72 H = .72\ 68.5 \approx 49.32''$

$$\varepsilon_s = \frac{\frac{1945(12)}{58.49} + 973.5}{29000\ (13.97)} \approx .00339$$

$$\beta = \frac{4.8}{1+750\varepsilon_s} = \frac{4.8}{1+750(.00339)} \approx 1.355$$

$\theta = 29 + 3500\varepsilon_s = 29 + 3500\ (.00339) \approx 40.86°$

$V_c = .0316\ (1.355)\sqrt{4.5}\ \ 54''\ (58.49'')$

$V_c \approx 286.9\ K$

Assume $s = 12''$

#7 @ 12" double bar = 2.4 in^2

$$V_s = \frac{2.4\,(60)\,(58.49)\,(\cot 40.86 + \cot 90)\,\sin 90}{12''}$$

$V_s \approx 811.4\ K$

$V_n = 286.9 + 811.4 \approx 1098.3$

$V_r = \phi V_n = .9\ (1098.3) \approx 988.4\ K$

$V_u < V_r$ ✓ okay

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@ 2.5' from bearing ($x = 9'$)

$M_u \approx 2432.1$ ft kip

$V_u \approx 974$ kip

$H \approx 6'\text{-}2\frac{1}{2}''$

$d_e = d_s = 74.5 - 2'' - 7/8'' - \frac{1.27}{2} \approx 70.99''$

$d_v = .9 d_e = .9(70.99) \approx 63.89''$ ← controls

$d_v = .72\,h = .72\,(74.5) = 53.64''$

$$\varepsilon_s = \frac{\frac{2432.1(12)}{63.89} + 974}{29000\,(13.97)} \approx .00353$$

$$\beta = \frac{4.8}{1+750\,\varepsilon_s} = \frac{4.8}{1+750(.00353)} \approx 1.316$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(.00353) \approx 41.36^\circ$$

$$V_c \approx .0316\,(1.316)\sqrt{4.5}\,(54'')(63.89'')$$

$$V_c \approx 304.3 \text{ K}$$

Assume $s = 12''$

#7 @ 12" double bar = 2.4 in^2

$$V_s = \frac{2.4\,(60)(63.89'')(\cot 41.36 + \cot 90)\sin 90}{12''}$$

$$V_s \approx 870.8 \text{ K}$$

$$V_n = 304.3 + 870.8 = 1175.1 \text{ K}$$

$$V_r = \phi V_n = .9(1175.1) \approx 1057.6 \text{ K}$$

$V_u < V_r$ ✓ okay

@ 3' from bearing $(x = 8.5')$

$M_u \approx 2920$ ft kip

$V_u = 975$ kip

$H \approx 6'-9''$

$d_e = d_s = 81'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 77.49''$

$d_v = .9 d_e = .9(77.49) \approx 69.74''$ ← controls

$d_v = .72h = .72(81) \approx 58.32''$

$$\varepsilon_s = \frac{\frac{2920(12)}{69.74} + 975 \text{ kip}}{29000 \quad (13.97)} \approx .004$$

$$\beta = \frac{4.8}{1+750\varepsilon_s} = \frac{4.8}{1+750(.004)} \approx 1.20$$

$$\theta = 29 + 3500\varepsilon_s = 29+3500(.004) \approx 43^\circ$$

$$V_c = .0316(1.2)\sqrt{4.5}\ 54''(69.74) \approx 302.9 \text{ K}$$

Assume $s = 12''$

#7 @ 12" double bar = 2.40 in^2

$$V_s = \frac{2.40(60)\ 69.74(\cot 43 + \cot 90) \sin 90}{12}$$

$V_s \approx 897.4$ K

$V_n = 302.9 + 897.4 = 1200$ K

$V_r = \phi V_n = .9(1200) \approx 1080$ K

$V_r > V_u$ ✓ okay

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DESIGN COMPUTATIONS (Grid)

@ 3.5' from bearing ($x = 8'$)

$M_u = 3407$

$V_u \approx 975.2$

$H \approx 7'-3\tfrac{1}{4}"$

$d_e \approx d_s = 87\tfrac{1}{4}" - 2" - \tfrac{7}{8}' - \frac{1.27"}{2} \approx 83.74"$

$d_v = .9\, d_e = .9(83.74) \approx 75.36"$ ⇐ controls

$d_v = .72\, H \approx .72\,(87) \approx 62.64"$

$$\varepsilon_s = \frac{\frac{3407(12)}{75.36} + 975.2}{29000 \quad (13.99)} \approx .004 \qquad (.00375)$$

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = \frac{4.8}{1 + 750(.004)} \approx 1.2$$

$\theta = 29 + 3500\,\varepsilon_s = 29 + 3500(.004) \approx 43°$ 42.125

$V_c = .0316\ (1.2)\ \sqrt{4.5}\ 54'\ 75.36"$

$V_c \approx 327.3\ K$

Assume $S = 12"$

#7 @ 12" double bar = $2.40\,in^2$

$$V_s = \frac{2.4\,(60)(75.36)\,(\cot 43 + \cot 90)\sin 90}{12}$$

$V_s \approx 966.9\ K$

$V_n = 326.4 + 966.9 \approx 1293.3$

$V_r = \phi V_n = .9\,(1293.3) \approx 1164.0\ K$

$V_u < V_r$ ✓ okay

#6 @ 12" $A_v = 1.76\,in^2$

$V_r = 931.9 < V_u$ no good

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@ 4' from bearing $(x = 7.5')$

$M_u \approx 3895$ ft k

$V_u = 976$ K

$H \approx 7'-10''$

$d_e = d_s = 94'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 90.49''$

$d_v = .9 d_e \approx .9(90.49'') \approx 81.44''$ ← controls

$d_v = .72h \approx .72(94) \approx 67.68''$

$$\varepsilon_s = \frac{\frac{3895(12)}{81.44''} + 976}{29000\ (13.97)} \approx .004$$

$$\beta = \frac{4.8}{1 + 750\,\varepsilon_s} = \frac{4.8}{1 + 750(.004)} \approx 1.2$$

$$\theta = 29 + 3500\,\varepsilon_s = 29 + 3500(.004) \approx 43^\circ$$

$$V_c = .0316\,(1.2)\sqrt{4.5}\,(54)(81.44'')$$

$$V_c \approx 353.7 \text{ K}$$

Assume $S = 12''$

#6 @ 12'' double bar = 1.76 in^2

$$V_s \approx \frac{1.76\,(60)(81.44)\,(\cot 43^\circ + \cot 90)\,\sin 90}{12}$$

$$V_s \approx 768.5 \text{ K}$$

$$V_n = 353.7 + 768.5 \approx 1122.2 \text{ K}$$

$$V_r = \phi V_n = .9\,(1122.2) \approx 1010.0 \text{ K}$$

$V_u < V_r$ ✓ okay

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@ 5' from bearing ($x = 6.5'$)

$M_u \approx 4871$ ft k

$V_u \approx 977$ k

$H \approx 9'-2''$

$d_e \approx d_s \approx 110'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 106.49''$

$d_v = .9\, d_e = .9(106.49) \approx 95.84''$ ← controls

$d_v = .72h = .72(110) \approx 79.2''$

$$\varepsilon_s = \frac{\frac{4871(12)}{95.84} + 977}{29000\ (13.97)} \approx .004$$

$$\beta = \frac{4.8}{1+750\,\varepsilon_s} = \frac{4.8}{1+750(.004)} = 1.2$$

$\theta = 29 + 3500\,\varepsilon_s = 29 + 3500(.004) \approx 43°$

$V_c = .0316(1.2)\sqrt{4.5}\ (54'')\ 95.84''$

$V_c \approx 416.3$ K

Assume $s = 12''$

#6 @ 12" double bar = 1.76 in²

$$V_s = \frac{1.76(60)(95.84'')(\cot 43° + \cot 90)\ \sin 90}{12}$$

$V_s \approx 904.4$

$V_n = 416.3 + 904.4 = 1320.7$ k

$V_r = \phi V_n = .9(1320.7) \approx 1188.6$ k

$V_u < V_r$ ✓ okay

double #5 @ 12" $A_v = 1.24\ in^2$

$V_r \approx 948.2 < V_u$ no good

@ 5.5' from bearing (x = 6')

$M_u \approx 5360$ ft kip $\qquad M_n \approx 3705$

$V_u \approx 978$ kip

$H \approx 9'-10''$

$d_e = d_s \approx 118'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 114.49''$

$d_v = .9 d_e = .9(114.49) = 103.04''$ ← controls

$d_v = .72h = .72(118) = 84.96''$

$$\varepsilon_s = \frac{\frac{5360(12)}{103.04} + 978 \text{ kip}}{29000\ (13.97)} \approx .004$$

$$\beta = \frac{4.8}{1+750\,\varepsilon_s} = \frac{4.8}{1+750(.004)} = 1.20$$

$$\theta = 29 + 3500\,\varepsilon_s = 29 + 3500(.004) \approx 43^\circ$$

$$V_c \approx .0316\,(1.2)\sqrt{4.5}\,(54'')(103.04'')$$

$V_c = 447.6$ K

$l_d \approx 15''$
$1.7\,l_d \approx 25.5''$
use 26''

Assume $s = 6''$ #5@6''
$A_v = .62$ in^2

$$V_s \approx \frac{1.62(60)(103.04)(\cot 43^\circ + \cot 90)\sin 90}{6''}$$

$V_s \approx 685.1$

$V_n = 447.6 + 685.1 = 1132.7$

$V_r = \phi V_n = .9(1132.7) \approx 1019.4$ K

$V_u < V_r$ ✓ okay

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DESIGN COMPUTATIONS (Grid)

@ 6.5' from bearing $X = 5'$

$M_U = 6338$ ft k

$V_U = 978.5$

$H = 11'-9''$

$d_e = d_s = 141'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 137.49''$

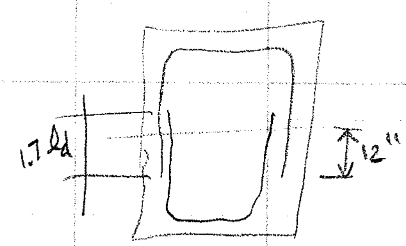
$d_v = .9 d_e = .9(137.49'') = 123.74''$ ← controls

$d_v = .72H = .72(141'') = 101.52''$

$$\varepsilon_s = \frac{\frac{6338(12)}{123.74} + 978.5}{29000\ (13.97)} \approx .004$$

$B = 1.2$

$\theta = 43°$



$V_c = .0316(1.2)\sqrt{4.5}\ (54'')\ 123.74''$

$V_c \approx 537.5$ K

Assume $s = 12'$ #6 @ 12"

$A_v = .88$ in²

#5 @ 12 double leg ≈ 1.24 in²

#4 @ 12" double leg ≈ .8 in²

$$V_s = \frac{.88(60)(123.74)(\cot 43° + \cot 90)\ \sin 90}{12''}$$

$V_s \approx 583.9$ K

$V_n = 537.5 + 583.9 \approx 1121.4$

$V_r = \phi V_n = .9(1121.4) \approx 1009.2$ K

$V_r > V_u$ ✓ okay

double #4 @ 12" $A_v = .8$ in²

$V_r \approx 961.5 < V_u$ no good

use #5 @ 12" double minimum use #5 @ 6"

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DESIGN COMPUTATIONS (Grid)

@ 7' from bearing $(x = 4.5)$

$M_u \approx 6827$

$V_u = 979$

$H = 12'-6\frac{1}{2}''$

$d_e = d_s \approx 150.5'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 146.99''$

$d_v = .9\ d_e = .9(146.99) \approx 132.29''$ ← controls

$d_v = .72h = .72(150.5) \approx 108.36''$

$$\varepsilon_s \approx \frac{\frac{6827(12)}{132.29} + 979 \text{ kip}}{29000\ (13.97)} \approx .004$$

$\beta = 1.20$

$\theta = 43°$

$V_c \approx .0316(1.2)\sqrt{4.5}(54'')(132.29'')$

$V_c \approx 574.6\ k$

$V_{s\ reqd} = 513.1$

Assume $S = 12''$ (#6 @ 12")

$A_v = .88\ in^2$

$$V_s \approx \frac{.88(60)(132.29)(\cot 43° + \cot 90)\sin 90}{12''}$$

$V_s \approx 624.2\ k$

$V_n = 574.6 + 624.2 = 1198.8$

$V_r = \phi V_n = .9(1198.8) \approx 1078.9\ k$

$V_r > V_u$ ✓ okay

double #4 legs @ 12" $A_v = .8\ in^2$ use #5 @ 6"

$V_r \approx 1027.8\ k$ ✓ okay

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@ 8' from bearing ($x = 3.5$)

$M_u = 7807$

$V_u = 981$

$H \approx 15'$

$d_e = d_s \approx 180'' - 2'' - 7/8'' - \frac{1.27}{2} \approx 176.49''$

$d_v = .9\, d_e = .9(176.49'') \approx 158.84''$ ← controls

$d_v = .72\, H = .72(180) = 129.6''$

$$\varepsilon_s = \frac{\frac{7807(12)}{158.84} + 981}{29000\;(13.97)} \approx .0039 \approx .004$$

$\beta = 1.20$

$\theta = 43°$

$V_c \approx .0316(1.2)\sqrt{4.5}\,(54'')(158.84'')$

$V_c = 690.0$ K

Assume $s = 12''$ #5@12''

$A_v = .62\ in^2$

$$V_s = \frac{.62(60)(158.84)(\cot 43° + \cot 90)\sin 90}{12''}$$

$V_s \approx 528.0$

$V_n = 690 + 528 = 1218.0$ K

$V_r = \phi V_n = .9(1218) \approx 1098$ K

$V_u < V_r$ ✓ okay

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

@ 9.25' from bearing $(x \approx 2.25')$

$M_u \approx 9050$

$V_u \approx 983.5$

$H = 22'-3"$

$d_e = d_s \approx 267" - 2" - 7/8" - \frac{1.27}{2} \approx 263.49"$

$d_v \approx .9 d_e = .9(263.49) \approx 237.14"$ ← controls

$d_v = .72H = .72(267) \approx 192.2"$

$$\varepsilon_s = \frac{\frac{9050(12)}{237.14} + 983.5}{29000\ (13.97)} \approx .0036$$

$$\beta = \frac{4.8}{1+750\varepsilon_s} = \frac{4.8}{1+750(.0036)} \approx 1.297$$

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500(.0036) \approx 41.6°$$

$V_c \approx .0316\ (1.297)\sqrt{4.5}\ (54")\ 237.14"$

$V_c \approx 1113.4\ K$

$V \geq .5\,\phi\,(V_c + V_p)$

∴ Transverse Reinforcement Required

$A_{v\,min} \approx .36\ in^2 @ 6"$

⇒ #4 ties @ 6"

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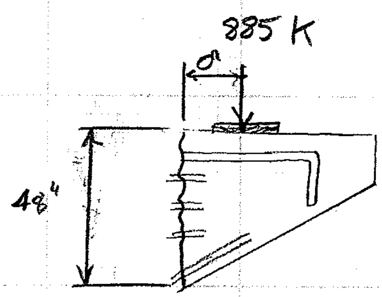
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)



$V_n = .2 f'_c b_w d_e$ (5.13.2.4.2-1)

$V_n = .2(4.5)(54")(44.94)$

$V_n \approx 2184.1$ k

$d_e \approx 44.94$

$V_n = .8 b_w d_e$ (5.13.2.4.2-2)

$V_n = .8(54)(44.94)$

$V_n = 1941.4$ k

$V_{ni} = c A_c + \mu (A_{vf} f_y \; f_y) + P_c$ (5.8.4.1-3)

$c = .40$ ksi (0 for brackets, ledges, corbels)

$\mu = 1.4$ monolithic pour

$2184.1 = .4(54)(48) + 1.4(A_v)(60 \text{ ksi})$

$A_v \approx 13.66 \text{ in}^2$

$A_{s\,top} \approx 13.97 \text{ in}^2$

A_s side $\approx$ #6 @ 6" $\Rightarrow$ $6 * .44 (2) \approx 5.28 \text{ in}^2$

A_s bottom $\approx$ #5 @ 6" $\Rightarrow$ $10 * .31 \approx 3.1 \text{ in}^2$

If treated as a corbel, $c = 0$

$A_v \approx 26.0 \text{ in}^2$

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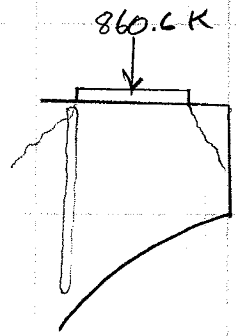
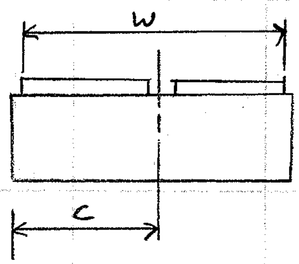
PUNCHING SHEAR



$$V_n = .125\sqrt{f'_c}\,(W + L + d_e)\,d_e \qquad (5.13.2.5.4\text{-}2)$$



$d_e \approx 2'\text{-}3''$

$L \approx 16''$

$W = 54''$

$C = 27''$

$C - .5W = 0 < d_e$

$V_n \approx .125\sqrt{4.5}\,(54 + 16 + 27)\,27$

$V_n \approx 694.5$ Kips

#7 bar

$$V_s = \frac{(1.2)\,(60\text{ksi})\,(45.84'')\,(\cot 45 + \cot 90)\sin 90}{6''}$$

$V_s \approx 549.9$ K

$V_R = .9\,[694.5 + 549.9] \approx 1120.0$ K ✓ ok

$V_R > V_u$

BEARING FORCE

MAX LOAD = 582.8 Kip

AREA ≈ 16" X 25"

$\text{FORCE} = \dfrac{582.8\text{k}}{(16)(25)} \approx 1.457$ ksi

$f'_c = 4.5$ ksi $\qquad P_n \approx .80\,[.85\,(4.5)] \approx 3.06$ ksi > force

✓ okay

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

SHEAR AT JACKING POINT

$V_{u\,service} \approx 860.6\ K$

$V_c = .0316\,\beta\sqrt{f'_c}\ b_v\ d_v$

@ bearing

$d_e = d_s \approx 27\tfrac{1}{2}'' - 2'' - \tfrac{7}{8}'' - 1.27/2 \approx 23.99''$

$d_v = .9\,d_e = .9\,(23.99) \approx 21.59''$ ← controls

$d_v = .72H = .72\,(27\tfrac{1}{2}) \approx 19.8''$

$M_u \approx \phi$

$V_u \approx 860.6K$

$$\varepsilon_s = \frac{\frac{0}{21.59} + 0 + 860.6 - 0}{29000\,(13.99)} \approx .0021$$

$$\beta = \frac{4.8}{1+750\varepsilon_s} = \frac{4.8}{1+750\,(.0021)} \approx 1.85$$

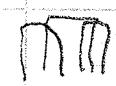
$\theta = 29 + 3500\,\varepsilon_s = 29 + 3500\,(.0021) \approx 36.35^\circ$

$V_c = .0316\,(1.85)\sqrt{4.5}\,(54'')\ 21.59''$

$V_c \approx 144.6\ K$

for $S = 6''$  6 legs

#7 triple bar = $3.6\ in^2$



$$V_s = \frac{3.6(60)\,21.59(\cot 36.35 + \cot 90)\sin 90}{6''}$$

$V_s = 1056.2$

for #8 4 legs
$A_s = 3.16$
$V_s \approx 927.0$
$V_n = 1071.7$
$V_r \approx 964.5\ K$

$V_n = 144.6 + 1056.2 = 1200$

$V_r = .9\,(1200) \approx 1080.6\ K$

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CDOT Form #1034 3/02

Side Face Steel (5.13.2.3)

$N_R = \phi f_y A_s \geq .12 b_v s$ (5.13.2.3-1)

$b_v = 54"$

$S = 12"$

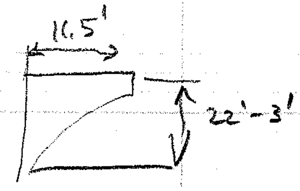
$A_s \geq \frac{.12 b_v s}{\phi f_y} = \frac{.12(54)(12")}{.9(60 ksi)}$

$A_s \geq 1.44 \ in^2/ft$

#8 @ 12" = 1.58 in^2

#7 @ 10" = 1.44 in^2

#6 @ 6" = 1.76 in^2



$A_s = 13.97 \ in^2$

for corbel design: $a_v < d$

$A_h \geq A_s/2$ distributed in 2/3 d_e

$A_h \geq \frac{13.97}{2} \approx 6.985 \ in^2$

2/3 $d_e = \frac{2(22'-3')}{3} \approx 14.83'$

$\Rightarrow .47 \ in^2/ft$

#5 @ 8"
#6 @ 11"
#7 @ 10"

@ 6.5 from bearing $d \approx 11'-9"$

$a_v < d$

2/3 $d_e \approx 2/3(137.49) \approx 91.66" \approx 7.6'$

$.455 \ in^2/ft/leg$ $\approx$ #6 @ 12"

Side Face Steel (LFD)

Per Standard Specs. 17th Edition 8.17.2.1.3

$A_{sk} \geq .012\,(d-30)$ $\qquad A_{sk\,total} \leq \frac{1}{2} A_s$

@ bearing $d = 48''$

$A_{sk} \geq .012(48-30) = .216 \text{ in}^2/\text{ft}$ each face

max spac $< d/6 < 12''$

@ bearing max spacing $= 48/6 = 8''$

#4 @ $8'' \approx .3 \text{ in}^2/\text{ft}$

min. temp steel will meet this requirement #4@6"

@ 3' from bearing $d \approx 81''$

$A_{sk} \geq .012\,(81-30) = .612 \text{ in}^2/\text{ft}$ E.F.

$\approx$ #7@ 12"

Temperature Steel

$$A_s \geq \frac{1.30\, bh}{2(b+h)\, f_y} \qquad (5.10.8-1)$$

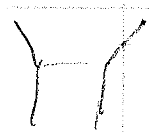
$b = 54"$

$h \approx 90"$

$$A_s \geq \frac{1.30\,(54)(90)}{2\,(54+90)\,60\text{ ksi}} = .366 \text{ in}^2$$

$\Rightarrow$ #4 @ 6" EW

at beginning of transverse flare



$h \approx 267"$ (22'-3")

$$A_s \geq \frac{1.3\,(54)(267)}{2\,(54+267)\,60} = .49 \text{ in}^2/\text{ft}$$

@ 10' long $A_s \geq \dfrac{1.3\,(54)(120)}{2\,(54+120)\,60} \approx .40 \text{ in}^2/\text{ft} \approx$ #4@6" E.W.

alternate #4 with #5 bar

$.31 + .20 = .51 \text{ in}^2/\text{ft}$

max length = 27'-6" = 330" width $\approx$ 80"

$$A_s \geq \frac{1.3\,(80")\,330"}{2\,(80+330)\,60} \approx .70 \text{ in}^2$$

#5 & #6 = .31 + .44 = .75 in²

@ 2'-2" below width $\approx$ 67" length $\approx$ 29' = 348"

$$A_s \geq \frac{1.3\,(67)(348)}{2\,(67+348)\,60} \approx .61 \text{ in}^2$$

use #5 @ 6"

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

54" column

$$A_s \geq \frac{1.3\ (54)(54)}{2\ (54+54)\ 60\text{ksi}} = .29\ \text{in}^2/\text{ft}$$

#4 @ 6" = .4 in^2/ft — use

#8 @ 18" = .79 $\text{in}^2/1.5'$ = .52 in^2/ft

#5 @ 12" = .31 in^2/ft

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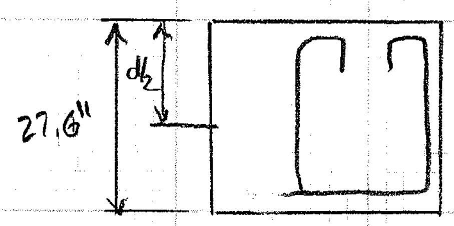
shear reinforcement anchorage (5.11.2.6.2)

$$\ell_e \geq \frac{.44 d_b f_y}{\sqrt{f'_c}} \quad (5.11.2.6.2\text{-}1)$$

$$\ell_e \geq \frac{(44)(.875'')\ 60\text{ksi}}{\sqrt{4.5}}$$

$$\ell_e \geq 10.89''$$



$$d/2 = 27.6/2 = 13.8''$$

$13.8'' - 2'' = 11.8'' > 10.89''$ ✓ okay

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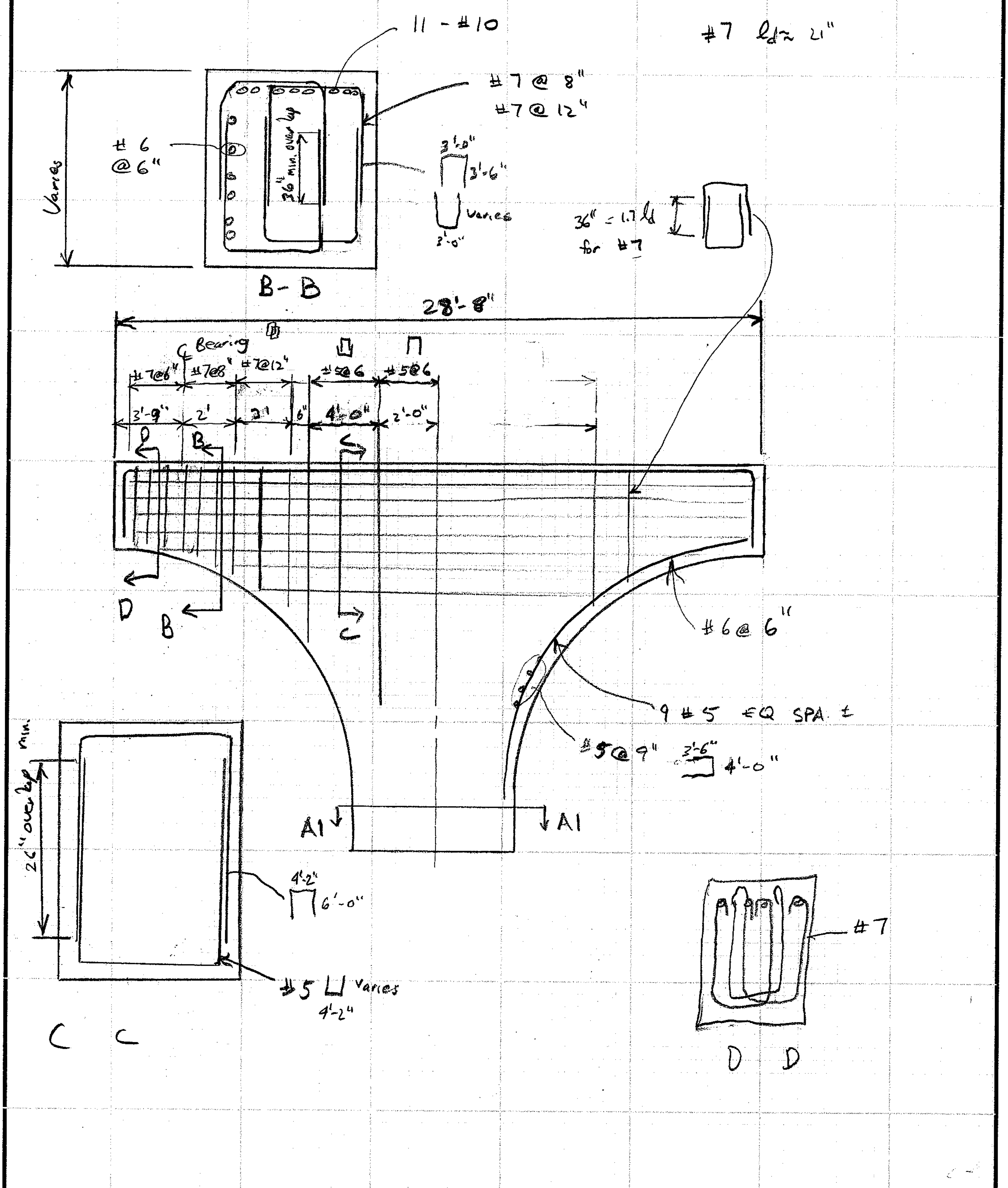
DESIGN COMPUTATIONS (Grid)



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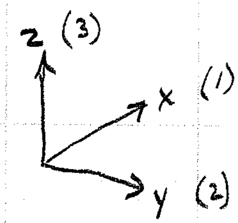
Rotational Deflection At Piers

FROM SAP2000 Model

x = bridge longitudinal
y = bridge transverse
z = elevation



Worst case deflections:

SERVICE I

$\Delta x \approx .29''$

$\Delta y \approx -2.45''$

$\Delta z \approx -.21''$

Worst case rotations:

$x = .0036$ radians

$.0036 * \frac{180}{\pi} \approx .202°$

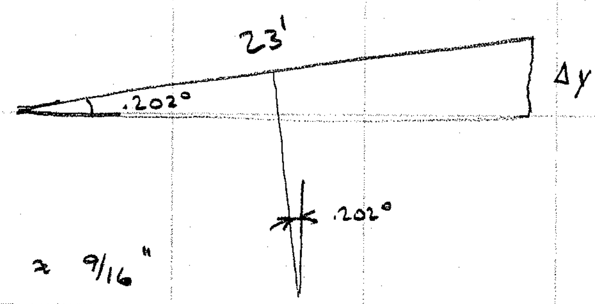
$y \approx .002$ radians

$.002 * \frac{180}{\pi} \approx .115°$

$z \approx .00017$ radians

$.00017 * \frac{180}{\pi} \approx .01°$

longitudinal deflection



$\Delta y = 23' \sin(.115)$

$\Delta y \approx .046' \approx .55'' \approx 9/16''$

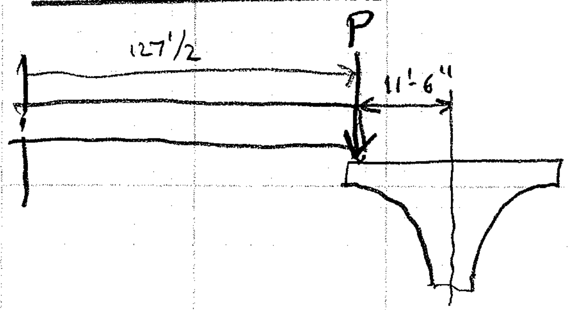
$\approx 3/16''$ per dead load

transverse deflection

$\Delta y \approx 5' \sin(.202)$

$\Delta y \approx .018' \approx .21'' = 7/32''$

offset pier load

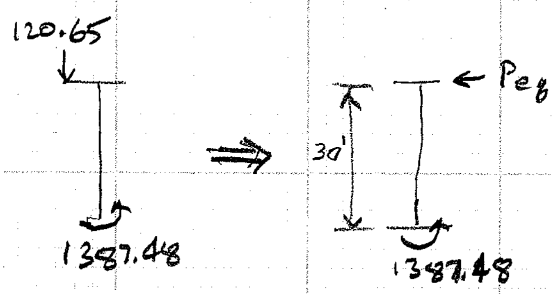


$$P_{load} = (127/2) * 1.9\ k/ft \approx 120.65\ k$$

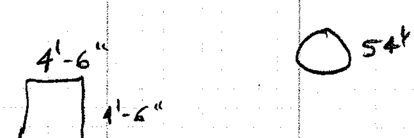
$$M = Pl = (120.65)(11.5) \approx 1387.48\ ft\,k$$

Pier designed for 11087 ft-k

okay for moment



$$P_{eq} = 1387.48/30 \approx 46.25\ k$$



$$\Delta_{max} = \frac{P_{eq.}\ l^3}{3EI}$$

$$I = \frac{(54")(54")^3}{12}$$

$$I \approx 708588$$

$$\Delta_{max} = \frac{(46.25k)(30*12)^3}{3\,(3860.8)(708588)}$$

$$E_{conc} \approx 3860.8\ ksi$$

$$\Delta_{max} \approx .263"$$

$$I_{circle} = \frac{\pi d^4}{64} \approx 417393$$

$$\Delta_{max} \approx .45"$$

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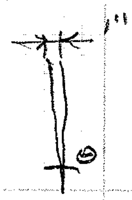
offset pier load (cont)

for point of fixity @ 45'

$$P_{eq} = \frac{1387.48}{45} = 30.8\ K$$

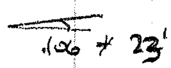
$$\Delta_{caisson} = \frac{(30.8)\left[45(12)\right]^3}{3(3860.8)(417,393)}$$

$$\Delta_{caisson} \approx 1''$$



$$\tan\theta = \frac{1''}{(45)(12)} = .00185$$

$$\theta = .106^\circ$$



$$(23')\sin(.106^\circ) \approx \underline{.51''}$$

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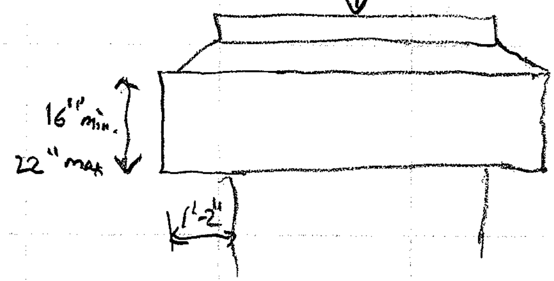
BEAM LEDGE

$V_u = 971.3\,k$



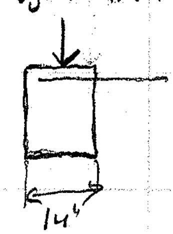
Bearing area = 73" × 16.75"

$$\sigma = \frac{971.3}{(73)(16.75)} = .8\ ksi$$

shear = $(.8\ ksi)(1'\text{-}2") = 11.2\ k/in$

$\approx 134.4\ k/ft$

$V_u = 134.4\ k/ft$



$$M_u = \frac{14"}{2}(134.4\ k/ft)$$

$M_u = 78.4\ k\,ft\ /\ ft$ of ledge

for 18" avg depth

need #7s @ 6" $A_s = .60\ in^2 \Rightarrow 1.20\ in^2/ft$

$$A_{vf} \geq \frac{.05 A_{cv}}{f_y} = \frac{.05\,(18")(12")}{60} = .18\ in^2/ft \quad \text{min.}$$

(5.8.4.4-1)

$A_n \geq N_{uc}/\phi f_y =$ (5.13.2.4.2-7)

$N_{uc} \geq .2 V_u$ (5.13.2.4.1)

$N_{uc} \geq (.2)(134.4/ft) = 26.88\ k/ft$

$A_n \geq (26.88\ k/ft)/(.9)\,60\ ksi$

$A_n \geq .50\ in^2$

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DESIGN COMPUTATIONS (Grid)

$$A_s \geq 2\,\frac{A_{vf}}{3} + A_n \qquad (5.13.2.4.2\text{-}5)$$

$$A_s \geq 2\,\frac{.18}{3} + .50\ in^2$$

$$A_s \geq .62\ in^2/ft$$

$A_s \Rightarrow$ #7 @ 6" $\Rightarrow$ 1.20 in^2/ft ✓ okay

$$A_h \geq .5\,(A_s - A_n) \qquad (5.13.2.4.2\text{-}6)$$

$$A_h \geq .5\,(.62 - .50)$$

$$A_h \geq .06\ in^2/ft$$

$$\tfrac{2}{3}d = \tfrac{2}{3}(16) = 10.67''$$

for #7 $\ell_d = \dfrac{1.25\,A_b\,f_y}{\sqrt{f'_c}} = \dfrac{1.25(.6)\,60}{\sqrt{4.5}} = 21.2''$

$$\ell_d = .4\,d_b\,f_y = .4\,(7/8)\,60 = 21''$$

top horizontal $\Rightarrow$ 1.4

$$\ell = 1.4\,(21.2) \approx 30'' \pm$$

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REINFORCING DESIGN

GIVEN:

fy=	60.00	ksi
f'c=	4.50	ksi
COVER=	2.00	inches
$\Phi_{flexure}$=	0.90	
Beam Thickness (t_s)=	18.00	inches
b=	12.00	inches
bar diameter=	0.625	inches

pier beam ledge

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K	in^2
M_h (UNFACTORED)	60.00	0.88
STRENGTH I	78.40	1.17
SERVICE I	60.00	0.88

d_s= 15.69 inches
per 5.10.8.2 AS_{temp}= 0.32 sq inches

Use # 7 at top face min. spacing = 6.16 inches
use spacing= 6.00 inches
As= 1.200 sq. inches

compressive steel:
Use # 5 at bottom face
As'= 0.00 sq. inches

M_n= 88.67 ft-K
M_r= 79.80 ft-K

Reinforcing is okay

Maximum Reinforcement per 5.7.3.3.1-1 (Ductility Check)

c= 1.90 inches
d_e=d_s= 15.69 inches (for no prestressing)
c/de= 0.12

okay - member is not overreinforced

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 32.99 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 104.27 ft-K <--- Test 2

Minimum Reinforcing is provided

Transverse Reinforcement per 9.7.3.2

Percentage of main reinforcement, 220 / sqrt(S) <= 67%

Girder Spacing = 15.25 ft
Flange Width = 21 inches
Flange Overhang = 18 inches
Effective Length S = 15.00 ft
220 / sqrt(S) = 56.80 % Use **56.804** % of required main reinforcement
Required As = 0.68 sq inches
Use # 6 transverse reinforcement
min. spacing = 7.74 inches

Punching Shear

$V_n = .125 \sqrt{f'_c}\,(W + 2L + 2d_e)\, d_e$ (5.13.2.5.4-1)

$W = 16.75''$

$L = 14''$

$d_e = 11.53''$

$V_n = .125 \sqrt{4.5}\,[16.75 + 2(14'') + 2(11.53)]\,11.53$

$V_n = 207.3$ K

$V_r = \phi V_n = .9(207.3) = 186.6$ K

Shear

$V_n = .2 f'_c\, b_w\, d_e$ (5.13.2.4.2-1)

$V_n = .8\, b_w\, d_e$ (5.13.2.4.2-2)

$V_n = .2(4.5)(16.75'')\,11.53$

$V_n = 173.8$ K

$V_n = .8(16.75)(11.53) = 154.5$

$V_r = \phi V_n = .9(154.5) = 139.1$ K

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DESIGN COMPUTATIONS (Grid)

$$A_s \geq \frac{1.30\ bh}{2(b+h)\ f_y} \qquad (5.10.8-1)$$

$$.11 \leq A_s \leq .60 \qquad (5.10.8-2)$$

$$A_s \geq \frac{(1.30)(14")(17")}{2(14+17)\ 60} = .08\ in^2/ft$$

$$A_s \geq .11\ in^2/ft$$ ← controls

for max depth = 27" ±

$$A_s \geq \frac{(1.30)(14")(27")}{2(14+27)\ 60}$$

$$A_s \geq .1\ in^2/ft$$

#4s @ 18" = .13 in²/ft ←

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CDOT Form #1034 3/02

Colorado Department of Transportation
Design Computations

11/18/2010

CAISSON LOADS

Φ (End Bearing) = 0.5

Φ (Side Shear) = 0.55

1480 sap

from SAP 2000

Service = 2000 Kips

Pier Load = [redacted] kips

Service = 379 K

Abutment Load = 525 kips

dia=	30	inches	dia=	36	inches
	2.5	feet		3	feet
A=	4.91	sf	A=	7.07	sf
P=	7.85	feet	P=	9.42	feet
Q1dia=	381.11	kips	Q1dia=	561.25	kips
Q2dia=	568.63	kips	Q2dia=	831.27	kips
Q3dia=	731.60	kips	Q3dia=	1065.94	kips
Q4dia=	796.39	kips	Q4dia=	1159.25	kips
Q5dia=	861.19	kips	Q5dia=	1252.55	kips
Q10dia=	1185.17	kips	Q10dia=	1719.08	kips

dia=	54	inches	dia=	60	inches
	~~4.5~~	~~feet~~		5	feet
A=	15.90	sf	A=	19.63	sf
P=	14.14	feet	P=	15.71	feet
Q1dia=	1309.46	kips	Q1dia=	1628.13	kips
Q2dia=	1917.00	kips	Q2dia=	2378.19	kips
Q3dia=	2445.02	kips	Q3dia=	[redacted]	kips
Q4dia=	2654.96	kips	Q4dia=	[redacted]	kips
Q5dia=	[redacted]	kips	Q5dia=	[redacted]	kips
Q10dia=	[redacted]	kips	Q10dia=	[redacted]	kips

use 5 diameter embed = 22.5'

dia=	66	inches	dia=	72	inches
	5.5	feet		6	feet
A=	23.76	sf	A=	28.27	sf
P=	17.28	feet	P=	18.85	feet
Q1dia=	1981.44	kips	Q1dia=	2369.39	kips
Q2dia=	[redacted]	kips	Q2dia=	[redacted]	kips
Q3dia=	[redacted]	kips	Q3dia=	[redacted]	kips
Q4dia=	[redacted]	kips	Q4dia=	[redacted]	kips
Q5dia=	[redacted]	kips	Q5dia=	[redacted]	kips
Q9.5dia=	[redacted]	kips	Q7dia=	[redacted]	kips

dia=	78	inches	dia=	84	inches
	6.5	feet		7	feet
A=	33.18	sf	A=	38.48	sf
P=	20.42	feet	P=	21.99	feet
Q1dia=	[redacted]	kips	Q1dia=	[redacted]	kips
Q2dia=	[redacted]	kips	Q2dia=	[redacted]	kips
Q3dia=	[redacted]	kips	Q3dia=	[redacted]	kips
Q4dia=	[redacted]	kips	Q4dia=	[redacted]	kips
Q5dia=	[redacted]	kips	Q5dia=	[redacted]	kips
Q10dia=	[redacted]	kips	Q10dia=	[redacted]	kips

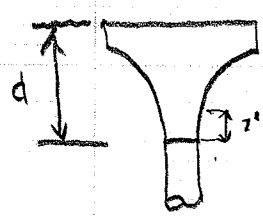
COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

PIER ELEVATIONS

* Grade Elevation assumes 3' of riprap

	PIER 2			PIER 3		
Girder #	*Grade	Pier CAP	T/C	*Grade	T/C	Pier Cap
1	184.8	209.92	180.42	184.3	181.20	210.70
2	184.1	210.57	181.07	183.6	181.70	211.20
3	182.4	211.20	181.70	184.1	182.18	211.68
4	182.9	211.82	182.32	184.1	182.63	212.13
5	183.4	211.92	182.42	184.2	182.63	212.13
6	183.4	211.51	182.01	184.1	182.00	211.50
7	183.7	211.08	181.58	184.0	181.42	210.92
8	184.9	210.64	181.14	184.1	180.81	210.31

Base Elevation 5000 ft



Top of Caisson should be 2' below grade ±

use $d \approx 29.5'$

min. depth to caisson top $\approx$ 0.5' (Girder 4 Pier 2)

max depth to caisson $\approx$ 4.3' max (Girder 1. Pier 2)

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

SEISMIC CALCULATIONS

REQ'D PER 5.10.11.2

$PGA = 6\%$ (Fig 3.10.2.1-1)

$S_s \approx 13\%$ (Fig 3.10.2.1-2)

$S_1 \approx 3.5\%$ (Fig 3.10.2.1-3)

Site Class = D (Table 3.10.3.1-1)

$F_{PGA} = 1.6$ (Table 3.10.3.2-1)

$F_a = 1.6$ (Table 3.10.3.2-2)

$F_v = 2.4$ (Table 3.10.3.2-3)

$S_{D1} = F_v S_1$ (3.10.4.2-6)

$S_D = (2.4)(.035) \approx .084 < .15$

$\Rightarrow$ Seismic Zone 1 (Table 3.10.6-1)

$A_s = F_{PGA}\ PGA$ (3.10.4.2-2)

$A_s = (1.6)(.06) = .096 > .05$

per 3.10.9.2 use .25 times Vertical Reaction for horizontal design connection force

NO SEISMIC ANALYSIS REQD. (Table 4.7.4.3.1-1)

$S_{D1} = .084 < .10$ $\Rightarrow$ ONLY REQUIREMENTS OF 3.10.9.2 ARE NECESSARY (5.10.11.2)

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

Minimum Support Length (4.7.4.4)

$N = (8 + .02L + .08H)(1 + 0.000125 S^2)$ (4.7.4.4-1)

$L = 368'$ (℄ Abut to ℄ Abut)

$H \approx 70'$

$S = 34°$

$N \approx 23.99''$

$A_s = .096 > .05$

Use 100% of N (Table 4.7.4.4-1)

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Pott, Andrew

From: Liu, Hsing-Cheng
Sent: Friday, February 13, 2009 10:36 AM
To: Pott, Andrew
Cc: Elkaissi, Jamal; Vessely, Mark; Thomas, David B
Subject: RE: Bronco Arch Geotechnical Investigation-Seismic

We do have a copy of the 2008 Interim but not the USGS 2007 Seismic Parameters CD. Since David and Mark are out of the office today, I reviewed the geology sheet and did come up with Site Class D using blow counts as well.

From: Pott, Andrew
Sent: Friday, February 13, 2009 9:58 AM
To: Liu, Hsing-Cheng
Cc: Elkaissi, Jamal; Vessely, Mark; Thomas, David B
Subject: RE: Bronco Arch Geotechnical Investigation-Seismic

I'm going through the exercise of seismic calculations for this project and wanted to verify what the Site Class Definition (Table 3.10.3.1-1) would be for this project. My guess is Site Class D based on blow counts 15<N<50. What would your determination be? Do you have a copy of all the tables? Thanks for the help.

593

BRIDGE DECK EXPANSION JOINTS (0-4")

Determining the maximum horizontal movement for expansion joint:

HM = horizontal movement
Length = contributory length in inches
TR = temperature range

$$TR_{steel} := 150°F$$

$$TR_{concrete} := 90°F$$

Ct = coefficient of thermal expansion

$$Ct_{concrete} := \frac{.000006}{°F}$$

$$Ct_{steel} := \frac{.0000065}{°F}$$

skew = skew angle
tn = empirical factor

$$tn_{concrete} := 2.0$$

$$tn_{steel} := 1.3$$

HM = Length*TR*Ct*sin(skew)*tn

Determining the "A" gap for expansion joint:

T = Structure temperature
A= Expansion Gap

$$A_{steel} := HM \bullet \left[\frac{1}{tn_{steel}} + \frac{(40 - T) \bullet °F}{TR_{steel}} \right]$$

$$A_{concrete} := 0.25 + HM \bullet \frac{(100 - T) \bullet °F}{tn_{concrete} \bullet TR_{concrete}}$$

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS

STRUCTURE F-16-XB

Determining the maximum horizontal movement for expansion joint:
concrete girder bridge:

Abutment 1 or 4: Length := **205.5**ft ≈ (368 + 20 + 20 + 1.5 + 1.5)/2 Length = **2466** in

skew := **56**deg

$HM := Length \bullet TR_{concrete} \bullet Ct_{concrete} \bullet \sin(skew) \bullet tn_{concrete}$

HM = **2.208** in < 4 inches => okay

Determining the "A" gap for expansion joint: T := **−10, 40.. 100**

$$A_{concrete}(T) := 0.25in + HM \bullet \frac{(100 - T) \bullet °F}{tn_{concrete} \bullet TR_{concrete}}$$

A(−30) = 1.845	A(−30) = 1 7/8"		2 7/8"
$A_{concrete}(-5)$ = **1.538** in	A(−5) = **"1 9/16"**		2 9/16
$A_{concrete}(30)$ = **1.109** in	A(30) = **"1 2/16"**	1 1/8"	2 1/8"
$A_{concrete}(40)$ = **0.986** in	A(40) = **"0 16/16"**	1"	2"
$A_{concrete}(50)$ = **0.863** in	A(50) = **"0 14/16"**	7/8"	1 7/8"
$A_{concrete}(60)$ = **0.741** in	A(60) = **"0 12/16"**	3/4"	1 3/4"
$A_{concrete}(70)$ = **0.618** in	A(70) = **"0 10/16"**	5/8"	1 5/8
$A_{concrete}(80)$ = **0.495** in	A(80) = **"0 8/16"**	1/2"	1 1/2"
$A_{concrete}(90)$ = **0.373** in	A(90) = **"0 6/16"**	3/8"	1 3/8"
$A_{concrete}(100)$ = **0.250** in	A(100) = **"0 4/16"**	1/4"	1 1/4"
A(120) = .005 in	A(120) = 0/16"		1"

USE

use minimum 1 1/2" @ 80°

A(0) = 1.477 in A(0) = 1 1/2" 2 1/2"

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Chk'd: Date

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STRUCTURE F-16-XB

Determining the maximum horizontal movement for expansion joint:
concrete girder bridge:

Abutment 1 or 4: $Length := 391ft$ WORST CASE FIXED AT ONE END $Length = 4692\,in$

$skew := 56deg$

$$HM := Length \bullet TR_{concrete} \bullet Ct_{concrete} \bullet \sin(skew) \bullet tn_{concrete}$$

$HM = 4.201\,in$ < 4 inches => okay

Determining the "A" gap for expansion joint: $T := -10, 40 .. 100$

$$A_{concrete}(T) := 0.25in + HM \bullet \frac{(100 - T) \bullet °F}{tn_{concrete} \bullet TR_{concrete}}$$

$A_{concrete}(-5) = 2.701\,in$	A(−5) = "2 11/16"	
$A_{concrete}(30) = 1.884\,in$	A(30) = "1 14/16"	1 7/8"
$A_{concrete}(40) = 1.650\,in$	A(40) = "1 10/16"	1 5/8"
$A_{concrete}(50) = 1.417\,in$	A(50) = "1 7/16"	
$A_{concrete}(60) = 1.184\,in$	A(60) = "1 3/16"	
$A_{concrete}(70) = 0.950\,in$	A(70) = "0 15/16"	
$A_{concrete}(80) = 0.717\,in$	A(80) = "0 11/16"	
$A_{concrete}(90) = 0.483\,in$	A(90) = "0 8/16"	1/2"
$A_{concrete}(100) = 0.250\,in$	A(100) = "0 4/16"	1/4"

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS

STRUCTURE F-16-XB

Determining the maximum horizontal movement for expansion joint:
concrete girder bridge:

Abutment 1 or 4: $\text{Length} := 205.5\text{ft}$ $\text{Length} = 2466\,\text{in}$

$\text{skew} := 56\text{deg}$

$HM := \text{Length} \cdot TR_{concrete} \cdot Ct_{concrete} \cdot \sin(\text{skew}) \cdot tn_{concrete}$

$HM = 2.208\,\text{in}$ < 4 inches => okay

Determining the "A" gap for expansion joint: $T := -10, 40..100$

$$A_{concrete}(T) := 0.25\text{in} + HM \cdot \frac{(100 - T) \cdot {}^\circ F}{tn_{concrete} \cdot TR_{concrete}}$$

			USE
$A_{concrete}(-30) = 1.845\,\text{in}$	A(−30) = "1 14/16"		2 7/8"
$A_{concrete}(30) = 1.109\,\text{in}$	A(30) = "1 2/16"	1 1/8"	2 1/8"
$A_{concrete}(40) = 0.986\,\text{in}$	A(40) = "0 16/16"	1"	2"
$A_{concrete}(50) = 0.863\,\text{in}$	A(50) = "0 14/16"	7/8"	1 7/8"
$A_{concrete}(60) = 0.741\,\text{in}$	A(60) = "0 12/16"	3/4"	1 3/4"
$A_{concrete}(70) = 0.618\,\text{in}$	A(70) = "0 10/16"	5/8"	1 5/8"
$A_{concrete}(80) = 0.495\,\text{in}$	A(80) = "0 8/16"	1/2"	1 1/2"
$A_{concrete}(90) = 0.373\,\text{in}$	A(90) = "0 6/16"	3/8"	1 3/8"
$A_{concrete}(120) = 0.005\,\text{in}$	A(120) = "0 0/16"		1"

Bridge Movement for Drains

$\ell = 205.5' = 2466$ in

Movemnt = 2466 in $*$ 60° $*$ .000006

$\approx$ .89"

use 1" gap for drains (min.)

12" pipe + 16" HDPE or 15" CPT $\approx$ 1" gap

10" pipe + 16" HDPE or 15" CPT $\approx$ 2" gap

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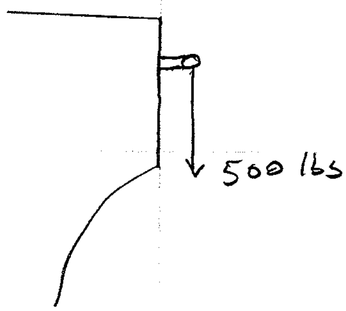
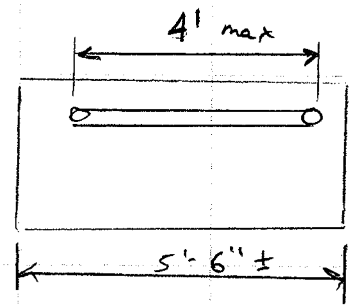
DESIGN COMPUTATIONS (Grid)

INSPECTION BAR





$M_1 = (500\ \text{lbs})(4.5" \approx .19$ ft kip

$M_2 = \frac{PL}{8} = \frac{(.5)\ 4'}{8} \approx .25$ ft kip

#8 rebar

$I = \frac{\pi d^4}{64} = \frac{\pi (1)^4}{64} = .0491\ \text{in}^4$

$\sigma = \frac{Mc}{I} = \frac{(.25\ \text{ft kip})\left(\frac{12"}{ft}\right)\ (.5\ \text{in})}{.0491\ \text{in}^4}$

$\sigma = 30.5$ Ksi

#9 rebar, $\phi = 1.128$, $I = .0795$

$\sigma = \frac{(.25)(12)\ \frac{1.128}{2}}{.0795} = 21.29$ ksi

$\sigma_{allow} = .6\ F_y = .6\ (60) \approx 36$ Ksi

use #8 rebar

A706

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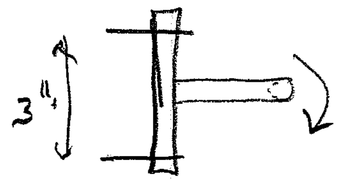
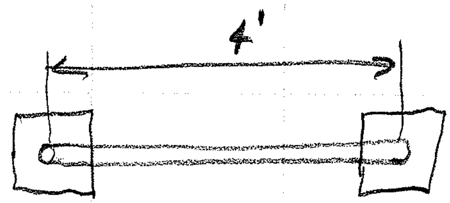
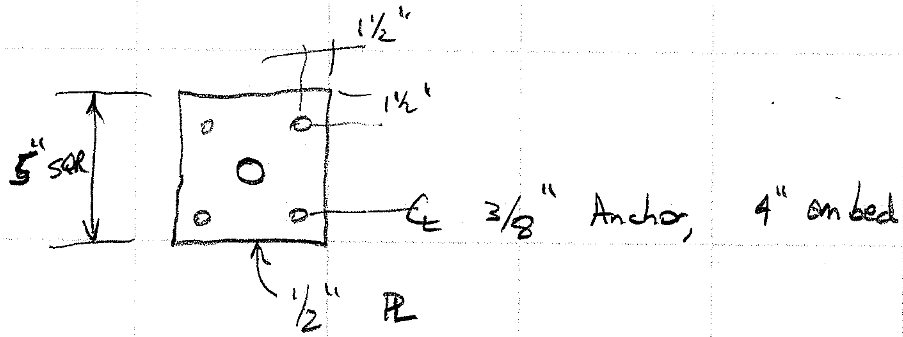
INSPECTION BAR

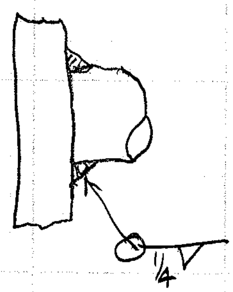




$M = .19$ ft kip

$$T = \frac{.19 \text{ ft kip}}{3''} = .76 \text{ K}$$





$$I_{weld} \approx \frac{\pi(1.5^4 - 1^4)}{64} \approx .1994$$

$$\sigma_{weld} = \frac{\left(\frac{.19 \text{ ft kip}}{2}\right)(.75'')}{.1994} = \frac{Mc}{I}$$

$$\sigma_{weld} \approx 4.29 \text{ ksi}$$

Galvanize after fabrication

$$\sigma_{allow} \approx .4(36) = 14.4 \text{ ksi}$$

✓ okay

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

BRIDGE DRAINAGE

BASED ON DESIGN MANUAL SUBSECTION 16.1 & HEC 21

DESIGN FREQUENCY = 5 Years

PLAINS REGION

NB High Point ≈ 817+96.53 El. 5219.61

Distance from High Point to end of approach slab = 820+68 - 817+96.53 ≈ 272' North End $S=1.5\%$

146' to south end $S=.8\%$

SB High Point = 417+15.88 El. 5219.74

Distance from High Point to end of approach slab = 414+62 - 417+15.88 ≈ 254' South End $S=1.9\%$

156' to North End $S=0.6\%$

assume $T_c \approx 5$ minutes

use $I = 4.6$ in/hr

Drainage Area ≈ (97')(272') = 26384 SF ≈ .61 Acres

$$Q = CIA$$

$$Q_{north} = .9(4.6)(.61) \approx 2.53 \text{ CFS}$$ 2.2

S @ end of bridge ≈ 1.5% = .015 ft/ft

$S_x = .02$ (2% crossslope)

$n = .016$

$$Q = \frac{.56}{n} S_x^{1.67} S^{.5} T^{2.67}$$

$$2.53 = \frac{.56}{.016} (.02)^{1.67} (.015)^{.5} T^{2.67}$$

$T = 9.48'$ Northbound - North end of bridge 9'

Q_{NB} southend ≈ 1.29 CFS T_{NB} southend ≈ 8.3'

Q_{SB} southend ≈ 2.53 cfs T_{SB} southend ≈ 9.1'

Q_{SB} North end ≈ 1.27 cfs T_{SB} Northend ≈ 7.3'

T ≤ shoulder width ∴ NO DRAINS REQUIRED

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BRIDGE DRAINAGE (cont)

for CDOT STANDARD VANE GRATE $w \approx 1.3'$

efficiency

$$E = 1 - (1 - w/T)^{2.67}$$

$$E = 1 - (1 - 1.3/9.48')^{2.67}$$

$$E \approx 32.6\% \text{ efficient}$$

Time of Concentration

Time from High Point to Gutter

$l \approx 97'$

$$t_o = \frac{K_w (l n)^{.6}}{(C i)^{.4}\, S^{.3}}$$ Eqn 2 HEC 21 3.2.1.3

assume $i = 4$

$S = .02$

$$t_o \approx \frac{(.93)[(97)(.016)]^{.6}}{[.9(4)]^{.4}(.02)^{.3}}$$

$$t_o \approx 2.35 \text{ min}$$

Time from Gutter to end of bridge

$l \approx 272'$ $S \approx .75\%$ avg

$$t_o \approx \frac{(.93)[272(.016)]^{.6}}{[.9(4)]^{.4}(.0075)^{.3}}$$

$$t_o \approx 5.84 \text{ min}$$

total $t_c \approx 8.2$ min

$I = 3.9$ in/hr ≈ 4 assumption okay

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

BRIDGE DRAINAGE (cont.)

Refined Qs & Ts

~ max for driving

$I = 4$ in/hr $A_{NB} \approx .53$ (from CAD)

$Q_{NB} \approx .9(4)(.53) \approx 1.9$ CFS

Northend

$T \approx 8.6'$

PHASE 1 CONSTRUCTION - PHASE 2 TRAVELWAY

$A \approx (272')(\sim 60') \approx 16320$ SF $\approx .37$ Acres

$Q = (.9)(4.3 \text{ in/hr})\ .37$

$Q \approx 1.45$ CFS (worst case)

$T \approx 7.7'$ shoulder = 4' encroaches on lane

BEST CASE:

$A \approx 272'(43') \approx 11696$ SF $\approx .27$ Acres

$Q = (.9)(4.3)(.27) \approx 1.04$ CFS

$T \approx 6.79'$

max $Q \approx .25$ CFS

max length $\approx 61.8'$

See CONSTRUCTION FLOWS calculations

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PLAINS REGION

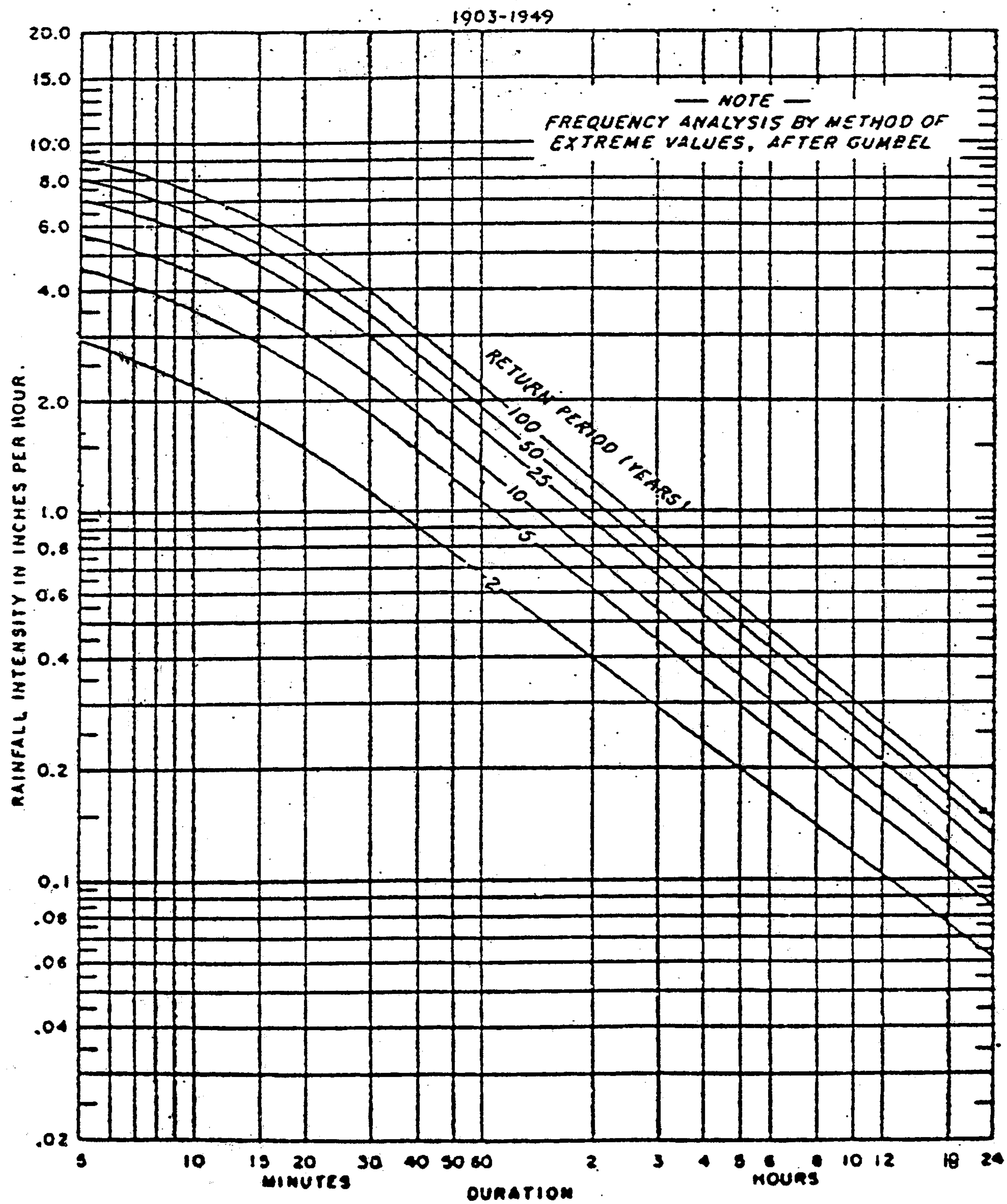


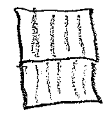
Figure 16.1.2-4
Duration Frequency

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

Efficiency of Approach Slab Drain

$w \approx 3'$ $\qquad T \approx 8.6'$



$$E = 1 - \left(1 - w/T\right)^{2.67}$$

$$E = 1 - \left(1 - 3/8.6\right)^{2.67}$$

$$E \approx .68 = 68\%$$

$$V \approx \frac{2\,K_g}{n} S^{.5} S_x^{.67} T^{.67} \qquad K_g = .56$$

$$V \approx \frac{2\,(.56)}{.016} (.015)^{.5} (.02)^{.67} (8.6)^{.67}$$

$$V \approx 2.64 \text{ fps}$$

depressed area ≈ 4.5'
1" depression

$l_{grate} \approx 2.5'$

grate type E - 30° Tilt Bar

From Chart 10 HEC21 Appendix C

1st 1' of grate will provide 100% efficiency of frontal flow

∴ after 1' of grate, 68% of water is gone

$Q_{remaining} = .32\,(1.9 \text{ cfs}) \approx .61 \text{ cfs}$

$T \approx 5.6'$ $\qquad V \approx 2.0$ fps

$E \approx .87 = 87\%$

2nd 1' of grate will take 87% of remaining

$Q_{remaining} \approx .13\,(.61 \text{ cfs}) \approx .08 \text{ cfs}$

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

$Q_{remaining} \approx .08$ cfs

$T \approx 2.6'$ $V \approx 1.2$ fps

$E \approx 100\%$

last $\frac{1}{2}'$ of grate will collect 100% of remaining water

∴ In theory, vane grate will collect all water at approach slab.

side by side drains
100% efficient

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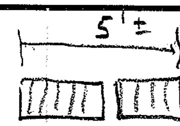
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)



$w = 1.3'$

for longitudinal drains

1st ft = 32.6% effecient

$E = 1 - (1 - w/T)^{2.67}$

$Q_{remain} = .674\,(1.9\ cfs) = 1.28\ cfs$

$T \approx 7.35'$ $V = 2.4\ fps$

$E \approx 40.5\%$ effcient

2nd ft

$Q_{remain} \approx .595\,(1.28\ cfs) \approx .76\ cfs$

$T \approx 6.04'$ $V \approx 2.1\ fps$

$E \approx 47.6\%$ effecient

3rd ft

$Q_{remain} \approx .524\,(.76) \approx .40\ cfs$

$T \approx 4.75'$ $V \approx 1.8\ fps$

$E \approx 57.4\%$ effecient

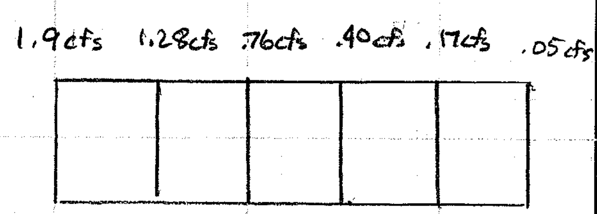
4th ft

$Q_{remain} \approx .426\,(.40) \approx .17\ cfs$

$T \approx 3.45$ $V \approx 1.43\ fps$

$E \approx 71.7\%$ efficient

$Q_{remain} \approx .28\ (.17) \approx .05\ cfs$



two drains ≈ 97.4% efficient

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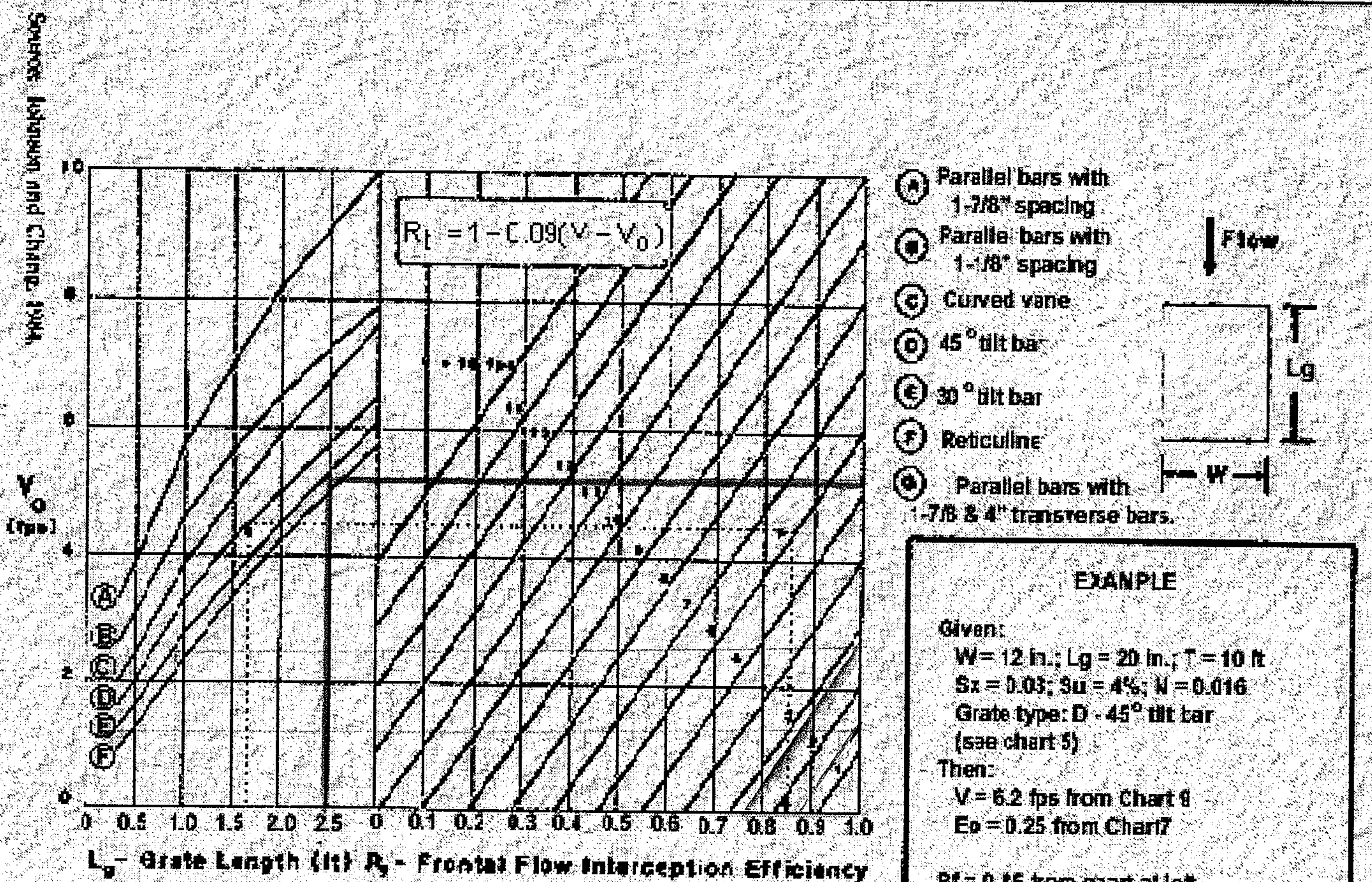
Go to Appendix C

Go to Chart 11



EXAMPLE

Given:
W = 12 in.; Lg = 20 in.; T = 10 ft
Sx = 0.03; Su = 4%; N = 0.016
Grate type: D - 45° tilt bar
(see chart 5)

Then:
V = 6.2 fps from Chart 9
Eo = 0.25 from Chart7

Rf = 0.85 from chart at left
Grate efficiency = E = RfEo
E = 0.85 x 0.25 = .21

Note: Side inflow to small rectangular inlets is negligible.

Chart 10. Frontal flow interception efficiency for typical grates.

Go to Appendix C

Go to Chart 8

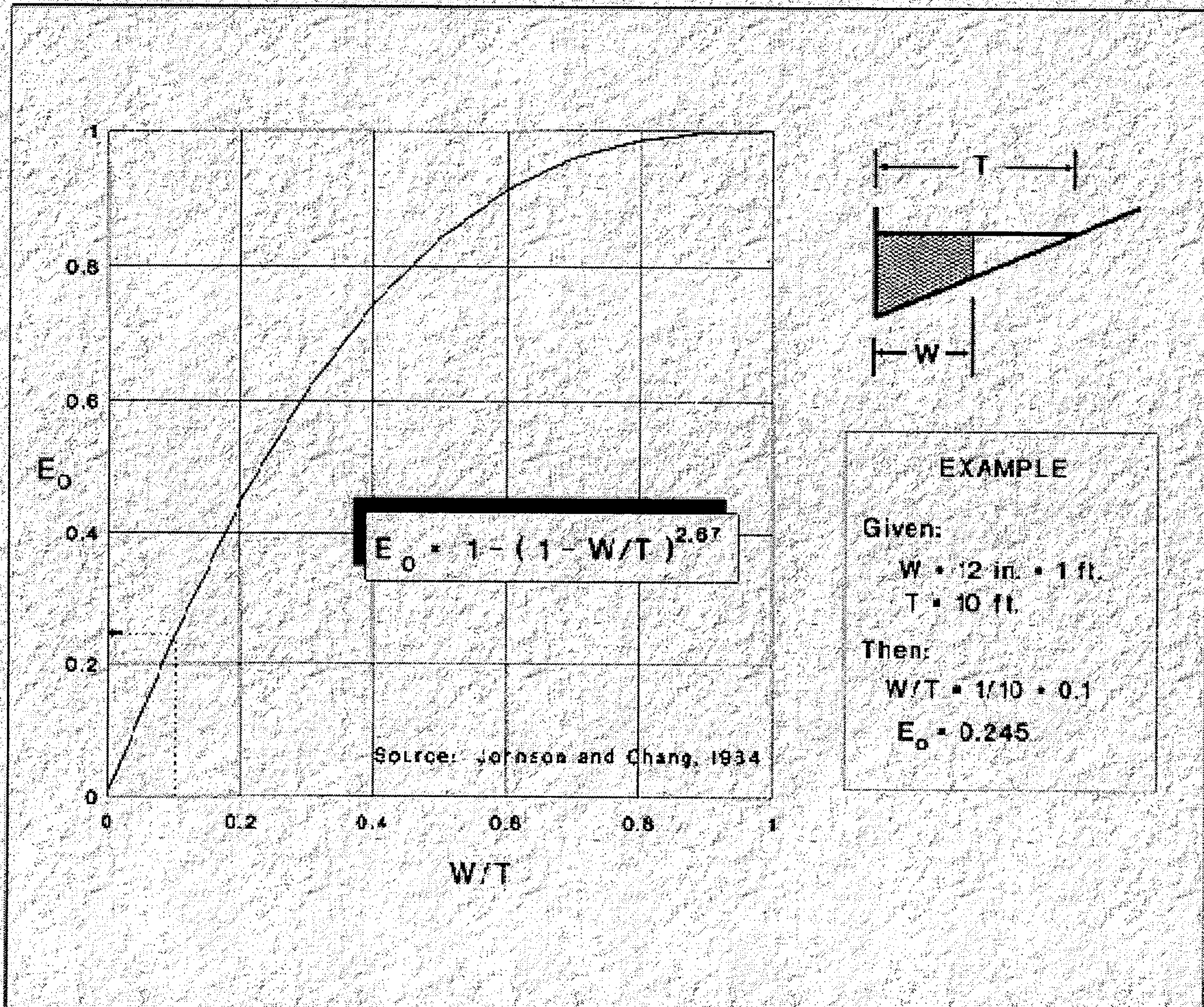


Chart 7. Frontal flow ratio for rectangular inlets.

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

BRIDGE DRAINAGE REVISION

FOR 1% Southbound Adverse Crown

$S_x = .01$ (1% cross-slope)

$n = .016$

$$Q = \frac{.56}{n} S_x^{1.67} S^{.5} T^{2.67}$$

$$2.53 = \frac{.56}{.016} (.01)^{1.67} (.015)^{.5} T^{2.67}$$

$$T \approx 14.63'$$

for T = 10' & 1.5% longitudinal slope

$$Q \approx \frac{.56}{.016} (.01)^{1.67} (.015)^{.5} 10^{2.67}$$

$$Q \approx .916 \text{ CFS} = CIA$$

$$A \approx \frac{.916}{.9(4.6)} \approx .22 \text{ acres} = 9637.9 \text{ SF}$$

@ 97' wide $l \approx \frac{9637.9}{97} \approx 99.36'$

for 10' shoulder storage, need drain @ 100' from high point for 1½% longitudinal slope

for T = 12' & 1.5% longitudinal slope

$$Q = .56/.016 (.01)^{1.67} (0.015)^{.5} 12^{2.67}$$

$$Q \approx 1.491 \text{ CFS}$$

$$A \approx \frac{1.491}{.9(4.6)} \approx .36 \text{ acres} = 15689.7 \text{ SF}$$

@ 97' wide $l \approx \frac{15689.7}{97} \approx 161.75'$

for 12' shoulder storage, need drain @ ~161' from high point for 1½% longitudinal slope

use drain @ ~140' from high point

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basin area for 140' drain
slope ≈ .007756 $S_x = .01$

$A \approx 10853.6$ SF ≈ .249 Acres

$Q = CIA$

$Q = .9(4.6)(.249) \approx 1.03$ CFS

$Q = \frac{.56}{n} S_x^{1.67} S^{.5} T^{2.67}$

$1.03 = \frac{.56}{.016}(.01)^{1.67}(.007756)^{.5} T^{2.67}$

$T \approx 11.83'$

R-404-TL drain

$w \approx 36''$

efficiency

$E = 1 - (1 - w/T)^{2.67}$

$E = 1 - (1 - 3'/12')^{2.67}$

$E \approx 53.6\%$ efficient

$.536 \times 1.03$ CFS ≈ .552 intercepted

$Q_{pass} = 1.03\text{ CFS} - .552 = .478$ CFS

$T \approx 8.87'$

with $w = 2.625'$

$E \approx 48.3\%$ efficient

.497 CFS intercepted

$Q_{pass} \approx .533$ CFS

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

$Q_{SB\ southend}$ $S \approx 1.1\%$

$A \approx 17296.5$ SF $\approx .40$ Acres

$Q = CIA$

$= .4(4.6).40$ Acres $\approx$ 1.66 CFS

$Q_{with\ drain} = 1.66 - .55 \approx 1.11$ CFS

$1.11 \text{ CFS} = \frac{.56}{.016}(.01)^{1.67}(.011)^{.5}T^{2.67}$

$T \approx 11.39'$

Use approach slab drain

$Q_{SB\ Northend}$ $S \approx 1.07\%$

$A \approx 16932.3$ SF $\approx .39$ Acres

$Q \approx .4(4.6)(.39) \approx 1.61$ CFS

$Q_{with\ drain} \approx 1.61 - .55 \approx 1.06$ CFS

$1.06 \text{ CFS} = \frac{.56}{.016}(.01)^{1.67}(.0107)^{.5}T^{2.67}$

$T \approx 11.26'$

Use approach slab drain

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

CONSTRUCTION FLOWS

50' Basin

TIME OF CONCENTRATION

Time from High Point to Gutter

$\ell \approx 97'$

$$t_o = \frac{K_w (\ell n)^{.6}}{(C i)^{.4} S^{.3}}$$

Eqn 2 Hec 21 3.2.1.3

assume $i \approx 3$

$S = .02$

$n = .016$

$$t_o \approx \frac{(.93)\left[(97)(.016)\right]^{.6}}{\left[.9(3)\right]^{.4}(.02)^{.3}}$$

$t_o \approx 2.63$ min

Time from Gutter to 50' Basin Point

$\ell \approx 50'$ $S \approx .14\%$ avg.

assume overland flow

$$t_o = \frac{.93\left[(50)(.016)\right]^{.6}}{\left[.9(3)\right]^{.4}(.0014)^{.3}}$$

$t_g \approx 3.93$ min

$t_c = t_o + t_g \approx 2.63 + 3.93 \approx 6.56$ min

$i \approx 2.7$ in/hr

$t_o \approx 2.74$ min

$t_g \approx 4.10$ min

$t_c \approx 6.84$ min

$i \approx 2.5$ in/hr $t_o \approx 2.83$ min

$t_g \approx 4.22$ min

$t_c \approx 7.1$ min

Use $i \approx 2.5$ in/hr

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$I = 2.5$ in/hr $\qquad A_{50' basin} \approx .10$ acres

$Q_{50' Basin} = .9(2.5)(.1) \approx .23$ cfs

$$Q = \frac{.56}{n} S_x^{1.67} S^{.5} T^{2.67}$$

$$.23 = \frac{.56}{.016} (.02)^{1.67} (.0025)^{.5} T^{2.67}$$

$$T \approx 5.4'$$

$K_g = .56$

$$V = \frac{2 K_g}{n} S^{0.5} S_x^{0.67} T^{.67}$$

$V \approx .79$ ft/sec $\qquad t = 50/.79 \approx 63$ sec

100' Basin

Time from High Point to Gutter

assume $i \approx 2.4$ in/hr

$t_o \approx 2.88$ min

Gutter flow

$S_{avg} \approx .0025$

$t_o \approx 5.47$ min

$t_c \approx 8.35$ min $\Rightarrow i \approx 2.4$ in/hr okay

$A_{100 \, basin} \approx .20$ acres

$Q \approx .9(2.4) .2 \approx .43$ cfs

$S \approx .54\%$

$T \approx 5.9'$

SPREAD FOR 3' BASIN ≈ 2.5'

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DESIGN COMPUTATIONS (Grid)

Assuming ½ width (one direction of travel)

$\ell \approx 48.5'$

$t_c \approx 5.2$ min

$I \approx 2.9$ in/hr

$A_{50} \approx .05$ acres

$Q_{50} \approx .9(2.9)(.05) \approx .13$ cfs

$T \approx 4.4'$

w/ 2' shoulders

spread will still be in driving lane

PLAN FOR THIS OCCURRENCE DURING CONSTRUCTION

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

PRELIMINARY DRAINAGE

$A \approx 22344$ SF $= .51$ ACRES $\quad l \approx 270'$ from high pt

$Q = CIA$

Design Frequency = 5 years

Plains Region

V @ full shoulder width

$V = \frac{1.12}{n} S_x^{.67} S^{.5} T^{.67}$

$n = .016$

$S_x = .02$

$S \approx .015$ @ end of bridge $\quad 0\%$ @ peak

$T = 12'$

$V = 3.3$ fps $\quad$ assume avg ≈ 1.7 fps

time $\approx 270' / 1.7/s = 160$ sec ≈ 2.7 minutes

use $T_c \approx 5$ minutes

$I \approx 4.6$ in/hr

$C = .9$

$Q \approx (.9)(4.6)(.51) \approx 2.1$ CFS

$T \approx 9$ ft < 12 ft okay

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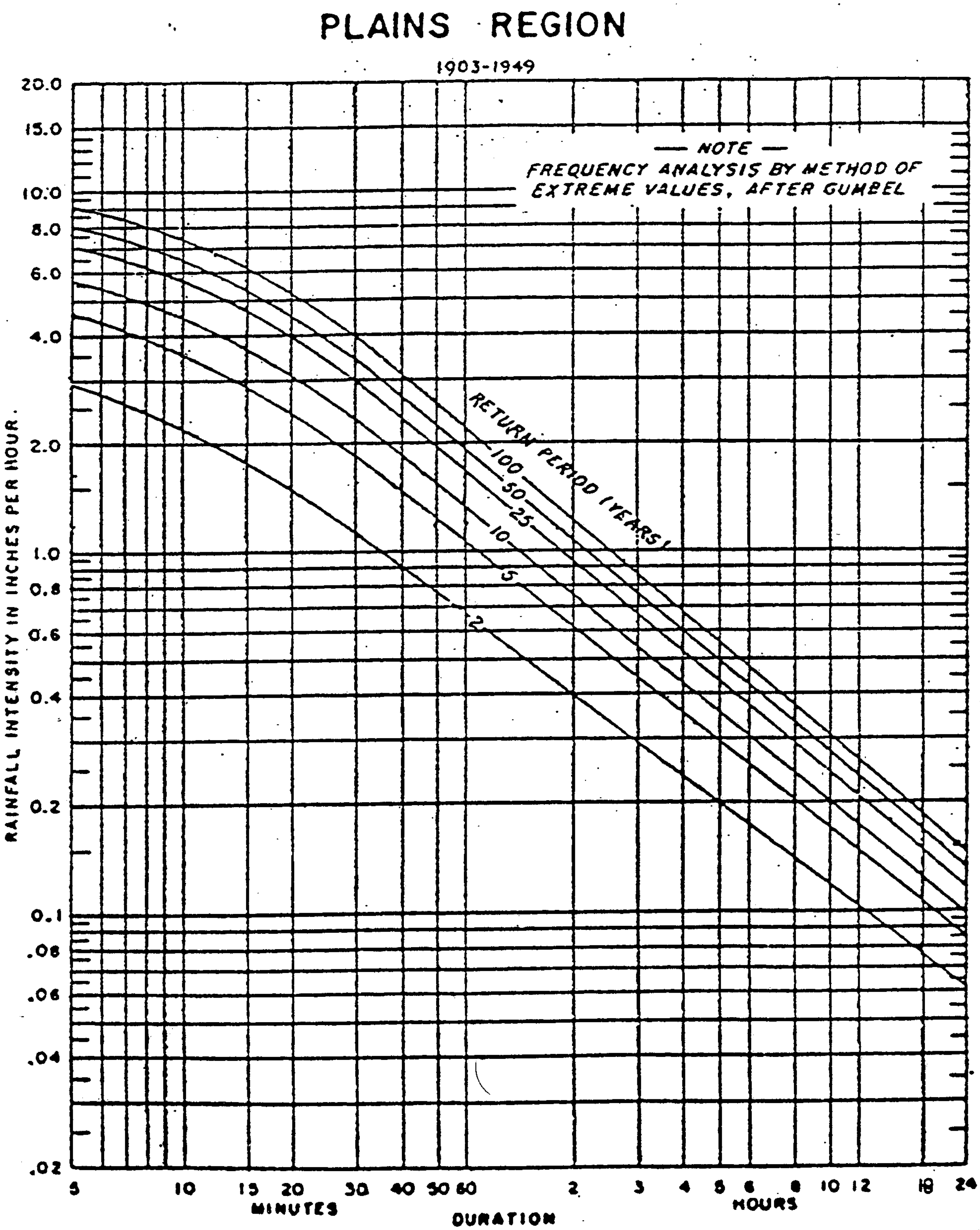

Figure 16.1.2-4
Duration Frequency

Go to Chart 5

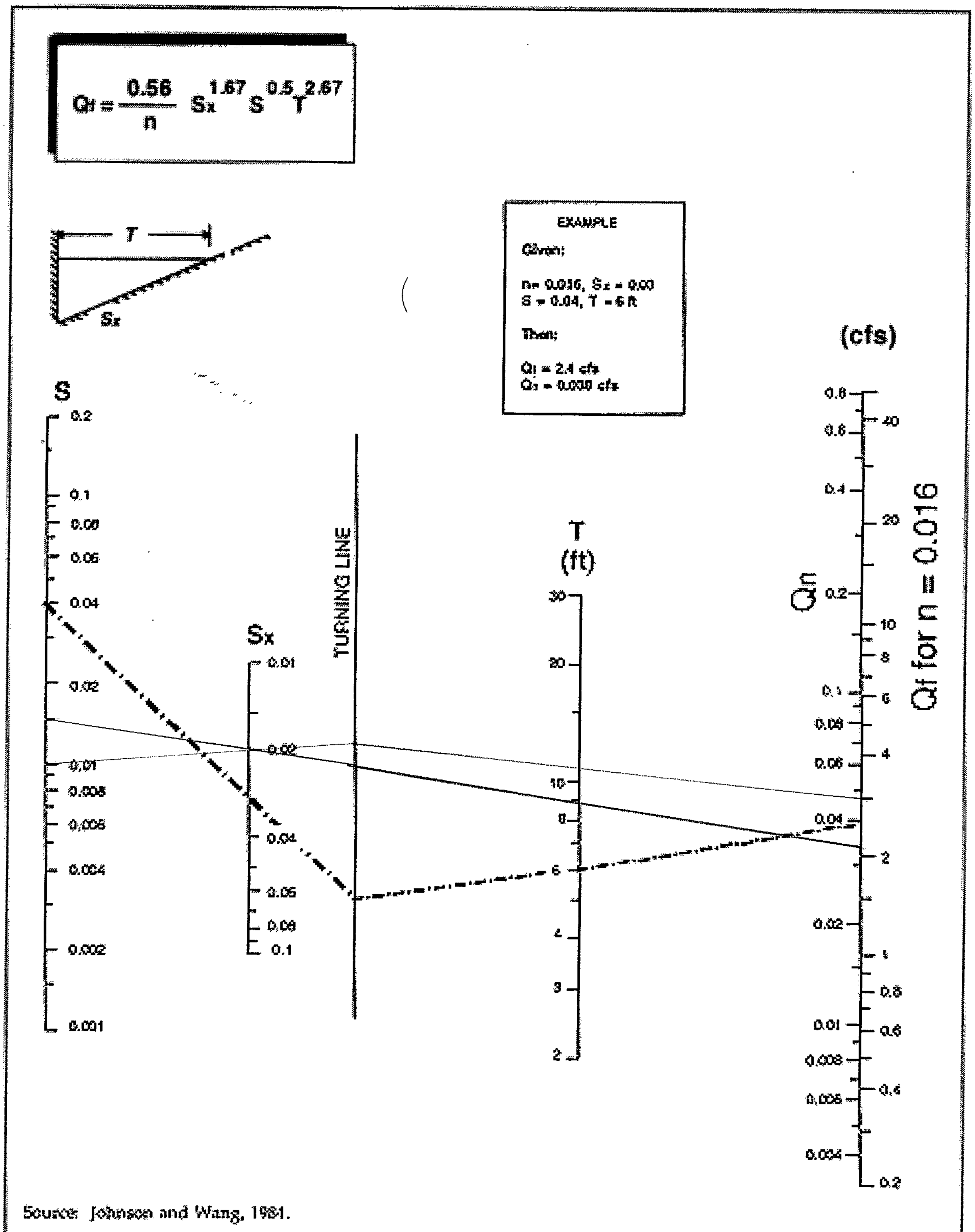

Chart 4. Flow in triangular gutter sections.

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

BRIDGE DRAIN SYSTEM

PER DESIGN MANUAL, USE 8" ϕ PIPE

INLETS - NEENAH R 4014-TL (or approved equal)
outlet = 6" flanged

$Q \approx 1$ CFS ≈ 450 gpm

full flow $V = \frac{.32 Q}{A} \approx \frac{(.32)(450 \text{ gpm})}{\frac{\pi(8)^2}{4}} \approx 2.86$ fps

gravity lines should be sized for $V \approx 3$ fps ✓ good

capacity of 8" line @ 1% ≈ 800 gpm
@ ½% ≈ 600 gpm

% Capacity $\approx \frac{450}{800} \approx 56\%$

$V \approx$ full flow V

HDPE Support spacing ≈ 70" for SDR 26 8" ← use
45" for SDR 32.5

steel support spacing ≈ 22' sch 40 8"
PVC support spacing ≈ 7.5' sch 40 8"
10.5' sch 80 8"

HDPE pipe wt ≈ 3.79 lb/ft
steel pipe wt = 28.55 lb/ft
water wt ≈ 22 lb/ft

for 10' spacing load ≈ 545 lbs
for 5' spacing load ≈ 272 lbs

2 embeds per support
@ 5' spacing 272/2 = 136 lbs/embed
use P15T, 1/4"-20NC (good for 750 lbs)

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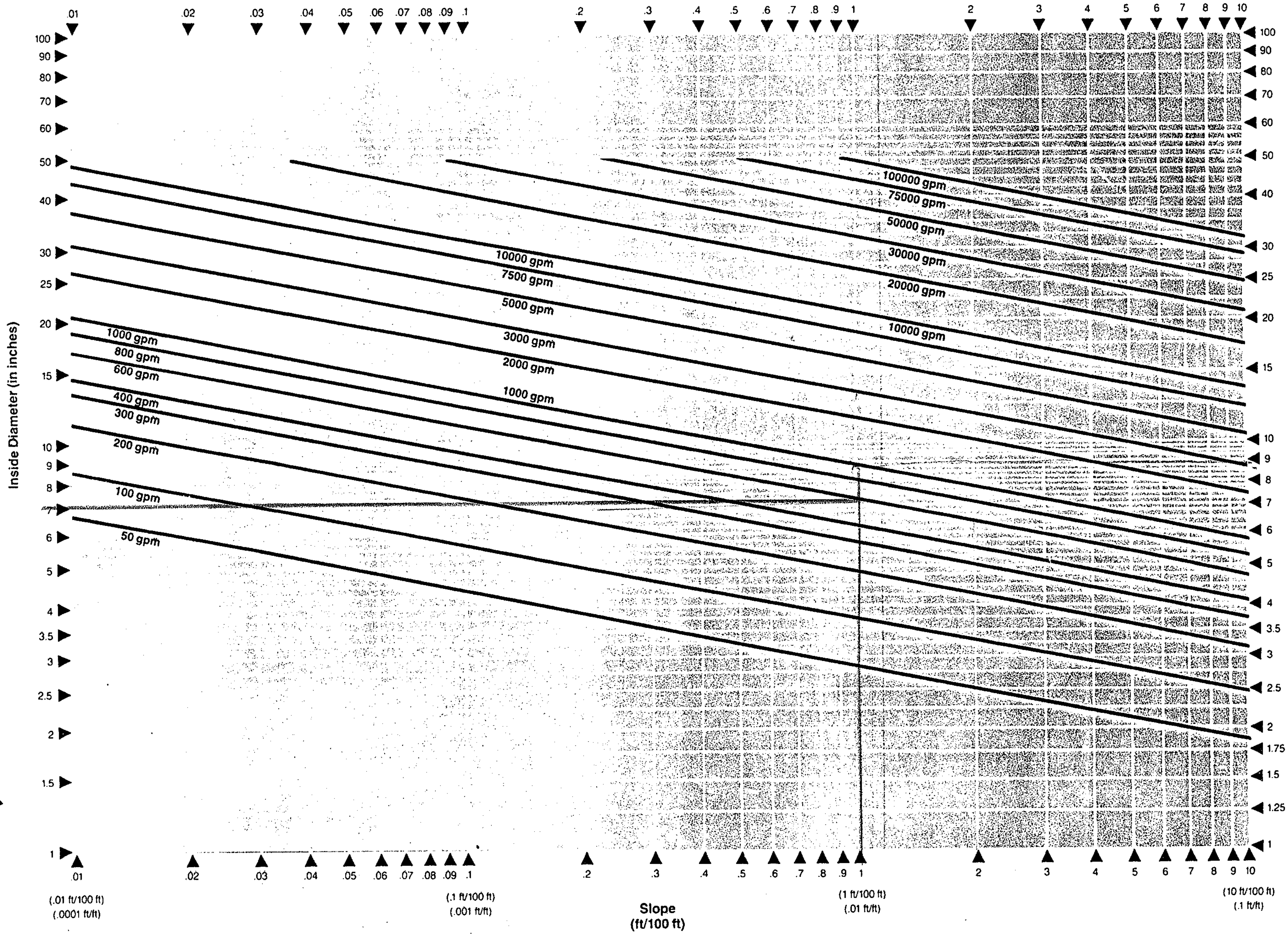
Chart 7

Inside Diameter vs. Flow Rate at a Given Slope
Gravity Full Flow: Water



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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

expansion $\approx 1.2 \times 10^{-4}$ in/in/°F

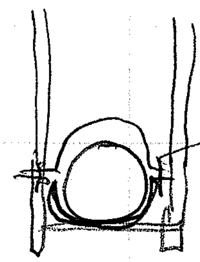
100° → 30°

$\Delta = 70° * 1.2 \times 10^{-4} * 45' * 12''/ft \approx 4.5''$

bridge movement $\Delta = 70° \times 1 \times 10^{-6} * 45' * 12''/ft \approx .22''$

$\Delta \approx 4.3''$

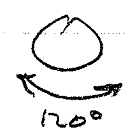
allow. for movement



top anchor every 10' or so to allow for movement

width of hanger = $1.5 * \Phi = 1\tfrac{1}{2} * 8.625'' \approx 13''$

minimum of 120° support



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Gravity Flow

Gravity Flow systems are typified by industrial and municipal waste and sewer lines as well as water and slurry pipelines. Some may operate with full flow and some may operate partially full. Because of the superior wall smoothness and excellent flow characteristics of Driscopipe, an efficient system can be designed.

Smaller diameters to carry a given flow mean reduced costs. Because Driscopipe does not "age", maintenance costs are less. Reduced operating costs supported by the reliability of Driscopipe can mean improved service.

Full Flow: Three things are required to select and size Driscopipe for a full flow gravity system: (1) GPM flow-rate requirements, (2) the slope of the pipeline and (3) a selection of an appropriate pipe I.D.

Based upon a full flow situation, the GPM flow rate can be calculated from the Manning equation as follows:

$$Q = 98.3\,A\,R_h^{2/3}S^{1/2}$$

Where: Q = Flow in gpm
R_h = Hydraulic radius (ID ÷ 4) (inches)
S = Slope (ft./foot)
A = Cross sectional area of pipe I.D. in sq. inches
V = Velocity (ft./sec.)
ID = Inside diameter in inches
(Note: Above formula includes $\eta = .009$)

The velocity can be calculated by:

$$V = 31.5\,R_h^{2/3}S^{1/2} = \left(\frac{.320\,Q}{A}\right)$$

The inside diameter by:

$$\text{I.D.} = \sqrt[2.67]{\frac{.03279\,Q}{S^{1/2}}}$$

And the slope by:

$$S = \frac{.001075\,Q^2}{\text{I.D.}^{5.34}}$$

All of this has been simplified and reduced into Chart 7. By use of this nomogram, the designer can specify a pipe I.D., slope, and the flow rate matched to the system requirements. By considering elevation changes, etc., the proper SDR can be selected. All I.D. and O.D. dimensions for each SDR can be found on Driscopipe dimensional charts.

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Partial Flow: Surprisingly enough, a gravity pipeline will carry more liquid when running 85% to 95% full than when 100% full. This is recognized as the effect of reduced friction due to the liquid's contact with less pipe wall surface. Chart 8 illustrates the changes in velocity and flow capacity when compared to full flow.

Chart 8

Carrying Capacity of Partially Full Pipes

% Full	Velocity (% of full)	Flow Capacity (% of full)
100	100	100
95	111	106.3
90	115	107.3
80	116	98
70	114	84
60	108	67
50	100	50
40	88	33
30	72	19
25	65	14
20	56	9
10	36	3

For gravity, partial flow pipelines, the GPM flow rate can be calculated through the use of the Manning equation as follows:

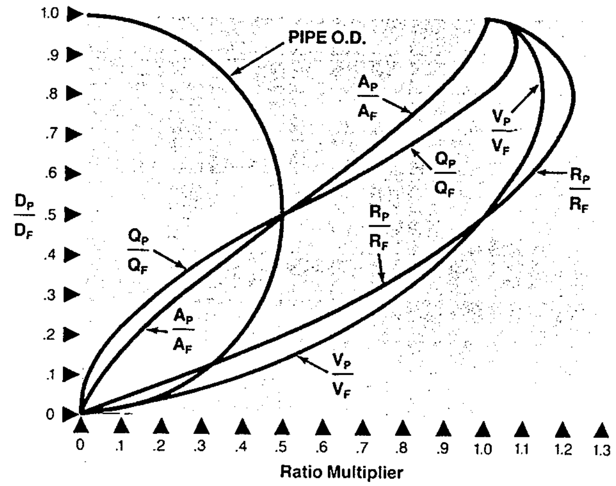
$$Q = 98.3\,A\,R_h^{2/3}S^{1/2}$$

Where: Q = Flow in gpm
A = Pipeline cross-sectional flow area in square inches
R_h = Hydraulic radius in inches
R_h = Flow area in sq. inches divided by wetted perimeter in inches
S = Slope or gradient (ft/ft)
$V = .320\frac{Q}{A}$ V = Velocity in ft/sec

Usually, a partially full gravity flow pipeline is studied as a full flow pipeline of a different, but smaller, "equivalent" diameter. The "equivalent" diameter matches all the hydraulic characteristics of the larger, partial flow gravity pipeline. The velocity, GPM flow rate and slope are identical in each case. The equivalent diameter is four times the hydraulic radius ($D_{EQ} = 4 \times R_h$). The hydraulic radius for partial flow gravity pipelines is defined as the ratio of the cross-sectional flow area divided by the wetted perimeter. Chart 9 simplifies these calculations by applying a multiplier to the full flow condition. Refer to the chart and example problem for proper calculation procedures.

Chart 9

Gravity-flow partially full parameters

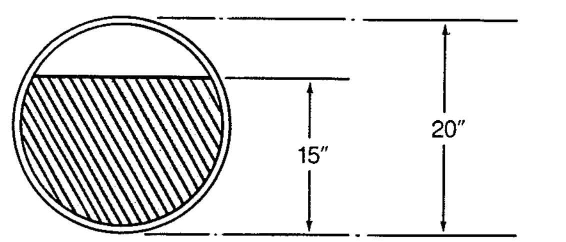


FULL FLOW
D_F = I.D.
A_F = Cross Sectional Flow Area
V_F = Full Flow Velocity
Q_F = Full Flow Rate
R_F = Full Hydraulic Radius

PARTIAL FLOW
D_P = Depth of Partial Flow
A_P = Cross Sectional Flow Area
V_P = Partial Flow Velocity
Q_P = Flow Rate
R_P = Hydraulic Radius

EXAMPLE

From Gravity Full Flow chart, a 20" I.D. Pipe Will Carry 9000 gpm at full flow. What will it carry if flow is 15" deep?



$$\frac{D_P}{D_F} = \frac{15''}{20''} = .75$$

$Q_P = Q_F \times .91 = 9000 \times .91 = 8100$ gpm
$A_P = A_F \times .82 = 314.2 \times .82 = 258$ sq. in.
$V_P = V_F \times 1.14 = 9.18 \times 1.14 = 10.5$ fps
[from $V_F = .408\,(Q_F/D_F^2)$]
$R_P = R_F \times 1.21 = (\frac{20}{4}) \times 1.21 = 6.05''$

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DRISCOPIPE® SYSTEMS INSTALLATION

Driscopipe products have been installed in many applications above and below ground. Polyethylene pipe has been used to cross land, lakes, deserts, bogs, and arctic tundra. Each installation requires thorough consideration of the environment in which the pipe is being installed.

Typical pipe installations can be categorized as one of seven types. The following pages discuss design details for each type of installation.

TYPE 1: Supported or Suspended Pipelines
TYPE 2: Overland Pipelines
TYPE 3: Marine Pipelines
TYPE 4: Water Surface Pipelines
TYPE 5: Marsh Pipelines
TYPE 6: Sliplined Pipelines
TYPE 7: Buried Pipelines

SUPPORTED OR SUSPENDED PIPELINES

Horizontally supported pipelines are affected by the weight of the pipe and its contents between supports. When the sag or deflection between supports is minimized, the stress in the pipe wall can be controlled. Supports should be spaced to limit the mid-span deflection to about ¼" using a simple, continuous beam analysis.

Supports should cradle the pipe for at least 4" or 1.5 times the pipe diameter, whichever is greater. A minimum of 120° of the pipe's circumference should be supported. The supports should be free of sharp edges.

Often, supported pipelines are installed outdoors. These installations are exposed to temperature changes due to weather. If possible, a supported or suspended pipeline should be installed as near its maximum operating temperature as practical (or in the hottest weather).

When a supported system is warmer than its installation temperature, the pipe will expand. As the pipe increases in length, lateral deflection or "snaking" will occur between restraints. The total amount of expansion that will occur depends on the pipe's length and the temperature increase above the system's installation temperature. While the total amount of expansion in a pipe cannot be changed, the designer can limit the deflection in a section of the pipe by selecting appropriate anchoring points.

The pipe must be restrained at all fittings and can be restrained at each support. Clamping the pipe at each support is recommended to limit deflections due to expansion. If the support is designed as an anchor point, it must be capable of restraining the pipe. If the pipeline is designed to move during expansion, the supports should provide a guide without restraint in the direction of movement.

PIPE SUPPORT SPACING Figures 9 through 13 give the required design support spacing for various DRs and pipe diameters. The distance between supports is based upon a continuous beam analysis and a mid-span deflection of 0.25" when the pipe is filled with water.

DRISCOPIPE®

Figure 9: Pipe Support Spacing for DR32.5

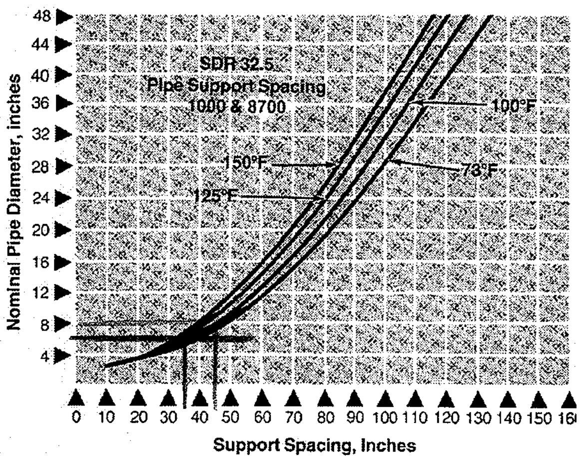


Figure 10: Pipe Support Spacing for DR26

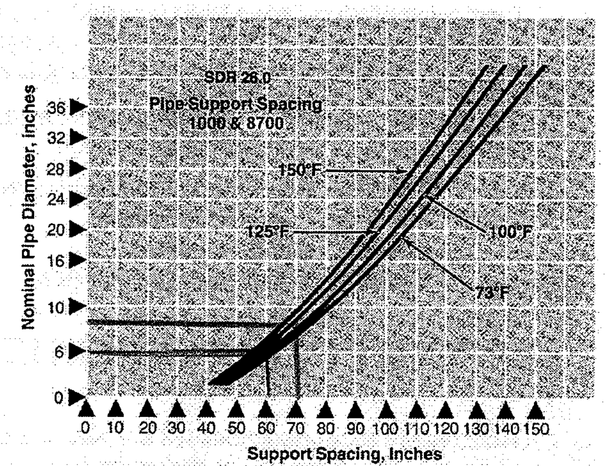


Figure 11: Pipe Support Spacing for DR17

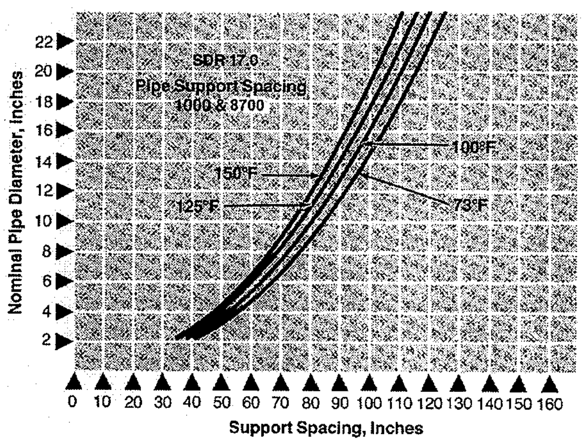


Figure 12: Pipe Support Spacing for DR11

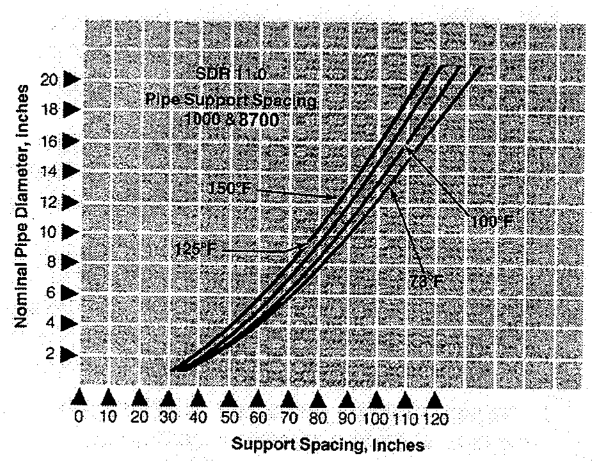
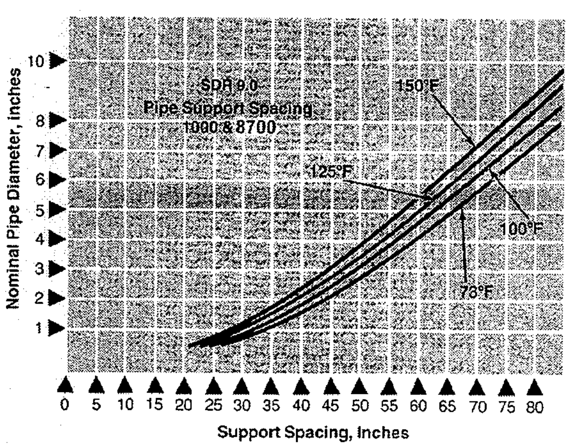


Figure 13: Pipe Support Spacing for DR9



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DRISCOPIPE® SYSTEMS INSTALLATION

Driscopipe products have been installed in many applications above and below ground. Polyethylene pipe has been used to cross land, lakes, deserts, bogs, and arctic tundra. Each installation requires thorough consideration of the environment in which the pipe is being installed.

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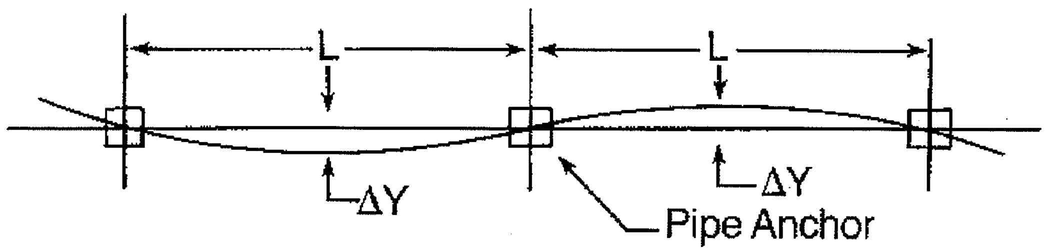
PIPE SUPPORT SPACING Figures 9 through 13 give the required design support spacing for various DRs and pipe diameters. The distance between supports is based upon a continuous beam analysis and a mid-span deflection of 0.25" when the pipe is filled with water.

Where:

ΔY = Lateral deflection, in.
L = Length of pipe between anchors, in.
α = Coefficient of thermal expansion, in./in./°F
ΔT = Change in temperature, °F
D = Pipe outside diameter, in.
ε = Tangential strain, in./in.

FIGURE 3: LATERAL DEFLECTION DUE TO THERMAL MOVEMENT IN OVERLAND PIPELINES



For any set of thermal conditions, an increase in anchor spacing will increase deflection, and vice versa. Increasing anchor spacing, L, to the maximum will reduce the number of anchor points needed but may increase wear on the pipe from movement and may increase the possibility of kinking the line if lateral movement does not occur uniformly.

One practical approach is to calculate anchor spacing by limiting strain, ε, in the pipe wall between 1% and 5%. The spacing at 5% strain will give the minimum distance between anchor points at the maximum allowable strain (ε_{max}). The spacing should be as large as possible considering other factors such as available right-of-way and slope of the ground. Higher values for L mean less strain and fewer anchor points.

Example: A pipeline installed on top of the ground in a straight condition and anchored at 50 foot intervals undergoes an increase in temperature of 50°F.

$$\Delta Y = 50ft \times 12in./ft\sqrt{0.50 \times .00012 \times 50F}$$

$$\Delta Y = 33in.$$

THERMAL CONSIDERATIONS IN BURIED PIPELINES In direct buried installations, soil friction will normally restrain pipe movement caused by seasonal temperature changes. Anchor requirements are minimized as stress relaxation occurs in the pipe. In some instances, concrete collars are used to transfer the thermal force into the soil around the pipe. The force in the pipe must be effectively transferred into the concrete collar. This is typically done by fusing branch saddles or a "waterstop" to the pipe and embedding the waterstop into the concrete collar.

The final tie-ins on a system should be made as close to operating temperature as practical. When installing polyethylene pipe that is warmer than the soil, a slightly longer length may be required to compensate for contraction of the pipe as it cools to ground temperature. The snaking in the trench which naturally occurs with pipe diameters 4" and below is normally sufficient to compensate for

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The amount of linear expansion or contraction for an unrestrained polyethylene pipe can be calculated from the following equation:

$$\Delta L = \alpha(T_2 - T_1)L$$

Where:

ΔL = Theoretical length change, in.
α = Coefficient of linear expansion, 1.2×10^{-4} in./in./°F
T_2 = Final temperature, °F
T_1 = Initial Temperature, °F
L = Length of pipe, in. at T_1

THERMAL STRESS RELAXATION When the temperature of a Driscopipe system changes, internal stresses develop as the pipe expands or contracts. This does not adversely affect or overstress the pipe. Polyethylene is a viscoelastic material and will relieve stresses by slightly realigning its molecular structure until equilibrium is achieved. This is a valuable engineering property which dissipates a major portion of the stress developed as the pipe tries to expand or contract.

The engineering formulas used to calculate forces resulting from expansion or contraction assume instantaneous temperature change. It is physically impossible to change the temperature of an object instantly. In laboratory experiments structured to create a near "instantaneous" temperature change on the pipe, the thermal stress has been measured and found to be about half the theoretical, calculated value. When the temperature change occurs over an extended period of time, the thermal stress is further reduced as stress relaxation occurs. Typically, Driscopipe systems are designed using one half the calculated tensile stress due to an "instantaneous" temperature change.

THERMAL CONSIDERATIONS IN SUPPORTED PIPELINES If practical, install the pipe when its temperature is near the maximum system operating temperature. As the pipe cools, tensile stress will develop and keep it straight between supports. When the pipe warms to its installation temperature, it returns to its installation condition and straightness. In this manner, sag between supports is minimized.

THERMAL CONSIDERATIONS IN OVERLAND PIPELINES By installing overland pipelines in a slightly snaked pattern, changes in the pipe's length can be controlled by lateral deflection. As the pipeline warms, the "S" configuration becomes slightly greater. As the pipe cools, the pipeline becomes straighter. Surface lines that are continuously operated full of fluid normally experience small temperature variations and are easy to control. The weight of the fluid also increases friction between the pipe and the ground and therefore reduces deflection.

It may be necessary to anchor the line at intervals to direct and limit the deflection to selected locations. In extreme cases, all deflection may occur in one area where friction is low. This condition is most likely to occur with empty lines or where large, sudden operating temperature changes occur.

LATERAL DEFLECTION DUE TO THERMAL MOVEMENT The following formulae will allow the designer to calculate lateral deflection of the pipeline and anchor point spacing.

$$\Delta Y = L\sqrt{0.50\alpha\Delta T}$$

$$L = \frac{\Delta Y}{\sqrt{0.50\alpha\Delta T}}$$

$$L = \frac{D\sqrt{96\alpha\Delta T}}{\varepsilon}$$

NC Threaded Inserts
For Suspending, Connecting and Anchoring to Concrete

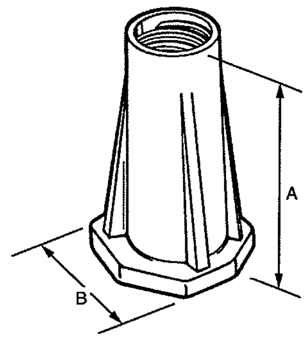
P-86 Star Insert®

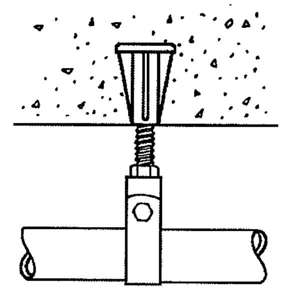
The Dayton Superior P-86 Star Inserts® are precision die-cast of zinc alloy and have excellent resistance to atmospheric conditions. These inserts are particularly suited for use in precast concrete elements and can be quickly secured to the form with an appropriately sized NC bolt. The Star Insert can also be nailed to the form utilizing the P-87 Star Adapter Plug.

P-86 Star Inserts are excellent inserts for securing machinery and equipment, for suspending piping and supporting railing/poles.

Two different types of inserts are available:

- Standard – Insert has a closed bottom to prevent concrete from entering the insert.
- Open Bottom – Threads run the full depth of the insert.



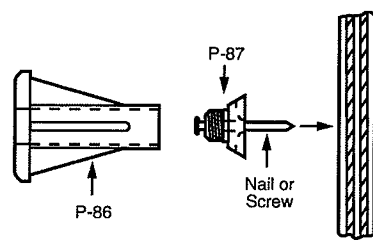


P-86 Star Insert®

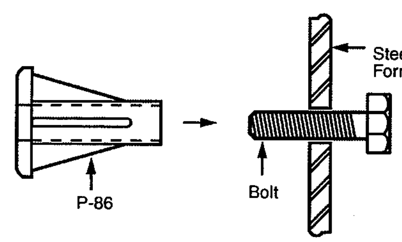
To Order:
Specify: (1) quantity, (2) name, (3) insert style.
Example:
200, P-86 Star Inserts, P45T.

P-86 Star Insert Selection Chart						
Insert Style	Insert Type	Bolt Diameter and Threads per Inch	Insert Length A	Bolt Torque	Safe Working Load Tension	B
P10T	Standard	10 – 24 NC	11/16"	10 in. lbs.	235 lbs.	7/16"
P15T	Standard	1/4" – 20 NC	1-1/2"	32 in. lbs.	750 lbs.	13/16"
P24T	Standard	3/8" – 16 NC	1"	100 in. lbs.	1,125 lbs.	7/8"
P25T	Standard	3/8" – 16 NC	1-3/8"	100 in. lbs.	1,300 lbs.	7/8"
P35T	Standard	1/2" – 13 NC	1-1/2"	19 ft. lbs.	1,500 lbs.	1-1/4"
P36T	Standard	1/2" – 13 NC	2-7/8"	19 ft. lbs.	3,600 lbs.	1-3/8"
P45T	Standard	5/8" – 11 NC	1-11/16"	40 ft. lbs.	1,900 lbs.	1-3/8"
P46T	Standard	5/8" – 11 NC	2-7/8"	40 ft. lbs.	4,625 lbs.	1-9/16"
P55T	Standard	3/4" – 10 NC	1-11/16"	56 ft. lbs.	2,275 lbs.	1-9/16"
P56T	Standard	3/4" – 10 NC	3"	56 ft. lbs.	4,550 lbs.	1-3/4"
P25TOB	Open Bottom	3/8" – 16 NC	1-3/8"	100 in. lbs.	1,300 lbs.	7/8"
P35TOB	Open Bottom	1/2" – 13 NC	1-1/2"	19 ft. lbs.	1,500 lbs.	1-1/4"

Safe working load provides an approximate factor of safety of approximately 3 to 1 in 3,500 psi normal weight concrete.



P-86 Form Installation Using P-87 Adapter Plug



P-86 Form Installation Using Bolt

07/07

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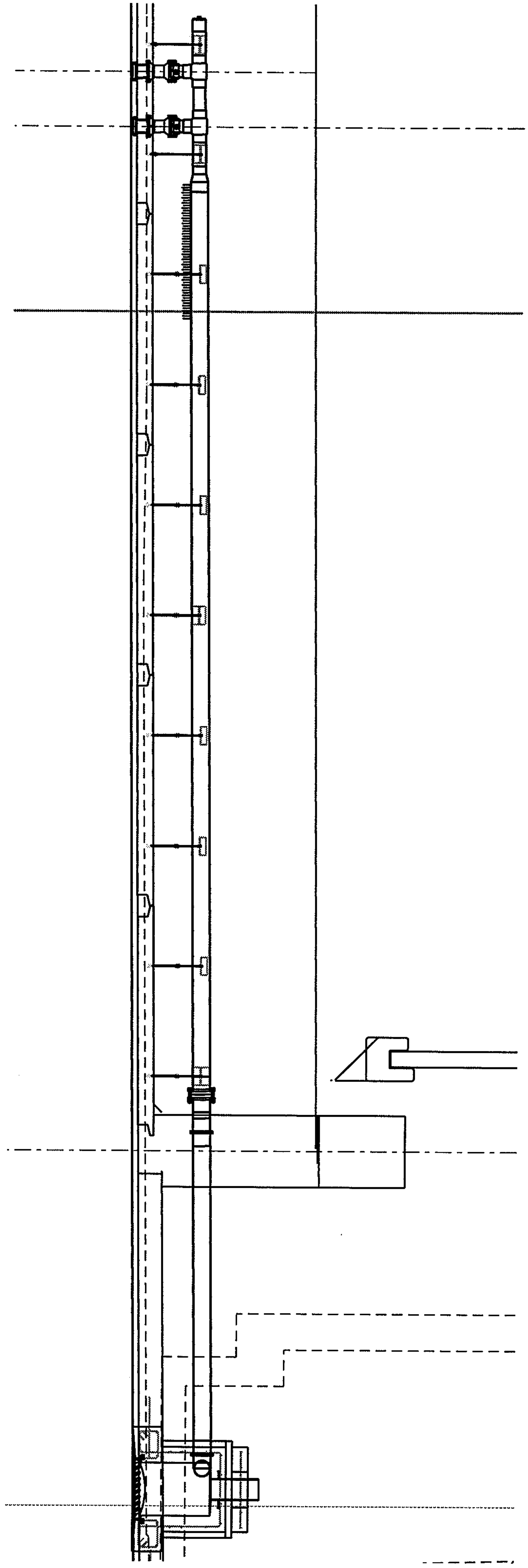

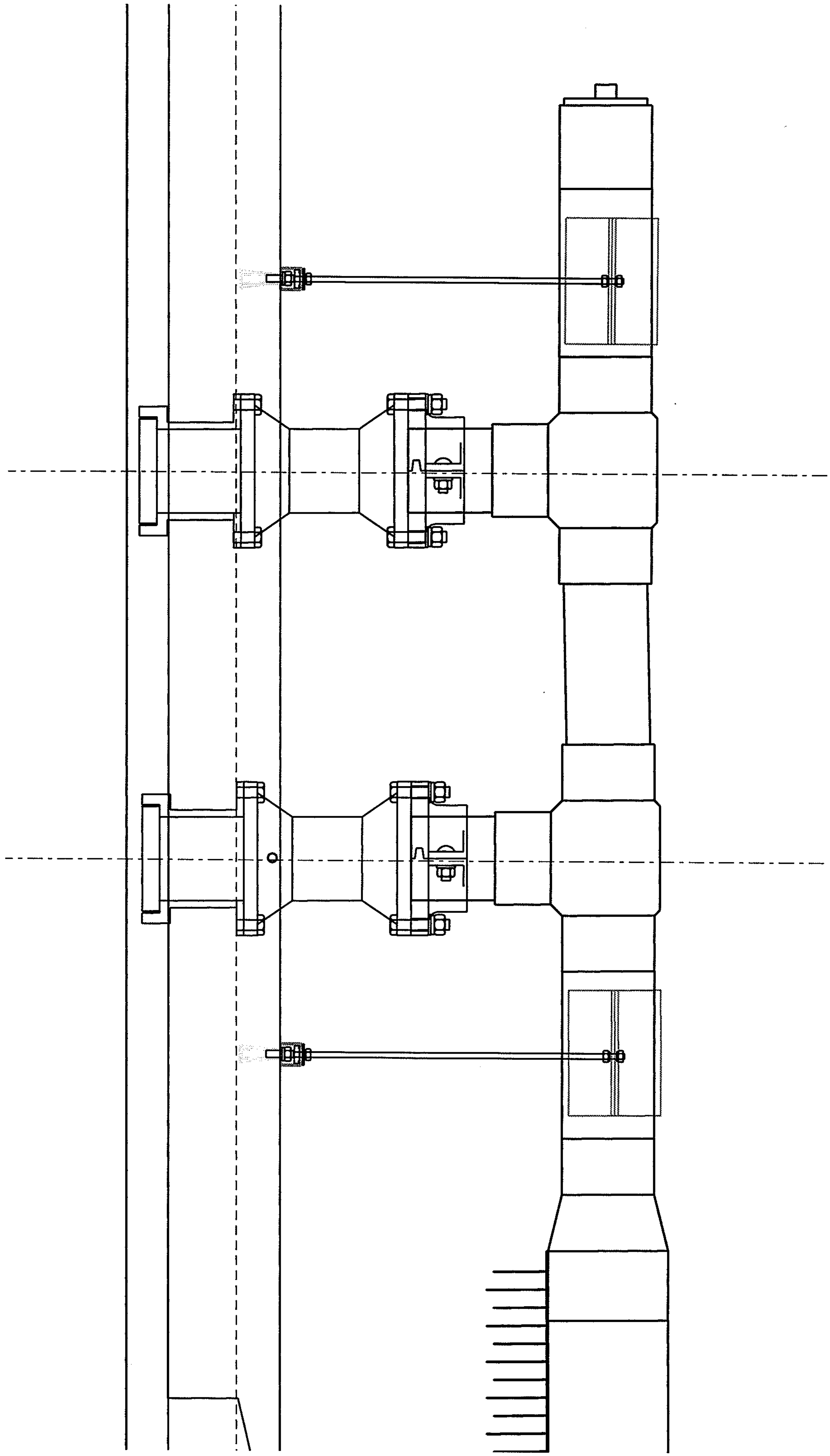

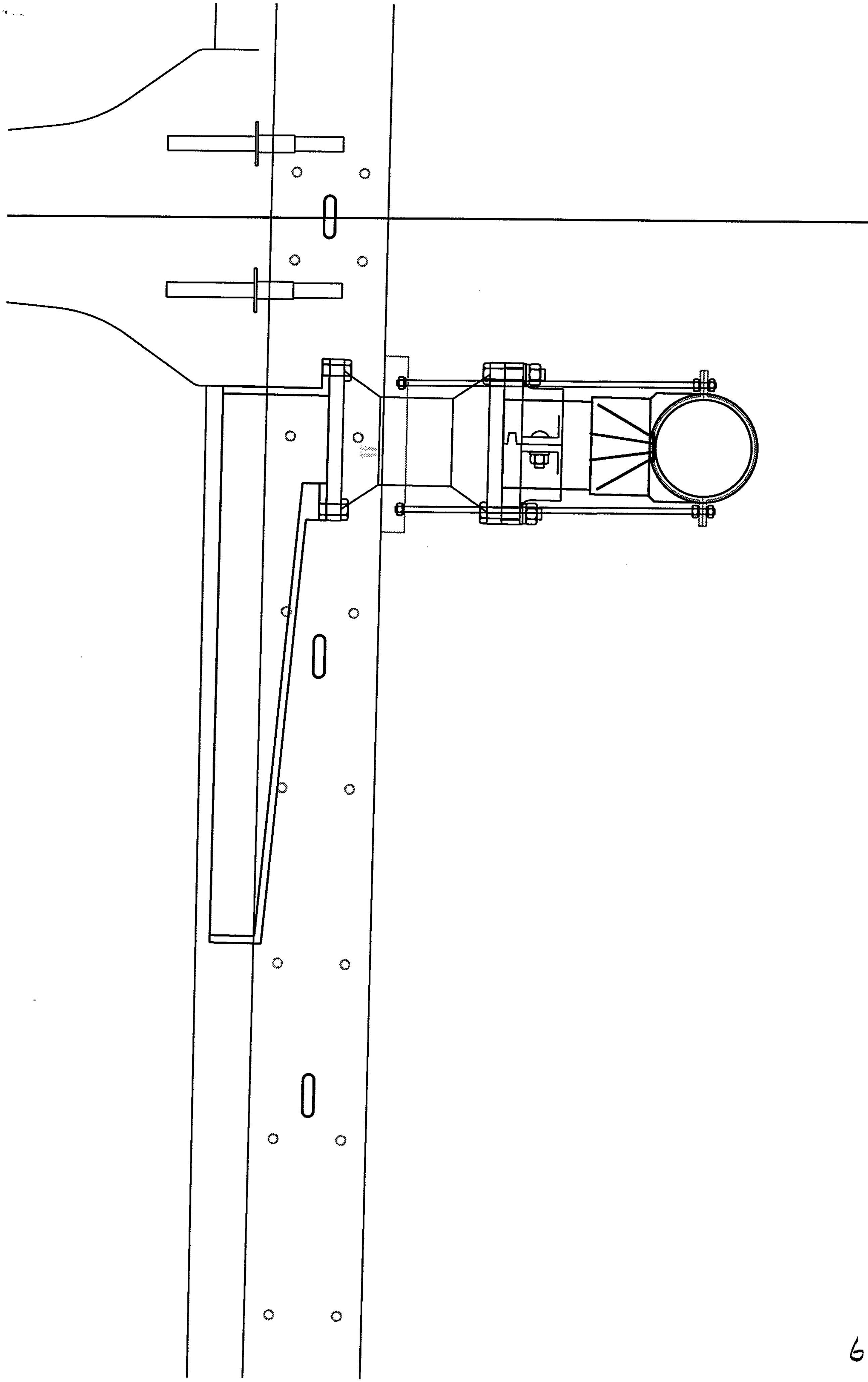

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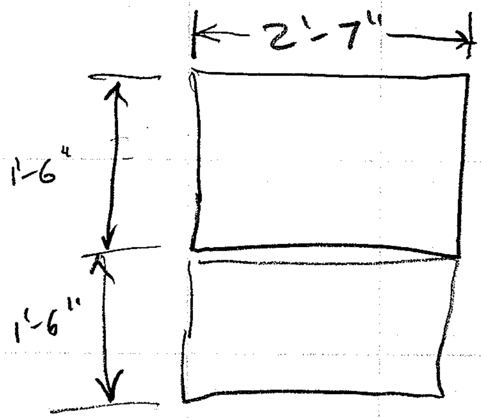
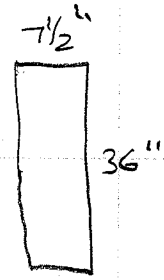
Temporary INlet Covers

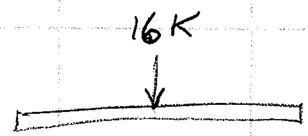




$M_{max} = 2.5$ ft k

$T_{min} = .87"$

Use 1" plate



$$M_{max} = \frac{P\ell}{4} = \frac{(16)(2.58)}{4} = 10.32 \text{ ft k}$$

$$\sigma = \frac{Mc}{I}$$

$\sigma_{allow} = .20$ ksi

$$20 \text{ ksi} = \frac{\left(10.32 \text{ ft k } \frac{12"}{ft}\right)\left(T_{min}\ \cancel{/2}\right)}{\frac{1}{6}\ \frac{1}{\cancel{12}}\ (\cancel{12"})\ (T_{min})^{\cancel{3}\ 2}}$$

$$T_{min}^2 = \frac{10.32}{20\ (1/6)} = 3.096$$

$$T_{min} \approx 1.76"$$

2¼"

$$\Delta = \frac{P\ell^3}{48EI} = .042"$$

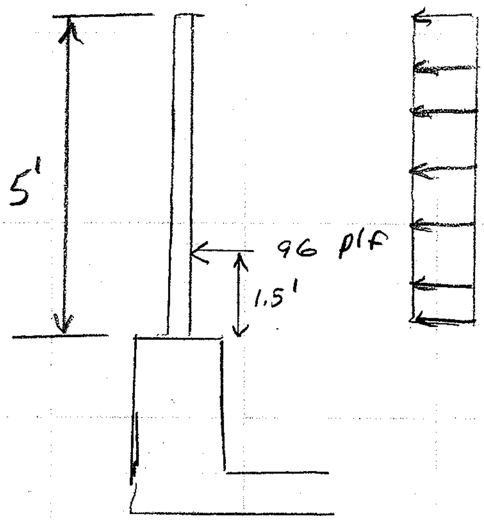
for 2" plate
$I = 8$

Use 2" Plate

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 631 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

FENCE



grounding requirements

5'-0" spacing of posts

$M = (32 psf)\ 5' * 5' * 5/2 + 96\ lb/ft * 5' * 1.5'$

$M \approx 2000 + 720$

$M \approx 2720$ ft lb $= 2.72$ ft K

$\sigma_{allow} \approx 19$ ksi

$\sigma = \frac{M}{S}$

$S_{min} = \frac{M}{\sigma} = \frac{2.72 \text{ ft K} * 12''/ft}{19 \text{ ksi}} = 1.72 \text{ in}^3$

use 3"ϕ min standard pipe - $S = 1.72 \text{ in}^3$

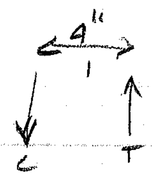
Use 2 1/2" XS pipe

2.323" inside ϕ
2.875 outside ϕ

use 2 1/4" ϕ bar at base

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 632 of ____

FENCE



$$M = T\,4''$$

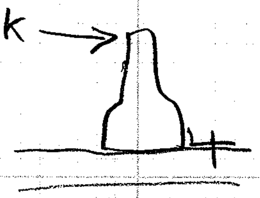
$$T = \frac{32.64'' k}{4''} = 8.16\ K$$

2 bolts ~ 4.08 k/ bolt

use ½" ϕ bolts

TEMPORARY BARRIER

54K →



assume TL-4 loading

$F_t = 54$ Kips

with 4 bolts per barrier

$P_{bolt} = 54/4 = 13.5$ K

with 6 bolts per barrier

$P_{bolt} = 54/6 = 9$ K

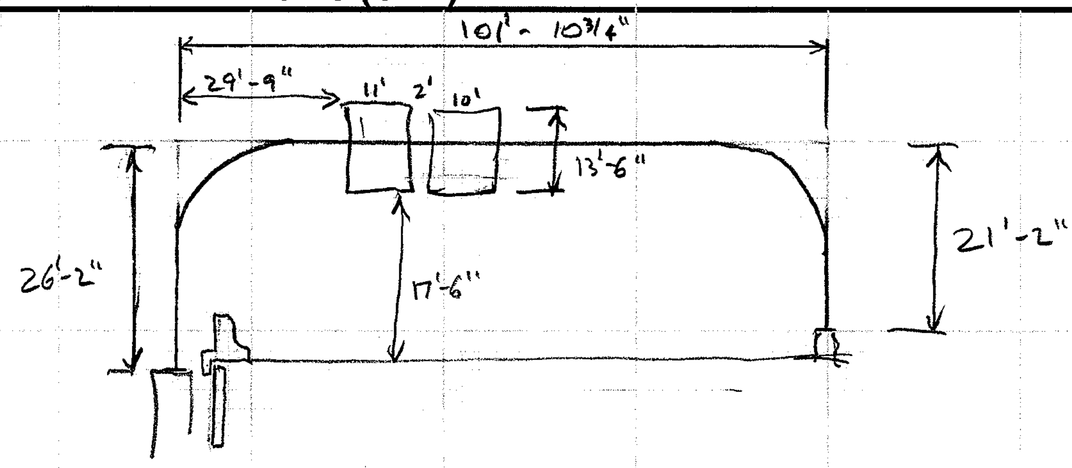
with 8 bolts per barrier

$P_{bolt} = 54/8 = 6.75$ K

3' or greater is available behind barrier, anchoring is not required

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

CAISSON EVALUATION



LOADS

Signs ≈ 36.3 lb/ft @ 21'

sign connections ≈ 21.2 lbs/ft @ 21'

mast arm ≈ 94.62 lb/ft @ 78'

pole = 125.5 lb/ft @ 31.4' & 26.4'

pole split ≈ 91.5 lbs

guide bar ≈ 1.92 lb/ft @ 78'

field splice ≈ 500 lbs

wind load ≈ 27.6 psf (for 100 mph)

VERTICAL LOAD

Signs 36.3 * 21 ≈ 762.3 lbs

sign connections 21.2 * 21 ≈ 445.2 lbs

mast arm 94.62 * 78 ≈ 7380.4 lbs

pole 125.5 * 31.4 ≈ 3940.7 lbs

pole split = 91.5 lbs

guide bar 1.92 * 78 ≈ 149.8 lbs

field splice ≈ 2 * 500 ≈ 1000 lbs

By:	Date	Project no.	Project code (SA#):
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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

VERTICAL LOAD (cont.)

total load 1 caisson

$3/5\,(762.37 + 445.2) + 1/2\,(7380.4) + 3940.7 + 965$
$+ 1/2\,(149.8) + 1000$

$\approx$ **9522 lbs**

MOMENT

wind load $\approx$ 27.6 psf $\Rightarrow$ 24.84 lb/ft on 2' ϕ pole

Signs

$(11 * 13.5 + 10 * 13.5)\, 27.6\,psf * \frac{26.17}{1000} \approx 204.8$ ft kip

Mast Arm

wind load $\approx$ 24.84 lb/ft

$1/2\,(78') * 24.84\,lb/ft * \frac{26.17}{1000} \approx 25.4$ ft kip

Pole

$(37.4') * 24.84\,lb/ft * \frac{26.17/2}{1000} = 12.2$ ft kip

Total Moment $\approx$ **242.4 ft kip**

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 636 of

CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

horizontal load

wind load ≈ 27.6 psf ⇒ 24.84 lb/ft on 2' Ø pole

Signs

(11 * 13.5 + 10 * 13.5) * 27.6 psf ≈ 7825 lbs

Mastarm

1/2 78 * 24.84 ≈ 969 lbs

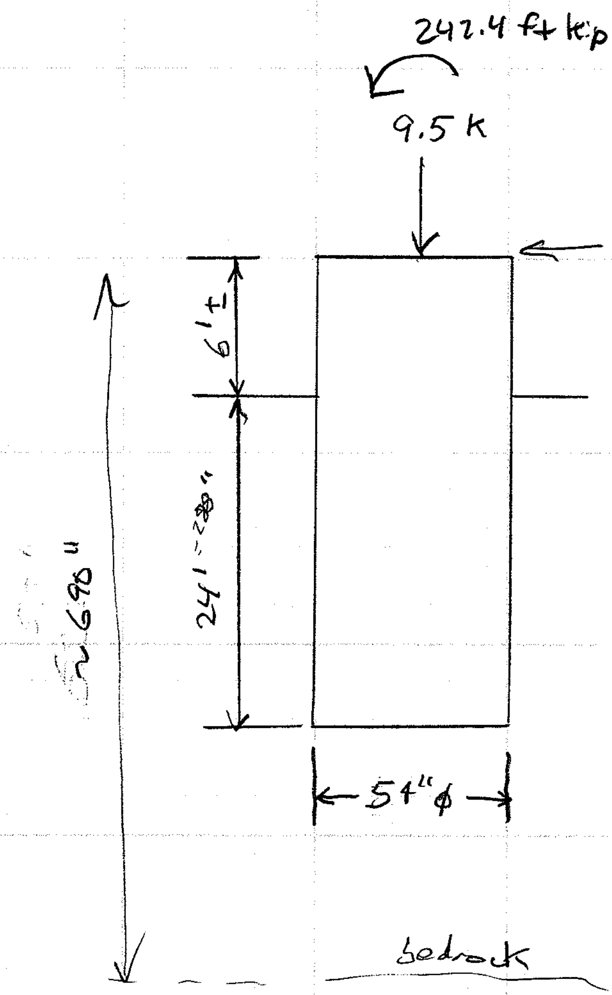
Pole

37.4 * 24.84 ≈ 929 lbs

total ≈ 9723 lbs

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 637 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)



max moment for 5' exposed ≈ 323 ft K

max moment for 10' exposed ≈ 370 ft K

Caisson Capacity ≈ 1840 ft K ✓ okay

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 638 of ___

```
sign structure caisson  5' exposed

   1
  100     4   .060D+03   .348D+03
  .000D+00   .540D+02   .417D+06   .229D+04   .364D+07
  .060D+03   .540D+02   .417D+06   .229D+04   .364D+07
  .060D+03   .540D+02   .417D+06   .229D+04   .364D+07
  .348D+03   .540D+02   .417D+06   .229D+04   .364D+07
    2     8     8     0
    4   .060D+03   .500D+03   .640D+02
    3   .500D+03   .100D+04   .200D+04
  .100D+01   .000D+00
  .060D+03   .000D+00
  .060D+03   .666D-01
  .414D+03   .666D-01
  .414D+03   .694D-01
  .690D+03   .694D-01
  .690D+03   .780D-01
  .100D+04   .780D-01
  .100D+01   .000D+00   .000D+00   .000D+00
  .060D+03   .000D+00   .000D+00   .000D+00
  .060D+03   .000D+00   .320D+02   .000D+00
  .414D+03   .000D+00   .320D+02   .000D+00
  .414D+03   .000D+00   .320D+02   .000D+00
  .690D+03   .000D+00   .320D+02   .000D+00
  .690D+03   .694D+02   .000D+00   .500D-02
  .100D+04   .694D+02   .000D+00   .500D-02
     5     1   .000D+00
  .100D+01   .000D+00
  .000D+03   .000D+00
  .000D+03   .972D+04
  .001D+03   .972D+04
  .001D+03   .000D+00
    1
    1   .972D+04   .291D+07   .950D+04
    0
    1     1     0
  100   .100D-05   .360D+03
```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```
sign structure caisson 10' exposed

   1
 100     4   .120D+03   .408D+03
 .000D+00   .540D+02   .417D+06   .229D+04   .364D+07
 .120D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .120D+03   .540D+02   .417D+06   .229D+04   .364D+07
 .408D+03   .540D+02   .417D+06   .229D+04   .364D+07
   2     8     8     0
   4   .120D+03   .500D+03   .640D+02
   3   .560D+03   .100D+04   .200D+04
 .100D+01   .000D+00
 .120D+03   .000D+00
 .120D+03   .666D-01
 .414D+03   .666D-01
 .414D+03   .694D-01
 .690D+03   .694D-01
 .690D+03   .780D-01
 .100D+04   .780D-01
 .100D+01   .000D+00   .000D+00   .000D+00
 .120D+03   .000D+00   .000D+00   .000D+00
 .120D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .414D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .000D+00   .320D+02   .000D+00
 .690D+03   .694D+02   .000D+00   .500D-02
 .100D+04   .694D+02   .000D+00   .500D-02
    5    1   .000D+00
 .100D+01   .000D+00
 .000D+03   .000D+00
 .000D+03   .972D+04
 .001D+03   .972D+04
 .001D+03   .000D+00
   1
   1   .972D+04   .291D+07   .950D+04
   0
   1     1     0
 100   .100D-05   .360D+03
```


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```
*******************************************************
*    PROGRAM LPILE1                                   *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                  *
*    ALL RIGHTS RESERVED                              *
*   ----------------------------------------------    *
*                                                     *
*              PREPARED ESPECIALLY FOR                *
*                                                     *
*           STATE DEPARTMENT OF HIGHWAYS              *
*                                                     *
*              DENVER, COLORADO 80222                 *
*                                                     *
*                  LICENSE NO. 138                    *
*                                                     *
*******************************************************
```

PROGRAM LPILE1

sign structure caisson 5' exposed

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N
****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 348.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	54.000	.417D+06	.229D+04	.364D+07
60.00	54.000	.417D+06	.229D+04	.364D+07
60.00	54.000	.417D+06	.229D+04	.364D+07
348.00	54.000	.417D+06	.229D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 60.00 IN

2 LAYER(S) OF SOIL

LAYER 1

THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 60.00 IN
X AT THE BOTTOM OF THE LAYER = 500.00 IN
MODULUS OF SUBGRADE REACTION = .640D+02 LBS/IN**3

LAYER 2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER = 500.00 IN
X AT THE BOTTOM OF THE LAYER = 1000.00 IN
MODULUS OF SUBGRADE REACTION = .200D+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
8 POINTS

X,IN	WEIGHT,LBS/IN**3
1.00	.00D+00
60.00	.00D+00
60.00	.67D-01
414.00	.67D-01
414.00	.69D-01
690.00	.69D-01
690.00	.78D-01
1000.00	.78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
8 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
1.00	.000D+00	.000D+00	-----
60.00	.000D+00	.000D+00	-----
60.00	.000D+00	.320D+02	-----
414.00	.000D+00	.320D+02	-----
414.00	.000D+00	.320D+02	-----
690.00	.000D+00	.320D+02	-----
690.00	.694D+02	.000	.500D-02
1000.00	.694D+02	.000	.500D-02

BOUNDARY AND LOADING CONDITIONS

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
.00	.000D+00
.00	.972D+04
1.00	.972D+04
1.00	.000D+00

LOADING NUMBER 1

BOUNDARY-CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .972D+04 LBS
MOMENT AT THE PILE HEAD = .291D+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .950D+04 LBS

FINITE-DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 100
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE = .100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100

GM2

MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE 5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
.00	.000D+00
.00	.972D+04
1.00	.972D+04
1.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .972D+04 LBS
MOMENT AT THE PILE HEAD = .291D+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .950D+04 LBS

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
.00	.164D+00	.291D+07	.972D+04	.000D+00	.193D+03	.152D+13
3.48	.161D+00	.294D+07	.972D+04	.000D+00	.195D+03	.152D+13
6.96	.158D+00	.298D+07	.972D+04	.000D+00	.197D+03	.152D+13
10.44	.155D+00	.301D+07	.972D+04	.000D+00	.199D+03	.152D+13
13.92	.151D+00	.305D+07	.972D+04	.000D+00	.201D+03	.152D+13
17.40	.148D+00	.308D+07	.972D+04	.000D+00	.204D+03	.152D+13
20.88	.145D+00	.311D+07	.972D+04	.000D+00	.206D+03	.152D+13
24.36	.142D+00	.315D+07	.972D+04	.000D+00	.208D+03	.152D+13
27.84	.139D+00	.318D+07	.972D+04	.000D+00	.210D+03	.152D+13
31.32	.136D+00	.321D+07	.972D+04	.000D+00	.212D+03	.152D+13
34.80	.133D+00	.325D+07	.972D+04	.000D+00	.214D+03	.152D+13
38.28	.130D+00	.328D+07	.972D+04	.000D+00	.217D+03	.152D+13
41.76	.127D+00	.332D+07	.972D+04	.000D+00	.219D+03	.152D+13
45.24	.124D+00	.335D+07	.972D+04	.000D+00	.221D+03	.152D+13
48.72	.121D+00	.338D+07	.972D+04	.000D+00	.223D+03	.152D+13
52.20	.118D+00	.342D+07	.972D+04	.000D+00	.225D+03	.152D+13
55.68	.115D+00	.345D+07	.972D+04	.000D+00	.228D+03	.152D+13
59.16	.112D+00	.349D+07	.972D+04	.000D+00	.230D+03	.152D+13
62.64	.110D+00	.352D+07	.969D+04	-.185D+02	.232D+03	.152D+13
66.12	.107D+00	.355D+07	.958D+04	-.419D+02	.234D+03	.152D+13
69.60	.104D+00	.359D+07	.940D+04	-.640D+02	.236D+03	.152D+13
73.08	.101D+00	.362D+07	.914D+04	-.849D+02	.238D+03	.152D+13
76.56	.987D-01	.365D+07	.881D+04	-.105D+03	.240D+03	.152D+13
80.04	.961D-01	.368D+07	.841D+04	-.123D+03	.242D+03	.152D+13

83.52	.934D-01	.371D+07	.795D+04	-.141D+03	.244D+03	.152D+13
87.00	.908D-01	.374D+07	.744D+04	-.157D+03	.246D+03	.152D+13
90.48	.883D-01	.376D+07	.686D+04	-.172D+03	.248D+03	.152D+13
93.96	.857D-01	.378D+07	.624D+04	-.186D+03	.249D+03	.152D+13
97.44	.832D-01	.380D+07	.557D+04	-.199D+03	.250D+03	.152D+13
100.92	.807D-01	.382D+07	.485D+04	-.211D+03	.252D+03	.152D+13
104.40	.783D-01	.384D+07	.410D+04	-.222D+03	.253D+03	.152D+13
107.88	.759D-01	.385D+07	.331D+04	-.232D+03	.253D+03	.152D+13
111.36	.735D-01	.386D+07	.248D+04	-.241D+03	.254D+03	.152D+13
114.84	.711D-01	.387D+07	.163D+04	-.250D+03	.255D+03	.152D+13
118.32	.688D-01	.387D+07	.747D+03	-.257D+03	.255D+03	.152D+13
121.80	.665D-01	.387D+07	-.157D+03	-.263D+03	.255D+03	.152D+13
125.28	.642D-01	.387D+07	-.108D+04	-.268D+03	.255D+03	.152D+13
128.76	.620D-01	.387D+07	-.202D+04	-.273D+03	.254D+03	.152D+13
132.24	.598D-01	.386D+07	-.298D+04	-.276D+03	.254D+03	.152D+13
135.72	.576D-01	.384D+07	-.394D+04	-.279D+03	.253D+03	.152D+13
139.20	.555D-01	.383D+07	-.492D+04	-.281D+03	.252D+03	.152D+13
142.68	.533D-01	.381D+07	-.590D+04	-.282D+03	.251D+03	.152D+13
146.16	.513D-01	.379D+07	-.688D+04	-.283D+03	.249D+03	.152D+13
149.64	.492D-01	.376D+07	-.787D+04	-.282D+03	.248D+03	.152D+13
153.12	.472D-01	.373D+07	-.885D+04	-.281D+03	.246D+03	.152D+13
156.60	.452D-01	.370D+07	-.982D+04	-.279D+03	.244D+03	.152D+13
160.08	.432D-01	.367D+07	-.108D+05	-.277D+03	.241D+03	.152D+13
163.56	.413D-01	.363D+07	-.117D+05	-.274D+03	.239D+03	.152D+13
167.04	.394D-01	.358D+07	-.127D+05	-.270D+03	.236D+03	.152D+13
170.52	.375D-01	.354D+07	-.136D+05	-.265D+03	.233D+03	.152D+13
174.00	.357D-01	.349D+07	-.145D+05	-.260D+03	.230D+03	.152D+13
177.48	.339D-01	.344D+07	-.154D+05	-.255D+03	.227D+03	.152D+13
180.96	.321D-01	.338D+07	-.163D+05	-.248D+03	.223D+03	.152D+13
184.44	.303D-01	.332D+07	-.172D+05	-.241D+03	.219D+03	.152D+13
187.92	.286D-01	.326D+07	-.180D+05	-.234D+03	.215D+03	.152D+13
191.40	.268D-01	.320D+07	-.188D+05	-.226D+03	.211D+03	.152D+13
194.88	.252D-01	.313D+07	-.196D+05	-.217D+03	.207D+03	.152D+13
198.36	.235D-01	.306D+07	-.203D+05	-.208D+03	.202D+03	.152D+13
201.84	.219D-01	.299D+07	-.210D+05	-.198D+03	.198D+03	.152D+13
205.32	.202D-01	.292D+07	-.217D+05	-.188D+03	.193D+03	.152D+13
208.80	.187D-01	.284D+07	-.223D+05	-.178D+03	.188D+03	.152D+13
212.28	.171D-01	.276D+07	-.229D+05	-.167D+03	.183D+03	.152D+13
215.76	.155D-01	.268D+07	-.235D+05	-.155D+03	.178D+03	.152D+13
219.24	.140D-01	.260D+07	-.240D+05	-.143D+03	.172D+03	.152D+13
222.72	.125D-01	.251D+07	-.245D+05	-.130D+03	.167D+03	.152D+13
226.20	.110D-01	.243D+07	-.249D+05	-.117D+03	.161D+03	.152D+13
229.68	.956D-02	.234D+07	-.253D+05	-.104D+03	.156D+03	.152D+13
233.16	.812D-02	.225D+07	-.256D+05	-.900D+02	.150D+03	.152D+13
236.64	.669D-02	.216D+07	-.259D+05	-.756D+02	.144D+03	.152D+13
240.12	.528D-02	.207D+07	-.261D+05	-.609D+02	.138D+03	.152D+13
243.60	.389D-02	.198D+07	-.263D+05	-.457D+02	.132D+03	.152D+13
247.08	.251D-02	.189D+07	-.265D+05	-.300D+02	.126D+03	.152D+13
250.56	.114D-02	.179D+07	-.265D+05	-.139D+02	.120D+03	.152D+13
254.04	-.205D-03	.170D+07	-.266D+05	.255D+01	.114D+03	.152D+13
257.52	-.154D-02	.161D+07	-.265D+05	.195D+02	.108D+03	.152D+13
261.00	-.286D-02	.152D+07	-.264D+05	.368D+02	.102D+03	.152D+13
264.48	-.417D-02	.143D+07	-.263D+05	.546D+02	.965D+02	.152D+13
267.96	-.547D-02	.134D+07	-.260D+05	.728D+02	.906D+02	.152D+13
271.44	-.676D-02	.125D+07	-.258D+05	.915D+02	.848D+02	.152D+13
274.92	-.804D-02	.116D+07	-.254D+05	.111D+03	.790D+02	.152D+13
278.40	-.931D-02	.107D+07	-.250D+05	.130D+03	.733D+02	.152D+13
281.88	-.106D-01	.982D+06	-.245D+05	.150D+03	.677D+02	.152D+13
285.36	-.118D-01	.898D+06	-.239D+05	.171D+03	.623D+02	.152D+13
288.84	-.131D-01	.815D+06	-.233D+05	.191D+03	.569D+02	.152D+13
292.32	-.143D-01	.735D+06	-.226D+05	.213D+03	.518D+02	.152D+13
295.80	-.155D-01	.658D+06	-.218D+05	.235D+03	.468D+02	.152D+13
299.28	-.168D-01	.584D+06	-.210D+05	.257D+03	.419D+02	.152D+13

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302.76	-.180D-01	.512D+06	-.200D+05	.280D+03	.373D+02	.152D+13
306.24	-.192D-01	.444D+06	-.190D+05	.303D+03	.329D+02	.152D+13
309.72	-.204D-01	.380D+06	-.179D+05	.326D+03	.287D+02	.152D+13
313.20	-.216D-01	.319D+06	-.168D+05	.351D+03	.248D+02	.152D+13
316.68	-.229D-01	.263D+06	-.155D+05	.375D+03	.212D+02	.152D+13
320.16	-.241D-01	.211D+06	-.141D+05	.401D+03	.178D+02	.152D+13
323.64	-.253D-01	.165D+06	-.127D+05	.426D+03	.148D+02	.152D+13
327.12	-.265D-01	.123D+06	-.112D+05	.453D+03	.121D+02	.152D+13
330.60	-.277D-01	.869D+05	-.955D+04	.479D+03	.978D+01	.152D+13
334.08	-.289D-01	.566D+05	-.784D+04	.507D+03	.781D+01	.152D+13
337.56	-.301D-01	.324D+05	-.603D+04	.534D+03	.625D+01	.152D+13
341.04	-.313D-01	.147D+05	-.412D+04	.563D+03	.510D+01	.152D+13
344.52	-.325D-01	.375D+04	-.211D+04	.592D+03	.439D+01	.152D+13
348.00	-.337D-01	.000D+00	.000D+00	.621D+03	.415D+01	.152D+13

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.368D-04 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.547D-05 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .164D+00 IN
MAXIMUM BENDING MOMENT = .387D+07 LBS-IN
MAXIMUM SHEAR FORCE = -.266D+05 LBS
NO. OF ITERATIONS = 4
NO. OF ZERO DEFLECTION POINTS = 1

S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.9720D+04	.2910D+07	.9500D+04	.1642D+00	.3873D+07	-.2656D+05

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**************************************************************
*    PROGRAM LPILE1                                          *
*    (C) COPYRIGHT 1986 ENSOFT, INC.                         *
*    ALL RIGHTS RESERVED                                     *
*   ---------------------------------------------------      *
*                                                            *
*                  PREPARED ESPECIALLY FOR                   *
*                                                            *
*               STATE DEPARTMENT OF HIGHWAYS                 *
*                                                            *
*                  DENVER, COLORADO 80222                    *
*                                                            *
*                     LICENSE NO. 138                        *
*                                                            *
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PROGRAM LPILE1

sign structure caisson 10' exposed

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N
****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 408.00 IN
4 POINTS

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2	MODULUS OF ELASTICITY LBS/IN**2
.00	54.000	.417D+06	.229D+04	.364D+07
120.00	54.000	.417D+06	.229D+04	.364D+07
120.00	54.000	.417D+06	.229D+04	.364D+07
408.00	54.000	.417D+06	.229D+04	.364D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = 120.00 IN

2 LAYER(S) OF SOIL

LAYER 1

```
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER         =     120.00 IN
X AT THE BOTTOM OF THE LAYER      =     500.00 IN
MODULUS OF SUBGRADE REACTION      =   .640D+02 LBS/IN**3

LAYER  2
THE SOIL IS A STIFF CLAY WITH NO FREE WATER
X AT THE TOP OF THE LAYER         =     500.00 IN
X AT THE BOTTOM OF THE LAYER      =    1000.00 IN
MODULUS OF SUBGRADE REACTION      =   .200D+04 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                   8 POINTS
        X,IN        WEIGHT,LBS/IN**3
        1.00          .00D+00
      120.00          .00D+00
      120.00          .67D-01
      414.00          .67D-01
      414.00          .69D-01
      690.00          .69D-01
      690.00          .78D-01
     1000.00          .78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                   8 POINTS
        X,IN        C,LBS/IN**2      PHI,DEGREES       E50
        1.00         .000D+00        .000D+00         -----
      120.00         .000D+00        .000D+00         -----
      120.00         .000D+00        .320D+02         -----
      414.00         .000D+00        .320D+02         -----
      414.00         .000D+00        .320D+02         -----
      690.00         .000D+00        .320D+02         -----
      690.00         .694D+02            .000         .500D-02
     1000.00         .694D+02            .000         .500D-02


BOUNDARY AND LOADING CONDITIONS
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DISTRIBUTED LOAD CURVE             5 POINTS
                      X,IN        LOAD,LBS/IN
                      1.00          .000D+00
                       .00          .000D+00
                       .00          .972D+04
                      1.00          .972D+04
                      1.00          .000D+00


LOADING NUMBER    1

BOUNDARY-CONDITION CODE                   =         1
LATERAL LOAD AT THE PILE HEAD             =   .972D+04 LBS
MOMENT AT THE PILE HEAD                   =   .291D+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD               =   .950D+04 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                               =       100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE        =   .100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS  =       100
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MAXIMUM ALLOWABLE DEFLECTION = .36D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N
****

DISTRIBUTED LOAD CURVE   5 POINTS

X,IN	LOAD,LBS/IN
1.00	.000D+00
.00	.000D+00
.00	.972D+04
1.00	.972D+04
1.00	.000D+00

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .972D+04 LBS
MOMENT AT THE PILE HEAD = .291D+07 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .950D+04 LBS

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
.00	.251D+00	.291D+07	.972D+04	.000D+00	.193D+03	.152D+13
4.08	.246D+00	.295D+07	.972D+04	.000D+00	.195D+03	.152D+13
8.16	.241D+00	.299D+07	.972D+04	.000D+00	.198D+03	.152D+13
12.24	.236D+00	.303D+07	.972D+04	.000D+00	.200D+03	.152D+13
16.32	.232D+00	.307D+07	.972D+04	.000D+00	.203D+03	.152D+13
20.40	.227D+00	.311D+07	.972D+04	.000D+00	.205D+03	.152D+13
24.48	.222D+00	.315D+07	.972D+04	.000D+00	.208D+03	.152D+13
28.56	.218D+00	.319D+07	.972D+04	.000D+00	.211D+03	.152D+13
32.64	.213D+00	.323D+07	.972D+04	.000D+00	.213D+03	.152D+13
36.72	.209D+00	.327D+07	.972D+04	.000D+00	.216D+03	.152D+13
40.80	.204D+00	.331D+07	.972D+04	.000D+00	.218D+03	.152D+13
44.88	.200D+00	.335D+07	.972D+04	.000D+00	.221D+03	.152D+13
48.96	.195D+00	.339D+07	.972D+04	.000D+00	.223D+03	.152D+13
53.04	.191D+00	.343D+07	.972D+04	.000D+00	.226D+03	.152D+13
57.12	.186D+00	.347D+07	.972D+04	.000D+00	.229D+03	.152D+13
61.20	.182D+00	.351D+07	.972D+04	.000D+00	.231D+03	.152D+13
65.28	.178D+00	.355D+07	.972D+04	.000D+00	.234D+03	.152D+13
69.36	.173D+00	.358D+07	.972D+04	.000D+00	.236D+03	.152D+13
73.44	.169D+00	.362D+07	.972D+04	.000D+00	.239D+03	.152D+13
77.52	.165D+00	.366D+07	.972D+04	.000D+00	.241D+03	.152D+13
81.60	.161D+00	.370D+07	.972D+04	.000D+00	.244D+03	.152D+13
85.68	.157D+00	.374D+07	.972D+04	.000D+00	.247D+03	.152D+13
89.76	.153D+00	.378D+07	.972D+04	.000D+00	.249D+03	.152D+13
93.84	.149D+00	.382D+07	.972D+04	.000D+00	.252D+03	.152D+13

97.92	.145D+00	.386D+07	.972D+04	.000D+00	.254D+03	.152D+13
102.00	.141D+00	.390D+07	.972D+04	.000D+00	.257D+03	.152D+13
106.08	.137D+00	.394D+07	.972D+04	.000D+00	.259D+03	.152D+13
110.16	.133D+00	.398D+07	.972D+04	.000D+00	.262D+03	.152D+13
114.24	.129D+00	.402D+07	.972D+04	.000D+00	.265D+03	.152D+13
118.32	.126D+00	.406D+07	.972D+04	.000D+00	.267D+03	.152D+13
122.40	.122D+00	.410D+07	.968D+04	-.187D+02	.270D+03	.152D+13
126.48	.118D+00	.414D+07	.954D+04	-.491D+02	.272D+03	.152D+13
130.56	.115D+00	.418D+07	.929D+04	-.776D+02	.275D+03	.152D+13
134.64	.111D+00	.422D+07	.891D+04	-.104D+03	.277D+03	.152D+13
138.72	.108D+00	.425D+07	.844D+04	-.129D+03	.279D+03	.152D+13
142.80	.104D+00	.429D+07	.787D+04	-.152D+03	.282D+03	.152D+13
146.88	.101D+00	.432D+07	.720D+04	-.173D+03	.284D+03	.152D+13
150.96	.974D-01	.434D+07	.645D+04	-.193D+03	.285D+03	.152D+13
155.04	.941D-01	.437D+07	.563D+04	-.211D+03	.287D+03	.152D+13
159.12	.908D-01	.439D+07	.474D+04	-.227D+03	.288D+03	.152D+13
163.20	.876D-01	.441D+07	.378D+04	-.242D+03	.290D+03	.152D+13
167.28	.844D-01	.442D+07	.276D+04	-.255D+03	.290D+03	.152D+13
171.36	.812D-01	.443D+07	.170D+04	-.267D+03	.291D+03	.152D+13
175.44	.781D-01	.443D+07	.588D+03	-.277D+03	.291D+03	.152D+13
179.52	.751D-01	.443D+07	-.562D+03	-.286D+03	.291D+03	.152D+13
183.60	.721D-01	.443D+07	-.174D+04	-.294D+03	.291D+03	.152D+13
187.68	.692D-01	.442D+07	-.295D+04	-.300D+03	.290D+03	.152D+13
191.76	.663D-01	.441D+07	-.419D+04	-.304D+03	.289D+03	.152D+13
195.84	.634D-01	.439D+07	-.544D+04	-.308D+03	.288D+03	.152D+13
199.92	.606D-01	.436D+07	-.670D+04	-.310D+03	.287D+03	.152D+13
204.00	.579D-01	.433D+07	-.796D+04	-.311D+03	.285D+03	.152D+13
208.08	.552D-01	.430D+07	-.923D+04	-.311D+03	.282D+03	.152D+13
212.16	.525D-01	.426D+07	-.105D+05	-.310D+03	.280D+03	.152D+13
216.24	.499D-01	.421D+07	-.118D+05	-.307D+03	.277D+03	.152D+13
220.32	.474D-01	.416D+07	-.130D+05	-.304D+03	.274D+03	.152D+13
224.40	.448D-01	.411D+07	-.142D+05	-.300D+03	.270D+03	.152D+13
228.48	.424D-01	.404D+07	-.154D+05	-.294D+03	.266D+03	.152D+13
232.56	.399D-01	.398D+07	-.166D+05	-.288D+03	.262D+03	.152D+13
236.64	.376D-01	.391D+07	-.178D+05	-.280D+03	.257D+03	.152D+13
240.72	.352D-01	.383D+07	-.189D+05	-.272D+03	.252D+03	.152D+13
244.80	.329D-01	.375D+07	-.200D+05	-.263D+03	.247D+03	.152D+13
248.88	.307D-01	.367D+07	-.211D+05	-.253D+03	.242D+03	.152D+13
252.96	.284D-01	.358D+07	-.221D+05	-.242D+03	.236D+03	.152D+13
257.04	.263D-01	.349D+07	-.230D+05	-.230D+03	.230D+03	.152D+13
261.12	.241D-01	.339D+07	-.240D+05	-.218D+03	.224D+03	.152D+13
265.20	.220D-01	.330D+07	-.248D+05	-.205D+03	.218D+03	.152D+13
269.28	.200D-01	.319D+07	-.256D+05	-.191D+03	.211D+03	.152D+13
273.36	.179D-01	.309D+07	-.264D+05	-.176D+03	.204D+03	.152D+13
277.44	.159D-01	.298D+07	-.271D+05	-.160D+03	.197D+03	.152D+13
281.52	.140D-01	.287D+07	-.277D+05	-.144D+03	.190D+03	.152D+13
285.60	.120D-01	.275D+07	-.282D+05	-.127D+03	.182D+03	.152D+13
289.68	.101D-01	.264D+07	-.287D+05	-.110D+03	.175D+03	.152D+13
293.76	.825D-02	.252D+07	-.291D+05	-.917D+02	.167D+03	.152D+13
297.84	.640D-02	.240D+07	-.295D+05	-.729D+02	.159D+03	.152D+13
301.92	.458D-02	.228D+07	-.297D+05	-.534D+02	.152D+03	.152D+13
306.00	.279D-02	.215D+07	-.299D+05	-.332D+02	.144D+03	.152D+13
310.08	.102D-02	.203D+07	-.300D+05	-.124D+02	.136D+03	.152D+13
314.16	-.734D-03	.191D+07	-.300D+05	.913D+01	.128D+03	.152D+13
318.24	-.246D-02	.179D+07	-.299D+05	.313D+02	.120D+03	.152D+13
322.32	-.417D-02	.167D+07	-.297D+05	.540D+02	.112D+03	.152D+13
326.40	-.586D-02	.155D+07	-.295D+05	.774D+02	.104D+03	.152D+13
330.48	-.754D-02	.143D+07	-.291D+05	.102D+03	.965D+02	.152D+13
334.56	-.920D-02	.131D+07	-.286D+05	.126D+03	.888D+02	.152D+13
338.64	-.108D-01	.119D+07	-.281D+05	.152D+03	.813D+02	.152D+13
342.72	-.125D-01	.108D+07	-.274D+05	.178D+03	.740D+02	.152D+13
346.80	-.141D-01	.968D+06	-.266D+05	.205D+03	.669D+02	.152D+13
350.88	-.157D-01	.862D+06	-.257D+05	.232D+03	.599D+02	.152D+13

354.96	-.173D-01	.759D+06	-.247D+05	.260D+03	.533D+02	.152D+13
359.04	-.189D-01	.660D+06	-.236D+05	.289D+03	.469D+02	.152D+13
363.12	-.205D-01	.566D+06	-.224D+05	.319D+03	.408D+02	.152D+13
367.20	-.220D-01	.477D+06	-.210D+05	.349D+03	.351D+02	.152D+13
371.28	-.236D-01	.394D+06	-.195D+05	.380D+03	.297D+02	.152D+13
375.36	-.252D-01	.318D+06	-.179D+05	.412D+03	.247D+02	.152D+13
379.44	-.268D-01	.248D+06	-.162D+05	.444D+03	.202D+02	.152D+13
383.52	-.283D-01	.186D+06	-.143D+05	.477D+03	.162D+02	.152D+13
387.60	-.299D-01	.132D+06	-.123D+05	.512D+03	.127D+02	.152D+13
391.68	-.314D-01	.861D+05	-.101D+05	.546D+03	.972D+01	.152D+13
395.76	-.330D-01	.494D+05	-.780D+04	.582D+03	.735D+01	.152D+13
399.84	-.345D-01	.224D+05	-.535D+04	.619D+03	.560D+01	.152D+13
403.92	-.361D-01	.576D+04	-.275D+04	.656D+03	.452D+01	.152D+13
408.00	-.377D-01	.000D+00	.000D+00	.694D+03	.415D+01	.152D+13

OUTPUT VERIFICATION

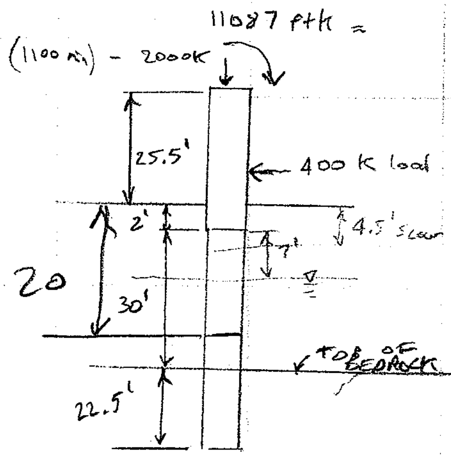
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.481D-04 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .629D-05 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .251D+00 IN
MAXIMUM BENDING MOMENT = .443D+07 LBS-IN
MAXIMUM SHEAR FORCE = -.300D+05 LBS
NO. OF ITERATIONS = 4
NO. OF ZERO DEFLECTION POINTS = 1

S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.9720D+04	.2910D+07	.9500D+04	.2507D+00	.4435D+07	-.3000D+05



$\gamma_t = 120\ lb/ft^3 = .0694\ lb/in^3$
$\phi = 32°$
$c = 0$
$K_n = 64\ lb/in^3$

$\gamma_t = 135\ lb/ft^3 = .0781\ lb/in^2$
$\phi = 0$
$c = 10000\ lb/ft^2 = 69.44\ lb/in^2$
$K_1 = 2000\ lb/in^3$
$\varepsilon_{50} = .005$

LPILE INPUT

LINE 1: TITLE

LINE 2: UNITS 1 = ENGLISH

LINE 3: VARIABLES; NI, NDIAM, XGS, XLN

NI = # of Increments = 100

NDIAM = # of segment with different ϕ dia or E = 1

XGS ≈ 27.5' = 330"

XLN ≈ 80' ≈ 960"

LINE 4: XDIAM(I), DIAM(I), MINERT(I) AREA(I), EPILE(I)

XDIAM = x coordinate = 0 TOP, 330" BOTTOM } COLUMN
330" TOP, 960 BOTTOM } CAISSON

DIAM = DIAMETER OF PILE = 54" - COLUMN
54" - CAISSON

MINERT = MOMENT of INERTIA = $\frac{\pi d^4}{64}$ = 417393

AREA = $\pi d^2/4$ = 2290.2

EPILE ≈ 3640 Ksi -

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

LINE 5: NL, NG1, NSTR, NPY

NL = # of soil layers = 2
NG1 = # of points on plot = 8
NSTR = # of points in input = 8
NPY = # of input p-y curves = 0

LINE 6: KSOIL (I), XTOP(I), XBOT(I), XK(I)

KSOIL - PY curve code
4 - sand
3 - stiff clay

XTOP I = 306" → 690 sand — 360" for scour condition

XBOT I = 690 → 10000 stiff clay

XK = KN = 64 lb/in³ sand
K_n = 2000 lb/in³ clay

LINE 7: XGI (I), GAM(I)

XG1 = location

GAM = unit weight = .0694 lb/in³ dry — .0723 lb/in³ wt
.0781 lb/in³ bedrock

LINE 8: XSTR (I), C1(I), PHI(I), EE50(I)

Soil 1 XSTR = 306 - 690 — 360" for scour

C = 0
PHI = 32
EE50 = 0

Soil 2 XSTR = 690 - 1000

C = 10000 lb/ft³ = 69.44 lb/in²

PHI = 0 EE50 = .005

LINE 12: NW = 5
KYCL = 1
RCYCL = 0

By:	Date	Project no.	Project code (SA#):
			Sheet 652 of
Chk'd:	Date	Structure no.	CDOT Form #1034

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

LINE 13: XW(I), WW(I)

no load for scour ?

0 0

60"

246, 0

SOIL @ 306"

246, 16666.7 lb/in } 400 kip load distributed over 2'

270, 16666.7 lb/in

4' above 258"

270, 0

LINE 14: NLPS = 1

LINE 15: KBC1(I) BC1(I), BC2(I), Q(I)

Q = axial load ≈ 1630 max — factored no live load

BC1 = ~~displacement = 0~~ assumed lateral load

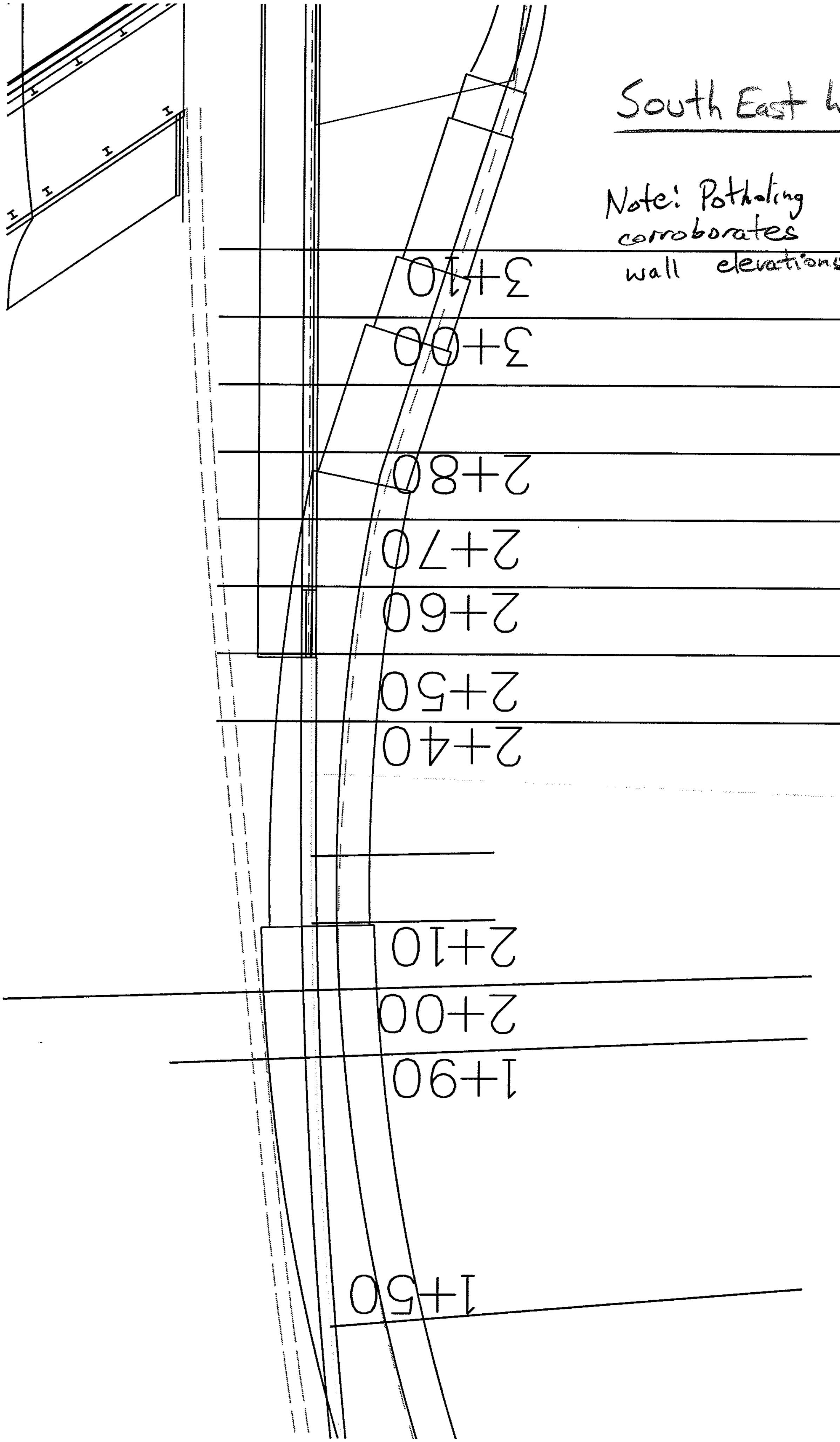
BC2 = moment ≈ 1480 ft kips = 17760 in kips

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 653 of ___

CDOT Form #1034

South East Wall

Note: Potholing info corroborates assumed wall elevations



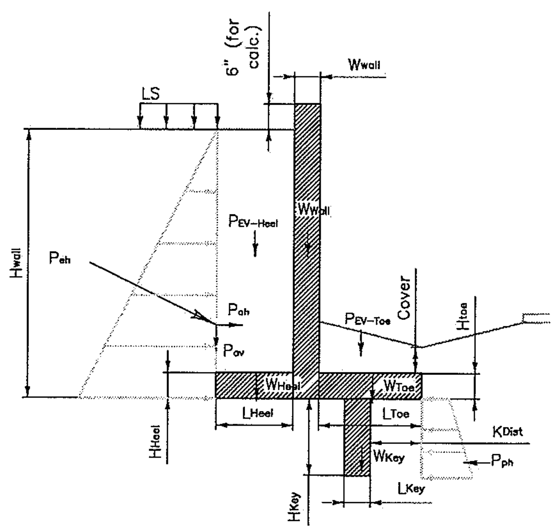
WALL LAYOUT

Given:

InSitu Material:	γ=	120	pcf
	C=	0.40	ksf
Backfill:	γ=	125	pcf
	Φ=	34.00	degrees
	slope of backfill=	10	degrees
	Ka=	0.28	
Wall Dimensions	Hwall=	26.00	ft
	Wwall=	1.50	ft
	Ltoe=	5.58	ft
	Htoe=	2.00	ft
	Lheel=	9.67	ft
	Hheel=	2.00	ft
	Lkey=	2.00	ft
	Hkey=	1.00	ft
	Kdist=	4.61	ft
Surcharge	LS=	2.00	ft

Ko= 0.44

γconc= 150 pcf



Assume Depth over Toe= 2 ft

LRFD DESIGN:

qult=	24.00	Ksf		(from Geotech Report)
Φbearing=	0.5			(from Geotech Report)
C=	unknown	Ksf	(cohesive soil strength)	
f=	0.45		(interface friction coefficient)	(from Geotech Report)
Φsliding=	0.8			(from Table 10.5.5-1)

ASD DESIGN:

Qallow=	6.00	Ksf		(from Geotech Report)
f=	0.45		(interface friction coefficient)	(from Geotech Report)
FSsliding=	1.5			
FSbearing=	3			

SUMMARY OF ECCENTRICITY CHECK

B= 16.75 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	X_O	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	53.30	15.06	659.98	143.81	9.68	-1.31	4.19	OKAY
STRENGTH I-B	73.36	15.06	864.15	143.81	9.82	-1.44	4.19	OKAY
STRENGTH IV	71.84	12.76	796.35	114.03	9.50	-1.12	4.19	OKAY
SERVICE I	53.14	9.82	615.84	94.55	9.81	-1.43	4.19	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 6.51

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	M_{vTOT}	M_{hTOT}	X_O	γq	q_r	
	K/ft	K/ft	K/ft	K/ft	Ksf	Ksf	
STRENGTH I-A	53.30	659.98	143.81	9.68	2.75	12.00	OKAY
STRENGTH I-B	73.36	864.15	143.81	9.82	3.74	12.00	OKAY
STRENGTH IV	71.84	796.35	114.03	9.50	3.78	12.00	OKAY
SERVICE I	53.14	615.84	94.55	9.81	2.71	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 8.86

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	γq_{max}	γq_{min}	$\gamma q_{max}/2$	Q_T	Q_{EP}	$\Phi_T Q_T + \Phi_{EP} Q_{EP}$	H_{TOT}	
	K/ft	Ksf	Ksf	Ksf	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	53.30	4.67	1.69	2.34	26.65	5.10	23.87	15.06	OKAY
STRENGTH I-B	73.36	6.65	2.11	3.32	36.68	5.10	31.89	15.06	OKAY
STRENGTH IV	71.84	6.01	2.56	3.01	35.92	5.10	31.29	12.76	OKAY
SERVICE I	53.14	4.80	1.54	2.40	26.57	5.67	24.09	9.82	OKAY
ASD	51.75	3.09	3.09	1.54	25.87	5.67	23.53	9.82	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 2.71
$FS_{sliding}$= 2.4

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	H_{TOT}
	K/ft	K/ft	K/ft
H (UNFACTORED)	1.39	8.36	9.75
STRENGTH I-A	2.44	12.54	14.97
STRENGTH I-B	2.44	12.54	14.97
STRENGTH IV	0.00	12.54	12.54
SERVICE I	1.39	8.36	9.75

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft
Mh (UNFACTORED)	16.71	66.85	83.57
STRENGTH I-A	0.00	100.28	100.28
STRENGTH I-B	29.25	100.28	129.53
STRENGTH IV	0.00	100.28	100.28
SERVICE I	16.71	66.85	83.57

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	83.57
STRENGTH I-A	100.28
STRENGTH I-B	129.53
STRENGTH IV	100.28
SERVICE I	83.57

$AS_{req'd}$
in^2
2.74
3.40
4.70
3.40
2.74

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
80.09	78.34	76.60
120.13	117.52	114.90
120.13	117.52	114.90
120.13	117.52	114.90
80.09	78.34	76.60
	steel okay 1.5' up	steel okay 2' up

ds= 15.30 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches
Stem Thickness = 18.00 inches 18.00 inches default thickness
Use # 11 at back face
use spacing= 5.25 inches
As= 3.57 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 5.18
de=ds= 15.30 inches (for no prestressing)
c/de= 0.34 member is not overreinforced per 5.7.3.3.1-1

Mn= 154.30 ft-K/ft
steel okay
steel okay 1' up

Vn = 21.28 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 26.94 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 172.27 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.94	5.64	6.58
STRENGTH I-A	1.64	8.45	10.10
STRENGTH I-B	1.64	8.45	10.10
STRENGTH IV	1.64	8.45	10.10
SERVICE I	0.94	5.64	6.58

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV-heel}$	P_{av}	$\gamma q_{max}'-\gamma q_{min}$	γq_{min}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	11.69	9.08	140.26	54.51	-29.33	-72.12	14.03	114.09
STRENGTH I-A	0.00	15.90	140.26	81.76	-26.84	-79.04	17.53	132.04
STRENGTH I-B	20.46	15.90	189.36	81.76	-40.76	-98.87	17.53	167.84
STRENGTH IV	0.00	0.00	189.36	81.76	-31.04	-119.90	21.04	120.18
SERVICE I	11.69	9.08	140.26	54.51	-29.33	-72.12	14.03	114.09

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mv (UNFACTORED)	114.09
STRENGTH I-A	132.04
STRENGTH I-B	167.84
STRENGTH IV	120.18
SERVICE I	114.09

$AS_{req'd}$
in^2
2.57
3.02
3.96
2.72
2.57

d= 21.37 inches

per 5.10.8.2 — AS_{temp}= 0.43 sq inches

Heel Thickness = 24.00 inches — 24.00 inches default thickness
Use # 10 at top
use spacing= 5.25 inches
As= 2.90 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 4.22
de=ds= 21.37 inches (for no prestressing)
c/de= 0.20 member is not overreinforced per 5.7.3.3.1-1

Mn= 188.47 ft-K/ft
steel okay

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV-heel}$	P_{av}	$\gamma q_{max}'-\gamma q_{min}$	γq_{min}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	2.42	0.94	29.01	5.64	-9.10	-14.92	2.90	13.99
STRENGTH I-A	0.00	1.64	29.01	8.45	-8.33	-16.35	3.63	14.43
STRENGTH I-B	4.23	1.64	39.16	8.45	-12.65	-20.45	3.63	20.40

STRENGTH IV	0.00	0.00	39.16	8.45	-9.63	-24.80	4.35	13.19
SERVICE I	2.42	0.94	29.01	5.64	-9.10	-14.92	2.90	13.99

Vn = 28.37 K/ft

Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 47.89 ft-K <--- Test 1

1.33MhTOT (max.)= 223.22 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

TOE REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}	γq_{min}	W_{Toe}
	K/ft	Ksf	Ksf	K/ft
V (UNFACTORED)	1.40	4.80	1.54	1.67
STRENGTH I-A	0.00	4.67	1.69	1.51
STRENGTH I-B	1.88	6.65	2.11	2.09
STRENGTH IV	1.88	6.01	2.56	2.51
SERVICE I	1.40	4.80	1.54	1.67

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-3.89	11.27	57.86	-4.67	60.57
STRENGTH I-A	0.00	10.32	57.29	-4.20	63.41
STRENGTH I-B	-5.25	15.66	79.96	-5.84	84.53
STRENGTH IV	-5.25	11.93	75.73	-7.01	75.40
SERVICE I	-3.89	11.27	57.86	-4.67	60.57

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	60.57	1.38
STRENGTH I-A	63.41	1.45
STRENGTH I-B	84.53	1.97
STRENGTH IV	75.40	1.75
SERVICE I	60.57	1.38

d= 20.30 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Toe Thickness = 24.00 inches — 24.00 inches default thickness
Use # 11 at bottom
use spacing= 6.00 inches
As= 3.12 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

c= 4.53
d_e=d_s= 20.30 inches (for no prestressing)
c/d_e= 0.22 member is not overreinforced per 5.7.3.3.1-1

Mn= 200.44 ft-K/ft
steel okay

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-1.40	3.03	20.74	-1.67	20.70
STRENGTH I-A	0.00	2.77	20.54	-1.51	21.80
STRENGTH I-B	-5.25	4.21	28.66	-2.09	25.52
STRENGTH IV	-5.25	3.21	27.14	-2.51	22.59

SERVICE I	-3.89	3.03	20.74	-1.67	18.20

Vn = 28.37 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$=	47.89	ft-K	<--- Test 1
1.33M_{hTOT} (max.)=	112.42	ft-K	<--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	3.78
STRENGTH I-A	5.10
STRENGTH I-B	5.10
STRENGTH IV	5.10
SERVICE I	3.78

$AS_{req'd}$
in^2
0.08
0.10
0.10
0.10
0.08

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

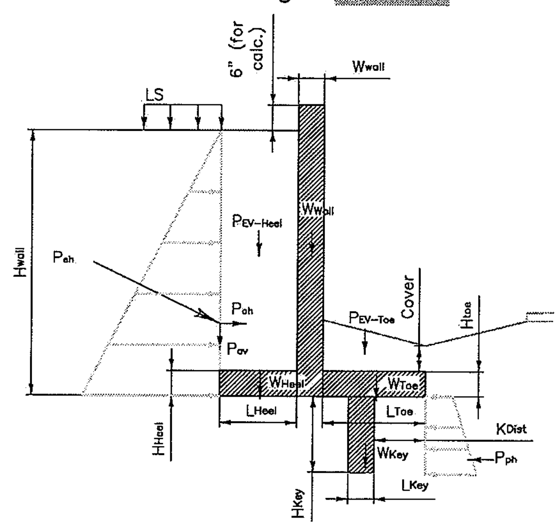
c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 2+10

Given:

InSitu Material:	γ=	120	pcf	
Backfill:	γ=	125	pcf	
	Φ=	34.00	degrees	angle of internal friction
	δ=	31.00	degrees	friction angle between fill & wall
	slope of backfill (β) =	5	degrees	
	angle of wall to horizontal (Θ) =	87	degrees	
	Γ =	2.99		
	Ka=	0.298		K_o= 0.44
Wall Dimensions	Hwall=	26.00	ft	γ_{conc}= 150 pcf
	Wwall=	1.83	ft	
	Ltoe=	4.33	ft	
	Htoe=	1.83	ft	
	Lheel=	8.25	ft	
	Hheel=	1.83	ft	
	Lkey=	2.00	ft	
	Hkey=	1.00	ft	
	Kdist=	8.56	ft	
Surcharge	LS=	3.50	ft	
	Dist to surcharge=	4.25	ft	



Assume Depth over Toe= 2 ft

LRFD DESIGN:

q_{ult}=	24.00	Ksf		(from Geotech Report)
$\Phi_{bearing}$=	0.5			(from Geotech Report)
C=	unknown	Ksf	(cohesive soil strength)	
f=	0.45		(interface friction)	(from Geotech Report)
$\Phi_{sliding}$=	0.8			(from Table 10.5.5-1)

ASD DESIGN:

Qallow=	6.00	Ksf		(from As-Built Dwgs)
f=	0.45		(interface friction)	(from Geotech Report)
$FS_{sliding}$=	1.5			
$FS_{bearing}$=	3			

SUMMARY OF ECCENTRICITY CHECK (NHI)

B= 14.41 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	X_O	e_b	e_{max}		e_{LRFD}
	K/ft	K/ft	K/ft	K/ft	ft	ft	ft		ft
STRENGTH I-A	37.01	24.69	347.68	245.17	2.77	4.44	3.60	[redacted]	[redacted]
STRENGTH I-B	54.12	24.69	501.98	245.17	4.75	2.46	3.60	OKAY	1.72
STRENGTH IV	53.81	18.78	473.26	168.47	5.66	1.54	3.60	OKAY	0.53
SERVICE I	40.18	15.89	367.34	158.66	5.19	2.01	3.60	OKAY	1.17

ASD Check ===> OKAY $FS_{overturning}$= 2.32

SUMMARY OF FACTORED BEARING PRESSURES (NHI)

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	M_{vTOT}	M_{hTOT}	X_O	γq	q_r		σ_{LRFD}
	K/ft	K/ft	K/ft	K/ft	Ksf	Ksf		Ksf
STRENGTH I-A	37.01	347.68	245.17	2.77	6.68	12.00	OKAY	5.15139
STRENGTH I-B	54.12	501.98	245.17	4.75	5.70	12.00	OKAY	4.93228
STRENGTH IV	53.81	473.26	168.47	5.66	4.75	12.00	OKAY	4.03083
SERVICE I	40.18	367.34	158.66	5.19	3.87	12.00	OKAY	3.32794

ASD Check ===> OKAY $FS_{bearing}$= 6.21

SUMMARY OF SLIDING RESISTANCE (NHI)

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
Φ_T= 0.8 (from Table 10.5.5-1) δfriction= 24.00 degrees
Φ_{EP}= 0.5 (from Table 10.5.5-1) tanδ= 0.45

LOAD CASE	V_{TOT}	γq_{max}	γq_{min}	$\gamma q_{max}/2$	Q_T	Q_{EP}	$\Phi_T Q_T + \Phi_{EP} Q_{EP}$	H_{TOT}	
	K/ft	Ksf	Ksf	Ksf	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	37.01	7.31	-2.17	3.66	18.50	8.50	19.05	24.69	[redacted]
STRENGTH I-B	54.12	7.60	-0.09	3.80	27.06	8.50	25.90	24.69	OKAY
STRENGTH IV	53.81	6.13	1.34	3.07	26.91	8.50	25.77	18.78	OKAY
SERVICE I	40.18	5.12	0.45	2.56	20.09	9.44	20.79	15.89	OKAY
ASD	39.09	2.71	2.71	1.36	19.55	9.44	20.36	15.89	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.26
$FS_{sliding}$= 1.7

LRFD LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	37.01	16.65	8.50	17.57	24.69	[redacted]
STRENGTH I-B	54.12	24.35	8.50	23.73	24.69	
STRENGTH IV	53.81	24.22	8.50	23.62	18.78	OKAY
SERVICE I	40.18	18.08	9.44	19.18	15.89	OKAY

ASD	39.09	17.59	9.44	18.79	15.89	OKAY

SUMMARY OF ECCENTRICITY CHECK (NHI)

B= 14.41 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	X_O	e_b	e_{max}		e_{LRFD}
	K/ft	K/ft	K/ft	K/ft	ft	ft	ft		ft
STRENGTH I-A	37.01	24.69	347.68	245.17	2.77	4.44	3.60	[redacted]	[redacted]
STRENGTH I-B	54.12	24.69	501.98	245.17	4.75	2.46	3.60	OKAY	1.72
STRENGTH IV	53.81	18.78	473.26	168.47	5.66	1.54	3.60	OKAY	0.53
SERVICE I	40.18	15.89	367.34	158.66	5.19	2.01	3.60	OKAY	1.17

ASD Check ===> OKAY $FS_{overturning}$= 2.32

SUMMARY OF FACTORED BEARING PRESSURES (NHI)

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	M_{vTOT}	M_{hTOT}	X_O	γq	q_r		σ_{LRFD}
	K/ft	K/ft	K/ft	K/ft	Ksf	Ksf		Ksf
STRENGTH I-A	37.01	347.68	245.17	2.77	6.68	12.00	OKAY	5.15139
STRENGTH I-B	54.12	501.98	245.17	4.75	5.70	12.00	OKAY	4.93228
STRENGTH IV	53.81	473.26	168.47	5.66	4.75	12.00	OKAY	4.03083
SERVICE I	40.18	367.34	158.66	5.19	3.87	12.00	OKAY	3.32794

ASD Check ===> OKAY $FS_{bearing}$= 6.21

SUMMARY OF SLIDING RESISTANCE (NHI)

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
Φ_T= 0.8 (from Table 10.5.5-1) δfriction= 24.00 degrees
Φ_{EP}= 0.5 (from Table 10.5.5-1) tanδ= 0.45

LOAD CASE	V_{TOT}	γq_{max}	γq_{min}	$\gamma q_{max}/2$	Q_T	Q_{EP}	$\Phi_T Q_T + \Phi_{EP} Q_{EP}$	H_{TOT}	
	K/ft	Ksf	Ksf	Ksf	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	37.01	7.31	-2.17	3.66	18.50	8.50	19.05	24.69	[redacted]
STRENGTH I-B	54.12	7.60	-0.09	3.80	27.06	8.50	25.90	24.69	OKAY
STRENGTH IV	53.81	6.13	1.34	3.07	26.91	8.50	25.77	18.78	OKAY
SERVICE I	40.18	5.12	0.45	2.56	20.09	9.44	20.79	15.89	OKAY
ASD	39.09	2.71	2.71	1.36	19.55	9.44	20.36	15.89	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.26
$FS_{sliding}$= 1.7

LRFD LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	37.01	16.65	8.50	17.57	24.69	[redacted]
STRENGTH I-B	54.12	24.35	8.50	23.73	24.69	
STRENGTH IV	53.81	24.22	8.50	23.62	18.78	OKAY
SERVICE I	40.18	18.08	9.44	19.18	15.89	OKAY
ASD	39.09	17.59	9.44	18.79	15.89	OKAY

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	H_{TOT}
	K/ft	K/ft	K/ft
H (UNFACTORED)	2.45	8.44	10.89
STRENGTH I-A	4.28	12.66	16.94
STRENGTH I-B	4.28	12.66	16.94
STRENGTH IV	0.00	12.66	12.66
SERVICE I	2.45	8.44	10.89

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft
Mh (UNFACTORED)	29.55	68.02	97.57
STRENGTH I-A	0.00	102.03	102.03
STRENGTH I-B	51.71	102.03	153.74
STRENGTH IV	0.00	102.03	102.03
SERVICE I	29.55	68.02	97.57

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	97.57
STRENGTH I-A	102.03
STRENGTH I-B	153.74
STRENGTH IV	102.03
SERVICE I	97.57

$AS_{req'd}$
in^2
2.45
2.57
4.11
2.57
2.45

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
93.53	91.51	89.50
140.30	137.27	134.24
140.30	137.27	134.24
140.30	137.27	134.24
93.53	91.51	89.50
	steel okay 1.5' up	steel okay 2' up

d_s= 19.26 inches

per 5.10.8.2
AS_{temp}= 0.40 sq inches
Stem Thickness = 23.70 inches
21.96 inches default thickness
Use # 11 at back face
use spacing= 6.00 inches
As= 3.12 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 4.53
d_e=d_s= 19.26 inches (for no prestressing)
c/d_e= 0.24 member is not overreinforced per 5.7.3.3.1-1

Mn= 179.23 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 8.17 inches
fs = [redacted] ksi

Vn = 28.02 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 46.70 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 204.48 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.27	0.95	1.22
STRENGTH I-A	0.48	1.42	1.90
STRENGTH I-B	0.48	1.42	1.90
STRENGTH IV	0.48	1.42	1.90
SERVICE I	0.27	0.95	1.22

σ_{LRFD}	e_{LRFD}
Ksf	ft
3.33	1.17
5.15	3.61
4.93	1.72
4.03	0.53
3.33	1.17

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	10.94	2.26	102.82	7.81	-58.16	9.34	75.01
STRENGTH I-A	0.00	3.96	102.82	11.72	-2.70	11.68	127.47
STRENGTH I-B	19.14	3.96	138.80	11.72	-57.10	11.68	128.19
STRENGTH IV	0.00	0.00	138.80	11.72	-104.19	14.01	60.34
SERVICE I	10.94	2.26	102.82	7.81	-58.16	9.34	75.01

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	75.01	1.83
STRENGTH I-A	127.47	3.29
STRENGTH I-B	128.19	3.31
STRENGTH IV	60.34	1.46
SERVICE I	75.01	1.83

d= 19.33 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches

Heel Thickness = 21.96 inches 21.96 inches default thickness
Use # 10 at top
use spacing= 6.00 inches
As= 2.54 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 3.69
de=ds= 19.33 inches (for no prestressing)
c/de= 0.19 member is not overreinforced per 5.7.3.3.1-1

Mn= 149.65 ft-K/ft
steel okay

n = 10
x = 7.17 inches
fs = [redacted] ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	γq_{max}'-γq_{min}	γq_{min}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	1.75	0.27	24.93	0.95	-11.03	-3.74	2.26	13.13
STRENGTH I-A	0.00	0.48	24.93	1.42	-22.40	17.94	2.83	22.36
STRENGTH I-B	3.06	0.48	33.65	1.42	-18.17	0.75	2.83	21.19
STRENGTH IV	0.00	0.00	33.65	1.42	-11.32	-11.04	3.40	12.71
SERVICE I	1.75	0.27	24.93	0.95	-11.03	-3.74	2.26	13.13

Vn = 25.96 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 40.09 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 170.50 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}	γq_{min}	W_{Toe}
	K/ft	Ksf	Ksf	K/ft
V (UNFACTORED)	1.08	5.12	0.45	1.19
STRENGTH I-A	0.00	7.31	-2.17	1.07
STRENGTH I-B	1.46	7.60	-0.09	1.49
STRENGTH IV	1.46	6.13	1.34	1.78
SERVICE I	1.08	5.12	0.45	1.19

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-2.34	8.77	34.87	-2.57	38.72
STRENGTH I-A	0.00	17.81	41.82	-2.32	57.32
STRENGTH I-B	-3.16	14.45	49.59	-3.22	57.66
STRENGTH IV	-3.16	9.00	43.97	-3.86	45.94
SERVICE I	-2.34	8.77	34.87	-2.57	38.72

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	38.72	0.97
STRENGTH I-A	57.32	1.46
STRENGTH I-B	57.66	1.47
STRENGTH IV	45.94	1.16
SERVICE I	38.72	0.97

d= 18.33 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches

Toe Thickness = 21.96 inches — 21.96 inches default thickness
Use # 10 at bottom
use spacing= 6.00 inches
As= 2.54 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

c= 3.69
de=ds= 18.33 inches (for no prestressing)
c/de= 0.20 member is not overreinforced per 5.7.3.3.1-1

Mn= 149.65 ft-K/ft
steel okay

n = 10
x = 7.17 inches
fs = 10.80 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-1.08	3.04	16.11	-1.19	16.87
STRENGTH I-A	0.00	6.17	19.32	-1.07	24.42
STRENGTH I-B	-3.16	5.00	22.91	-1.49	23.26
STRENGTH IV	-3.16	3.12	20.31	-1.78	18.48
SERVICE I	-2.34	3.04	16.11	-1.19	15.61

Vn = 25.96 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

$1.2^*M_{cracking}$= 40.09 ft-K <--- Test 1
$1.33M_{hTOT}$ (max.)= 76.68 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	6.29
STRENGTH I-A	8.50
STRENGTH I-B	8.50
STRENGTH IV	8.50
SERVICE I	6.29

$AS_{req'd}$
in^2
0.13
0.17
0.17
0.17
0.13

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:

Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 2+20

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 5.00 degrees (11.4:1 slope)

δ= 31.00 degrees — friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.99
K_a= 0.298 — 0.286 — 0.286

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 1.00 ft
Distance to Dead Load Surcharge (a)= 5.30 ft
Live Load - q (LS)= 2.00 ft
Distance to Live Load Surcharge (b)= 7.30 ft
Distance to broken backslope (c)= 0.00 ft (To create Broken Backslope, Input # > 0)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 25.75 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.83 ft
L_{toe}= 4.33 ft — γ_{conc}= 0.150 kcf
H_{toe}= 1.83 ft
L_{heel}= 8.25 ft
H_{heel}= 1.83 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft — Assume Depth
K_{dist}= 8.56 ft — over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) — Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf — (from Geotech Report)
$\Phi_{bearing}$= 0.5 — (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) — (from Geotech Report)
$\Phi_{sliding}$= 0.8 — (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf — (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient) (from Geotech Report)
$FS_{sliding}$=	1.5	
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 14.42 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	36.83	26.06	-80.34	236.30	4.23	3.60	[redacted]
STRENGTH I-B	51.95	26.06	-102.28	236.30	2.58	3.60	OKAY
STRENGTH IV	53.64	4.25	-90.56	170.62	1.49	3.60	OKAY
SERVICE I	38.95	15.30	-72.51	153.80	2.09	3.60	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.30

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	36.83	4.23	6.19	12.00	OKAY
STRENGTH I-B	51.95	2.58	5.61	12.00	OKAY
STRENGTH IV	53.64	1.49	4.69	12.00	OKAY
SERVICE I	38.95	2.09	3.80	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 6.31

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	36.83	16.57	8.50	17.51	26.06	[redacted]
STRENGTH I-B	51.95	23.38	8.50	22.96	26.06	
STRENGTH IV	53.64	24.14	8.50	23.56	4.25	OKAY
SERVICE I	38.95	17.53	9.45	18.75	15.30	OKAY
ASD	37.86	17.04	9.45	18.36	15.30	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.7

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	1.38	8.27	0.00	0.00	9.65
STRENGTH I-A	2.42	12.40	0.00	0.00	14.82
STRENGTH I-B	2.42	12.40	0.00	0.00	14.82
STRENGTH IV	0.00	12.40	0.00	0.00	12.40
SERVICE I	1.38	8.27	0.00	0.00	9.65

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	16.53	65.90	0.00	0.00	82.44
STRENGTH I-A	0.00	98.86	0.00	0.00	98.86
STRENGTH I-B	28.93	98.86	0.00	0.00	127.79
STRENGTH IV	0.00	98.86	0.00	0.00	98.86
SERVICE I	16.53	65.90	0.00	0.00	82.44

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	82.44
STRENGTH I-A	98.86
STRENGTH I-B	127.79
STRENGTH IV	98.86
SERVICE I	82.44

$AS_{req'd}$
in^2
2.03
2.48
3.31
2.48
2.03

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
78.99	77.27	75.54
118.48	115.90	113.31
118.48	115.90	113.31
118.48	115.90	113.31
78.99	77.27	75.54
	steel okay 1.5' up	steel okay 2' up

d_s= 19.29 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches
Stem Thickness = 23.70 inches 22.00 inches default thickness
Use # 11 at back face
use spacing= 7.00 inches
As= 2.67 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 3.88
$d_e=d_s$= 19.29 inches (for no prestressing)
c/d_e= 0.20 member is not overreinforced per 5.7.3.3.1-1

Mn= 171.73 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 7.70 inches
fs = [redacted] ksi

Vn = 28.02 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 46.70 ft-K <— Test 1
1.33M_{hTOT} (max.)= 169.96 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.16	0.93	1.08	3.80	2.09
STRENGTH I-A	0.27	1.39	1.66	6.19	4.23
STRENGTH I-B	0.27	1.39	1.66	5.61	2.58
STRENGTH IV	0.27	1.39	1.66	4.69	1.49
SERVICE I	0.16	0.93	1.08	3.80	2.09

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	2.42	1.28	4.47	7.65	-31.58	9.36	-6.41
STRENGTH I-A	0.00	2.24	4.47	11.47	-0.15	11.70	29.73
STRENGTH I-B	4.24	2.24	6.03	11.47	-26.81	11.70	8.87
STRENGTH IV	0.00	0.00	6.03	11.47	-65.03	14.04	-33.49
SERVICE I	2.42	1.28	4.47	7.65	-31.58	9.36	-6.41

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	-6.41	-0.15
STRENGTH I-A	29.73	0.69
STRENGTH I-B	8.87	0.20
STRENGTH IV	-33.49	-0.75
SERVICE I	-6.41	-0.15

d= 19.44 inches

per 5.10.8.2 ASTemp= 0.40 sq inches

Heel Thickness = 22.00 inches — 22.00 inches default thickness
Use # 9 at top
use spacing= 7.00 inches
As= 1.71 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 2.49
de=ds= 19.44 inches (for no prestressing)
c/de= 0.13 member is not overreinforced per 5.7.3.3.1-1

Mn= 104.67 ft-K/ft
steel okay

n = 10
x = 6.16 inches
fs = -2.58 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.36	0.16	1.08	0.93	-15.50	2.27	-12.98
STRENGTH I-A	0.00	0.27	1.08	1.39	1.36	2.84	4.10
STRENGTH I-B	0.63	0.27	1.46	1.39	-17.35	2.84	-13.60
STRENGTH IV	0.00	0.00	1.46	1.39	-24.70	3.40	-21.85
SERVICE I	0.36	0.16	1.08	0.93	-15.50	2.27	-12.98

Vn = 26.01 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 40.24 ft-K <--- Test 1
1.33MhTOT (max.)= 39.54 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	1.08	16.48	1.19
STRENGTH I-A	0.00	26.84	1.07
STRENGTH I-B	1.46	24.32	1.49
STRENGTH IV	1.46	20.33	1.79
SERVICE I	1.08	16.48	1.19

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-2.35	35.70	-2.58	30.77
STRENGTH I-A	0.00	58.14	-2.32	55.82
STRENGTH I-B	-3.17	52.69	-3.23	46.30
STRENGTH IV	-3.17	44.05	-3.87	37.01
SERVICE I	-2.35	35.70	-2.58	30.77

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	30.77	0.76
STRENGTH I-A	55.82	1.42
STRENGTH I-B	46.30	1.17
STRENGTH IV	37.01	0.92
SERVICE I	30.77	0.76

d= 18.36 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches

Toe Thickness = 22.00 inches 22.00 inches default thickness
Use # 10 at bottom
use spacing= 6.00 inches
As= 2.54 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 149.99 ft-K/ft
steel okay

c= 3.69
de=ds= 18.36 inches (for no prestressing)
c/de= 0.20 member is not overreinforced
per 5.7.3.3.1-1
n = 10
x = 7.18 inches
fs = 8.57 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-1.08	16.48	-1.19	14.20
STRENGTH I-A	0.00	26.84	-1.07	25.76
STRENGTH I-B	-3.17	24.32	-1.49	19.66
STRENGTH IV	-3.17	20.33	-1.79	15.38
SERVICE I	-2.35	16.48	-1.19	12.94

Vn = 26.01 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 40.24 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 74.24 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	6.30
STRENGTH I-A	8.50
STRENGTH I-B	8.50
STRENGTH IV	8.50
SERVICE I	6.30

$AS_{req'd}$
in^2
0.13
0.17
0.17
0.17
0.13

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 2+70

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 5.00 degrees (11.4:1 slope)

δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.99
K_a= 0.298 0.286 0.286

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 2.30 ft
Distance to Dead Load Surcharge (a)= 9.00 ft
Live Load - q (LS)= 2.00 ft
Distance to Live Load Surcharge (b)= 9.00 ft
Distance to broken backslope (c)= 0.00 ft (To create Broken Backslope, Input # > 0)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 24.40 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.83 ft
L_{toe}= 4.33 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.83 ft
L_{heel}= 8.25 ft
H_{heel}= 1.83 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft Assume Depth
K_{dist}= 8.56 ft over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient)	(from Geotech Report)
$FS_{sliding}$=	1.5		
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)	
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)	

SUMMARY OF ECCENTRICITY CHECK

B= 14.42 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	34.93	25.08	-75.42	221.61	4.19	3.60	
STRENGTH I-B	48.37	25.08	-89.61	221.61	2.73	3.60	OKAY
STRENGTH IV	51.02	5.44	-84.69	146.65	1.21	3.60	OKAY
SERVICE I	36.46	14.85	-64.39	143.12	2.16	3.60	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.29

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	34.93	4.19	5.78	12.00	OKAY
STRENGTH I-B	48.37	2.73	5.40	12.00	OKAY
STRENGTH IV	51.02	1.21	4.26	12.00	OKAY
SERVICE I	36.46	2.16	3.61	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 6.65

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	34.93	15.72	8.50	16.83	25.08	
STRENGTH I-B	48.37	21.77	8.50	21.67	25.08	
STRENGTH IV	51.02	22.96	8.50	22.62	5.44	OKAY
SERVICE I	36.46	16.41	9.45	17.85	14.85	OKAY
ASD	35.38	15.92	9.45	17.46	14.85	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.7

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	1.30	7.36	0.00	0.00	8.66
STRENGTH I-A	2.28	11.04	0.00	0.00	13.32
STRENGTH I-B	2.28	11.04	0.00	0.00	13.32
STRENGTH IV	0.00	11.04	0.00	0.00	11.04
SERVICE I	1.30	7.36	0.00	0.00	8.66

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	14.72	55.36	0.00	0.00	70.08
STRENGTH I-A	0.00	83.04	0.00	0.00	83.04
STRENGTH I-B	25.76	83.04	0.00	0.00	108.80
STRENGTH IV	0.00	83.04	0.00	0.00	83.04
SERVICE I	14.72	55.36	0.00	0.00	70.08

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	70.08
STRENGTH I-A	83.04
STRENGTH I-B	108.80
STRENGTH IV	83.04
SERVICE I	70.08

$AS_{req'd}$
in^2
1.71
2.05
2.76
2.05
1.71

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
66.98	65.42	63.87
100.46	98.13	95.81
100.46	98.13	95.81
100.46	98.13	95.81
66.98	65.42	63.87
	steel okay 1.5' up	steel okay 2' up

ds= 19.29 inches

per 5.10.8.2 — AStemp= 0.40 sq inches
Stem Thickness = 23.70 inches — 22.00 inches default thickness
Use # 11 at back face
use spacing= 7.00 inches
As= 2.67 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 3.88
de=ds= 19.29 inches (for no prestressing)
c/de= 0.20 member is not overreinforced
per 5.7.3.3.1-1

Mn= 171.73 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 7.70 inches
fs = 17.06 ksi

Vn = 28.02 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 46.70 ft-K <--- Test 1
1.33MhTOT (max.)= 144.71 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.15	0.83	0.97	3.61	2.16
STRENGTH I-A	0.26	1.24	1.49	5.78	4.19
STRENGTH I-B	0.26	1.24	1.49	5.40	2.73
STRENGTH IV	0.26	1.24	1.49	4.26	1.21
SERVICE I	0.15	0.83	0.97	3.61	2.16

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	3.08	1.21	4.47	6.81	-27.90	9.36	-2.98
STRENGTH I-A	0.00	2.11	4.47	10.21	-0.04	11.70	28.45
STRENGTH I-B	5.39	2.11	6.03	10.21	-21.05	11.70	14.40
STRENGTH IV	0.00	0.00	6.03	10.21	-72.12	14.04	-41.83
SERVICE I	3.08	1.21	4.47	6.81	-27.90	9.36	-2.98

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	-2.98	-0.07
STRENGTH I-A	28.45	0.66
STRENGTH I-B	14.40	0.33
STRENGTH IV	-41.83	-0.93
SERVICE I	-2.98	-0.07

d= 19.44 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches

Heel Thickness = 22.00 inches 22.00 inches default thickness
Use # 9 at top
use spacing= 7.00 inches
As= 1.71 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 2.49
de=ds= 19.44 inches (for no prestressing)
c/de= 0.13 member is not overreinforced per 5.7.3.3.1-1

Mn= 104.67 ft-K/ft
steel okay

n = 10
x = 6.16 inches
fs = -1.20 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.36	0.15	1.08	0.83	-14.19	2.27	-11.78
STRENGTH I-A	0.00	0.26	1.08	1.24	0.70	2.84	3.28
STRENGTH I-B	0.63	0.26	1.46	1.24	-15.08	2.84	-11.49
STRENGTH IV	0.00	0.00	1.46	1.24	-24.78	3.40	-22.08
SERVICE I	0.36	0.15	1.08	0.83	-14.19	2.27	-11.78

Vn = 26.01 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

$1.2 \cdot M_{cracking}$= 40.24 ft-K <--- Test 1
$1.33 M_{hTOT}$ (max.)= 37.84 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	1.08	15.65	1.19
STRENGTH I-A	0.00	25.04	1.07
STRENGTH I-B	1.46	23.40	1.49
STRENGTH IV	1.46	18.44	1.79
SERVICE I	1.08	15.65	1.19

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
M_v (UNFACTORED)	-2.35	33.90	-2.58	28.97
STRENGTH I-A	0.00	54.25	-2.32	51.92
STRENGTH I-B	-3.17	50.69	-3.23	44.30
STRENGTH IV	-3.17	39.96	-3.87	32.92
SERVICE I	-2.35	33.90	-2.58	28.97

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
M_v (UNFACTORED)	28.97	0.72
STRENGTH I-A	51.92	1.31
STRENGTH I-B	44.30	1.11
STRENGTH IV	32.92	0.82
SERVICE I	28.97	0.72

d= 18.36 inches

per 5.10.8.2 AS_{temp}= 0.40 sq inches

Toe Thickness = 22.00 inches 22.00 inches default thickness
Use # 10 at bottom
use spacing= 6.00 inches
As= 2.54 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 149.99 ft-K/ft
steel okay

c= 3.69
de=ds= 18.36 inches (for no prestressing)
c/de= 0.20 member is not overreinforced
per 5.7.3.3.1-1
n = 10
x = 7.18 inches
fs = 8.07 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
M_v (UNFACTORED)	-1.08	15.65	-1.19	13.37
STRENGTH I-A	0.00	25.04	-1.07	23.96
STRENGTH I-B	-3.17	23.40	-1.49	18.74
STRENGTH IV	-3.17	18.44	-1.79	13.49
SERVICE I	-2.35	15.65	-1.19	12.11

Vn = 26.01 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2
1.2*$M_{cracking}$= 40.24 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 69.06 ft-K <--- Test 2
Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	6.30
STRENGTH I-A	8.50
STRENGTH I-B	8.50
STRENGTH IV	8.50
SERVICE I	6.30

$AS_{req'd}$
in^2
0.13
0.17
0.17
0.17
0.13

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/de= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 2+80

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 5.00 degrees (11.4:1 slope)

δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.99
K_a= 0.298 0.286 0.286

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 0.00 ft
Distance to Dead Load Surcharge (a)= 14.00 ft
Live Load - q (LS)= 2.00 ft
Distance to Live Load Surcharge (b)= 15.50 ft
Distance to broken backslope (c)= 0.00 ft (To create Broken Backslope, Input # > 0)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 24.40 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.50 ft
L_{toe}= 4.42 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.67 ft
L_{heel}= 7.92 ft
H_{heel}= 1.67 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft Assume Depth
K_{dist}= 7.89 ft over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient) (from Geotech Report)
$FS_{sliding}$=	1.5	
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 13.84 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	32.67	20.66	-72.41	151.66	2.43	3.46	OKAY
STRENGTH I-B	45.30	20.66	-86.20	151.66	1.45	3.46	OKAY
STRENGTH IV	47.55	1.06	-82.14	145.50	1.33	3.46	OKAY
SERVICE I	34.07	11.91	-62.14	102.75	1.19	3.46	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.90

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	32.67	2.43	3.63	12.00	OKAY
STRENGTH I-B	45.30	1.45	4.14	12.00	OKAY
STRENGTH IV	47.55	1.33	4.25	12.00	OKAY
SERVICE I	34.07	1.19	2.97	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 8.07

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	32.67	14.70	7.51	15.51	20.66	
STRENGTH I-B	45.30	20.39	7.51	20.06	20.66	
STRENGTH IV	47.55	21.40	7.51	20.87	1.06	OKAY
SERVICE I	34.07	15.33	8.34	16.43	11.91	OKAY
ASD	32.96	14.83	8.34	16.04	11.91	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.9

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	1.31	7.47	0.00	0.00	8.78
STRENGTH I-A	2.30	11.20	0.00	0.00	13.50
STRENGTH I-B	2.30	11.20	0.00	0.00	13.50
STRENGTH IV	0.00	11.20	0.00	0.00	11.20
SERVICE I	1.31	7.47	0.00	0.00	8.78

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	14.94	56.60	0.00	0.00	71.53
STRENGTH I-A	0.00	84.90	0.00	0.00	84.90
STRENGTH I-B	26.14	84.90	0.00	0.00	111.04
STRENGTH IV	0.00	84.90	0.00	0.00	84.90
SERVICE I	14.94	56.60	0.00	0.00	71.53

LOAD TYPE	M_{hTOT}	$AS_{req'd}$	M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
	ft-K/ft	in^2	ft-K/ft	ft-K/ft	ft-K/ft
Mh (UNFACTORED)	71.53	2.30	68.39	66.81	65.24
STRENGTH I-A	84.90	2.79	102.58	100.22	97.86
STRENGTH I-B	111.04	3.85	102.58	100.22	97.86
STRENGTH IV	84.90	2.79	102.58	100.22	97.86
SERVICE I	71.53	2.30	68.39	66.81	65.24
				steel okay 1.5' up	steel okay 2' up

ds= 15.30 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches
Stem Thickness = 23.70 inches 18.00 inches default thickness
Use # 11 at back face
use spacing= 7.00 inches
As= 2.67 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 3.88
de=ds= 15.30 inches (for no prestressing)
c/de= 0.25 member is not overreinforced
per 5.7.3.3.1-1

Mn= 171.73 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 7.70 inches
fs = 17.42 ksi

Vn = 28.02 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 46.70 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 147.68 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.15	0.84	0.98	2.97	1.19
STRENGTH I-A	0.26	1.26	1.51	3.63	2.43
STRENGTH I-B	0.26	1.26	1.51	4.14	1.45
STRENGTH IV	0.26	1.26	1.51	4.25	1.33
SERVICE I	0.15	0.84	0.98	2.97	1.19

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	3.61	1.17	4.38	6.63	-45.57	7.84	-21.94
STRENGTH I-A	0.00	2.04	4.38	9.95	-17.11	9.80	9.06
STRENGTH I-B	6.32	2.04	5.91	9.95	-52.33	9.80	-18.31
STRENGTH IV	0.00	0.00	5.91	9.95	-58.75	11.76	-31.13
SERVICE I	3.61	1.17	4.38	6.63	-45.57	7.84	-21.94

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	-21.94	-0.55
STRENGTH I-A	9.06	0.23
STRENGTH I-B	-18.31	-0.46
STRENGTH IV	-31.13	-0.77
SERVICE I	-21.94	-0.55

d= 17.44 inches

per 5.10.8.2 AS_{temp}= 0.36 sq inches

Heel Thickness = 20.00 inches 20.00 inches default thickness
Use # 9 at top
use spacing= 7.00 inches
As= 1.71 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

Mn= 93.24 ft-K/ft
steel okay

c= 2.49
de=ds= 17.44 inches (for no prestressing)
c/de= 0.14 member is not overreinforced per 5.7.3.3.1-1

n = 10
x = 5.77 inches
fs = -9.90 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.33	0.15	1.11	0.84	-16.46	1.98	-14.04
STRENGTH I-A	0.00	0.26	1.11	1.26	-11.15	2.48	-8.53
STRENGTH I-B	0.58	0.26	1.49	1.26	-20.81	2.48	-17.23
STRENGTH IV	0.00	0.00	1.49	1.26	-22.36	2.97	-19.61
SERVICE I	0.33	0.15	1.11	0.84	-16.46	1.98	-14.04

Vn = 23.65 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 33.26 ft-K <— Test 1
1.33M_{hTOT} (max.)= 12.05 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	1.11	13.14	1.11
STRENGTH I-A	0.00	16.07	0.99
STRENGTH I-B	1.49	18.29	1.38
STRENGTH IV	1.49	18.81	1.66
SERVICE I	1.11	13.14	1.11

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
M_v (UNFACTORED)	-2.44	29.05	-2.44	24.16
STRENGTH I-A	0.00	35.51	-2.20	33.31
STRENGTH I-B	-3.30	40.41	-3.05	34.06
STRENGTH IV	-3.30	41.56	-3.66	34.60
SERVICE I	-2.44	29.05	-2.44	24.16

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
M_v (UNFACTORED)	24.16	0.67
STRENGTH I-A	33.31	0.94
STRENGTH I-B	34.06	0.96
STRENGTH IV	34.60	0.98
SERVICE I	24.16	0.67

d= 16.37 inches

per 5.10.8.2 AS_{temp}= 0.36 sq inches

Toe Thickness = 20.00 inches — 20.00 inches default thickness
Use # 10 at bottom
use spacing= 6.00 inches
As= 2.54 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

c= 3.69
d_e=d_s= 16.37 inches (for no prestressing)
c/d_e= 0.23 member is not overreinforced per 5.7.3.3.1-1

Mn= 133.05 ft-K/ft
steel okay

n = 10
x = 6.71 inches
fs = 7.55 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
M_v (UNFACTORED)	-1.11	13.14	-1.11	10.93
STRENGTH I-A	0.00	16.07	-0.99	15.07
STRENGTH I-B	-3.30	18.29	-1.38	13.61
STRENGTH IV	-3.30	18.81	-1.66	13.85
SERVICE I	-2.44	13.14	-1.11	9.60

Vn = 23.64 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 33.26 ft-K <— Test 1
1.33M_{hTOT} (max.)= 46.02 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
Mh (UNFACTORED)	5.56
STRENGTH I-A	7.51
STRENGTH I-B	7.51
STRENGTH IV	7.51
SERVICE I	5.56

$AS_{req'd}$
in^2
0.11
0.15
0.15
0.15
0.11

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:

Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 3+00

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 5.00 degrees (11.4:1 slope)

δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.99
K_a= 0.298 0.286 0.286

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 0.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Dead Load Surcharge (a)= 20.00 ft
Live Load - q (LS)= 0.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Live Load Surcharge (b)= 20.00 ft
Distance to broken backslope (c)= 0.00 ft (To create Broken Backslope, Input # > 0)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 20.30 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.44 ft
L_{toe}= 3.83 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.50 ft
L_{heel}= 6.20 ft
H_{heel}= 1.50 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft Assume Depth
K_{dist}= 6.67 ft over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient) (from Geotech Report)
$FS_{sliding}$=	1.5	
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 11.47 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	22.13	13.52	-41.05	84.54	1.96	2.87	OKAY
STRENGTH I-B	31.06	13.52	-48.29	84.54	1.17	2.87	OKAY
STRENGTH IV	32.82	0.00	-47.21	84.54	1.14	2.87	OKAY
SERVICE I	23.46	7.73	-35.02	58.15	0.99	2.87	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.92

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	22.13	1.96	2.93	12.00	OKAY
STRENGTH I-B	31.06	1.17	3.40	12.00	OKAY
STRENGTH IV	32.82	1.14	3.57	12.00	OKAY
SERVICE I	23.46	0.99	2.47	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 9.72

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	22.13	9.96	6.04	10.99	13.52	[redacted]
STRENGTH I-B	31.06	13.98	6.04	14.20	13.52	OKAY
STRENGTH IV	32.82	14.77	6.04	14.83	0.00	OKAY
SERVICE I	23.46	10.56	6.71	11.80	7.73	OKAY
ASD	22.50	10.12	6.71	11.45	7.73	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 2.2

REINFORCING DESIGN

GIVEN:
fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	0.00	5.11	0.00	0.00	5.11
STRENGTH I-A	0.00	7.66	0.00	0.00	7.66
STRENGTH I-B	0.00	7.66	0.00	0.00	7.66
STRENGTH IV	0.00	7.66	0.00	0.00	7.66
SERVICE I	0.00	5.11	0.00	0.00	5.11

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	0.00	32.01	0.00	0.00	32.01
STRENGTH I-A	0.00	48.01	0.00	0.00	48.01
STRENGTH I-B	0.00	48.01	0.00	0.00	48.01
STRENGTH IV	0.00	48.01	0.00	0.00	48.01
SERVICE I	0.00	32.01	0.00	0.00	32.01

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	32.01
STRENGTH I-A	48.01
STRENGTH I-B	48.01
STRENGTH IV	48.01
SERVICE I	32.01

$AS_{req'd}$
in^2
1.02
1.57
1.57
1.57
1.02

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
30.31	29.46	28.60
45.46	44.18	42.91
45.46	44.18	42.91
45.46	44.18	42.91
30.31	29.46	28.60
	steel okay 1.5' up	steel okay 2' up

ds= 14.58 inches

per 5.10.8.2 AStemp= 0.31 sq inches
Stem Thickness = 22.50 inches 17.28 inches default thickness
Use # 11 at back face
use spacing= 8.50 inches
As= 2.20 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 3.20
de=ds= 14.58 inches (for no prestressing)
c/de= 0.22 member is not overreinforced
per 5.7.3.3.1-1

Mn= 134.88 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 6.88 inches
fs = 9.97 ksi

Vn = 26.60 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 42.09 ft-K <--- Test 1
1.33MhTOT (max.)= 63.86 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.00	0.57	0.57	2.47	0.99
STRENGTH I-A	0.00	0.86	0.86	2.93	1.96
STRENGTH I-B	0.00	0.86	0.86	3.40	1.17
STRENGTH IV	0.00	0.86	0.86	3.57	1.14
SERVICE I	0.00	0.57	0.57	2.47	0.99

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	2.64	0.00	2.97	3.55	-22.07	4.32	-8.58
STRENGTH I-A	0.00	0.00	2.97	5.33	-7.56	5.41	6.14
STRENGTH I-B	4.63	0.00	4.01	5.33	-25.40	5.41	-6.04
STRENGTH IV	0.00	0.00	4.01	5.33	-27.49	6.49	-11.67
SERVICE I	2.64	0.00	2.97	3.55	-22.07	4.32	-8.58

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	-8.58	-0.24
STRENGTH I-A	6.14	0.18
STRENGTH I-B	-6.04	-0.17
STRENGTH IV	-11.67	-0.33
SERVICE I	-8.58	-0.24

d= 15.44 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches

Heel Thickness = 18.00 inches — 18.00 inches default thickness
Use # 9 at top
use spacing= 8.50 inches
As= 1.41 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

Mn= 68.30 ft-K/ft
steel okay

c= 2.05
de=ds= 15.44 inches (for no prestressing)
c/de= 0.13 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.96 inches
fs = -5.29 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.20	0.00	0.96	0.57	-10.44	1.40	-8.71
STRENGTH I-A	0.00	0.00	0.96	0.86	-6.66	1.74	-4.84
STRENGTH I-B	0.35	0.00	1.29	0.86	-13.14	1.74	-10.63
STRENGTH IV	0.00	0.00	1.29	0.86	-14.01	2.09	-11.85
SERVICE I	0.20	0.00	0.96	0.57	-10.44	1.40	-8.71

Vn = 21.28 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 26.94 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 8.17 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

TOE REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.96	9.46	0.86
STRENGTH I-A	0.00	11.25	0.78
STRENGTH I-B	1.29	13.03	1.08
STRENGTH IV	1.29	13.68	1.29
SERVICE I	0.96	9.46	0.86

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-1.84	18.14	-1.65	14.65
STRENGTH I-A	0.00	21.55	-1.49	20.07
STRENGTH I-B	-2.48	24.97	-2.07	20.42
STRENGTH IV	-2.48	26.22	-2.48	21.26
SERVICE I	-1.84	18.14	-1.65	14.65

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	14.65	0.46
STRENGTH I-A	20.07	0.63
STRENGTH I-B	20.42	0.65
STRENGTH IV	21.26	0.67
SERVICE I	14.65	0.46

d= 14.44 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches

Toe Thickness = 18.00 inches — 18.00 inches default thickness
Use # 9 at bottom
use spacing= 8.50 inches
As= 1.41 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 68.30 ft-K/ft
steel okay

c= 2.05
de=ds= 14.44 inches (for no prestressing)
c/de= 0.14 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.96 inches
fs = 9.03 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-0.96	9.46	-0.86	7.64
STRENGTH I-A	0.00	11.25	-0.78	10.47
STRENGTH I-B	-2.48	13.03	-1.08	9.47
STRENGTH IV	-2.48	13.68	-1.29	9.91
SERVICE I	-1.84	9.46	-0.86	6.76

Vn = 21.28 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 26.94 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 28.27 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
Mh (UNFACTORED)	4.47
STRENGTH I-A	6.04
STRENGTH I-B	6.04
STRENGTH IV	6.04
SERVICE I	4.47

$AS_{req'd}$
in^2
0.09
0.12
0.12
0.12
0.09

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
de=ds= 21.75 inches (for no prestressing)
c/de= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 3+10

Given:

InSitu Material: γ= 120 pcf

Backfill: γ= 125 pcf

Φ= 34.00 degrees — angle of internal friction

δ= 31.00 degrees — friction angle between fill & wall

slope of backfill (β) = 17 degrees

angle of wall to horizontal (Θ) = 87 degrees

Γ = 2.48

Ka= 0.359

Ko= 0.44

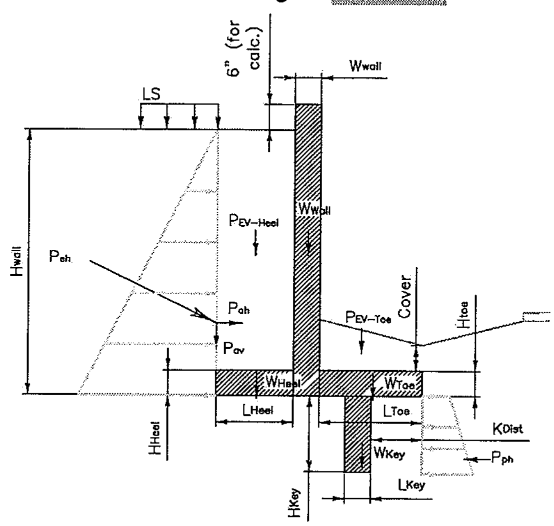
Wall Dimensions

Hwall=	19.00	ft
Wwall=	1.41	ft
Ltoe=	3.33	ft
Htoe=	1.33	ft
Lheel=	5.31	ft
Hheel=	1.33	ft
Lkey=	2.00	ft
Hkey=	1.00	ft
Kdist=	5.72	ft

γ_{conc}= 150 pcf

Surcharge LS= 2.00 ft

Dist to surcharge= 24.00 ft



Assume Depth over Toe= 2 ft

LRFD DESIGN:

q_{ult}=	24.00	Ksf		(from Geotech Report)
$\Phi_{bearing}$=	0.5			(from Geotech Report)
C=	unknown	Ksf	(cohesive soil strength)	
f=	0.45		(interface friction)	(from Geotech Report)
$\Phi_{sliding}$=	0.8			(from Table 10.5.5-1)

ASD DESIGN:

Qallow=	6.00	Ksf		(from As-Built Dwgs)
f=	0.45		(interface friction)	(from Geotech Report)
$FS_{sliding}$=	1.5			
$FS_{bearing}$=	3			

SUMMARY OF ECCENTRICITY CHECK (NHI)

B= 10.053 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	X_o	e_b	e_{max}		e_{LRFD}
	K/ft	K/ft	K/ft	K/ft	ft	ft	ft		ft
STRENGTH I-A	21.69	14.46	156.07	103.36	2.43	2.60	2.51	[redacted]	2.14
STRENGTH I-B	29.07	14.46	197.98	103.36	3.26	1.77	2.51	OKAY	1.34
STRENGTH IV	30.61	11.61	196.13	76.27	3.92	1.11	2.51	OKAY	0.35
SERVICE I	21.58	9.37	144.57	67.55	3.57	1.46	2.51	OKAY	0.95

ASD Check ===> OKAY FS$_{overturning}$= 2.14

SUMMARY OF FACTORED BEARING PRESSURES (NHI)

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	M_{vTOT}	M_{hTOT}	X_o	γq	q_r		σ_{LRFD}
	K/ft	K/ft	K/ft	K/ft	Ksf	Ksf		Ksf
STRENGTH I-A	21.69	156.07	103.36	2.43	4.46	12.00	OKAY	3.75975
STRENGTH I-B	29.07	197.98	103.36	3.26	4.46	12.00	OKAY	3.9408
STRENGTH IV	30.61	196.13	76.27	3.92	3.91	12.00	OKAY	3.27133
SERVICE I	21.58	144.57	67.55	3.57	3.02	12.00	OKAY	2.64684

ASD Check ===> OKAY FS$_{bearing}$= 7.94

SUMMARY OF SLIDING RESISTANCE (NHI)

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
Φ_T= 0.8 (from Table 10.5.5-1) δfriction= 24.00 degrees
Φ_{EP}= 0.5 (from Table 10.5.5-1) tanδ= 0.45

LOAD CASE	V_{TOT}	γq_{max}	γq_{min}	$\gamma q_{max}/2$	Q_T	Q_{EP}	$\Phi_T Q_T + \Phi_{EP} Q_{EP}$	H_{TOT}	
	K/ft	Ksf	Ksf	Ksf	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	21.69	5.50	-1.18	2.75	10.84	4.11	10.73	14.46	[redacted]
STRENGTH I-B	29.07	5.95	-0.17	2.97	14.53	4.11	13.68	14.46	NO GOOD
STRENGTH IV	30.61	5.06	1.03	2.53	15.30	4.11	14.30	11.61	OKAY
SERVICE I	21.58	4.01	0.28	2.01	10.79	4.57	10.91	9.37	OKAY
ASD	20.74	2.06	2.06	1.03	10.37	4.57	10.58	9.37	OKAY

ASD Check ===> OKAY FS$_{sliding}$= 1.15
FS$_{sliding}$= 1.5

LRFD LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	21.69	9.76	4.11	9.86	14.46	[redacted]
STRENGTH I-B	29.07	13.08	4.11	12.52	14.46	
STRENGTH IV	30.61	13.77	4.11	13.07	11.61	OKAY
SERVICE I	21.58	9.71	4.57	10.05	9.37	OKAY

ASD	20.74	9.33	4.57	9.75	9.37	OKAY

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	H_{TOT}
	K/ft	K/ft	K/ft
H (UNFACTORED)	1.00	4.40	5.40
STRENGTH I-A	1.74	6.61	8.35
STRENGTH I-B	1.74	6.61	8.35
STRENGTH IV	0.00	6.61	6.61
SERVICE I	1.00	4.40	5.40

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft
Mh (UNFACTORED)	8.81	25.93	34.74
STRENGTH I-A	0.00	38.90	38.90
STRENGTH I-B	15.41	38.90	54.31
STRENGTH IV	0.00	38.90	38.90
SERVICE I	8.81	25.93	34.74

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	34.74
STRENGTH I-A	38.90
STRENGTH I-B	54.31
STRENGTH IV	38.90
SERVICE I	34.74

$AS_{req'd}$
in^2
1.14
1.28
1.84
1.28
1.14

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
32.78	31.79	30.81
49.16	47.69	46.21
49.16	47.69	46.21
49.16	47.69	46.21
32.78	31.79	30.81
	steel okay 1.5' up	steel okay 2' up

ds= 14.29 inches

per 5.10.8.2 AS_{temp}= 0.30 sq inches
Stem Thickness = 21.80 inches 16.92 inches default thickness
Use # 10 at back face
use spacing= 8.00 inches
As= 1.91 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 2.77
de=ds= 14.29 inches (for no prestressing)
c/de= 0.19 member is not overreinforced per 5.7.3.3.1-1

Mn= 82.87 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 6.37 inches
fs = 12.84 ksi

Vn = 25.77 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 39.51 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 72.24 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.46	2.05	2.51
STRENGTH I-A	0.81	3.07	3.88
STRENGTH I-B	0.81	3.07	3.88
STRENGTH IV	0.81	3.07	3.88
SERVICE I	0.46	2.05	2.51

σ_{LRFD}	e_{LRFD}
Ksf	ft
2.65	0.95
3.76	2.14
3.94	1.34
3.27	0.35
2.65	0.95

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.00	2.46	31.13	10.86	-15.38	2.82	31.90
STRENGTH I-A	0.00	4.30	31.13	16.30	-1.97	3.52	53.28
STRENGTH I-B	0.00	4.30	42.03	16.30	-13.66	3.52	52.49
STRENGTH IV	0.00	0.00	42.03	16.30	-34.80	4.23	27.75
SERVICE I	0.00	2.46	31.13	10.86	-15.38	2.82	31.90

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mv (UNFACTORED)	31.90
STRENGTH I-A	53.28
STRENGTH I-B	52.49
STRENGTH IV	27.75
SERVICE I	31.90

$AS_{req'd}$
in^2
1.11
1.94
1.91
0.96
1.11

d= 13.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Heel Thickness = 16.00 inches 16.00 inches default thickness
Use # 9 at top
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 2.18
de=ds= 13.44 inches (for no prestressing)
c/de= 0.16 member is not overreinforced per 5.7.3.3.1-1

Mn= 62.29 ft-K/ft
steel okay

n = 10
x = 4.68 inches
fs = [redacted] ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	$\gamma q_{max}\text{-}\gamma q_{min}$	γq_{min}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.00	0.46	11.73	2.05	-5.23	-1.49	1.06	7.51
STRENGTH I-A	0.00	0.81	11.73	3.07	-9.37	6.29	1.33	12.52
STRENGTH I-B	0.00	0.81	15.83	3.07	-8.57	0.88	1.33	12.01
STRENGTH IV	0.00	0.00	15.83	3.07	-5.66	-5.45	1.59	7.79
SERVICE I	0.00	0.46	11.73	2.05	-5.23	-1.49	1.06	7.51

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 70.87 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}	γq_{min}	W_{Toe}
	K/ft	Ksf	Ksf	K/ft
V (UNFACTORED)	0.83	4.01	0.28	0.67
STRENGTH I-A	0.00	5.50	-1.18	0.60
STRENGTH I-B	1.12	5.95	-0.17	0.83
STRENGTH IV	1.12	5.06	1.03	1.00
SERVICE I	0.83	4.01	0.28	0.67

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
M_v (UNFACTORED)	-1.39	4.58	15.41	-1.11	17.50
STRENGTH I-A	0.00	8.21	18.24	-1.00	25.44
STRENGTH I-B	-1.87	7.51	21.78	-1.39	26.02
STRENGTH IV	-1.87	4.95	20.69	-1.67	22.10
SERVICE I	-1.39	4.58	15.41	-1.11	17.50

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
M_v (UNFACTORED)	17.50	0.65
STRENGTH I-A	25.44	0.95
STRENGTH I-B	26.02	0.98
STRENGTH IV	22.10	0.82
SERVICE I	17.50	0.65

d= 12.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Toe Thickness = 16.00 inches — 16.00 inches default thickness
Use # 9 at bottom
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

c= 2.18
d_e=d_s= 12.44 inches (for no prestressing)
c/d_e= 0.18 member is not overreinforced per 5.7.3.3.1-1

Mn= 62.29 ft-K/ft
steel okay

n = 10
x = 4.68 inches
fs = 11.78 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	γq_{max}-$\gamma q_{min'}$	$\gamma q_{min'}$	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	K/ft	ft-K/ft
M_v (UNFACTORED)	-0.83	2.06	9.25	-0.67	9.81
STRENGTH I-A	0.00	3.69	10.94	-0.60	14.04
STRENGTH I-B	-1.87	3.38	13.07	-0.83	13.74
STRENGTH IV	-1.87	2.23	12.41	-1.00	11.77
SERVICE I	-1.39	2.06	9.25	-0.67	9.26

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 34.61 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	3.04
STRENGTH I-A	4.11
STRENGTH I-B	4.11
STRENGTH IV	4.11
SERVICE I	3.04

$AS_{req'd}$
in^2
0.06
0.08
0.08
0.08
0.06

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

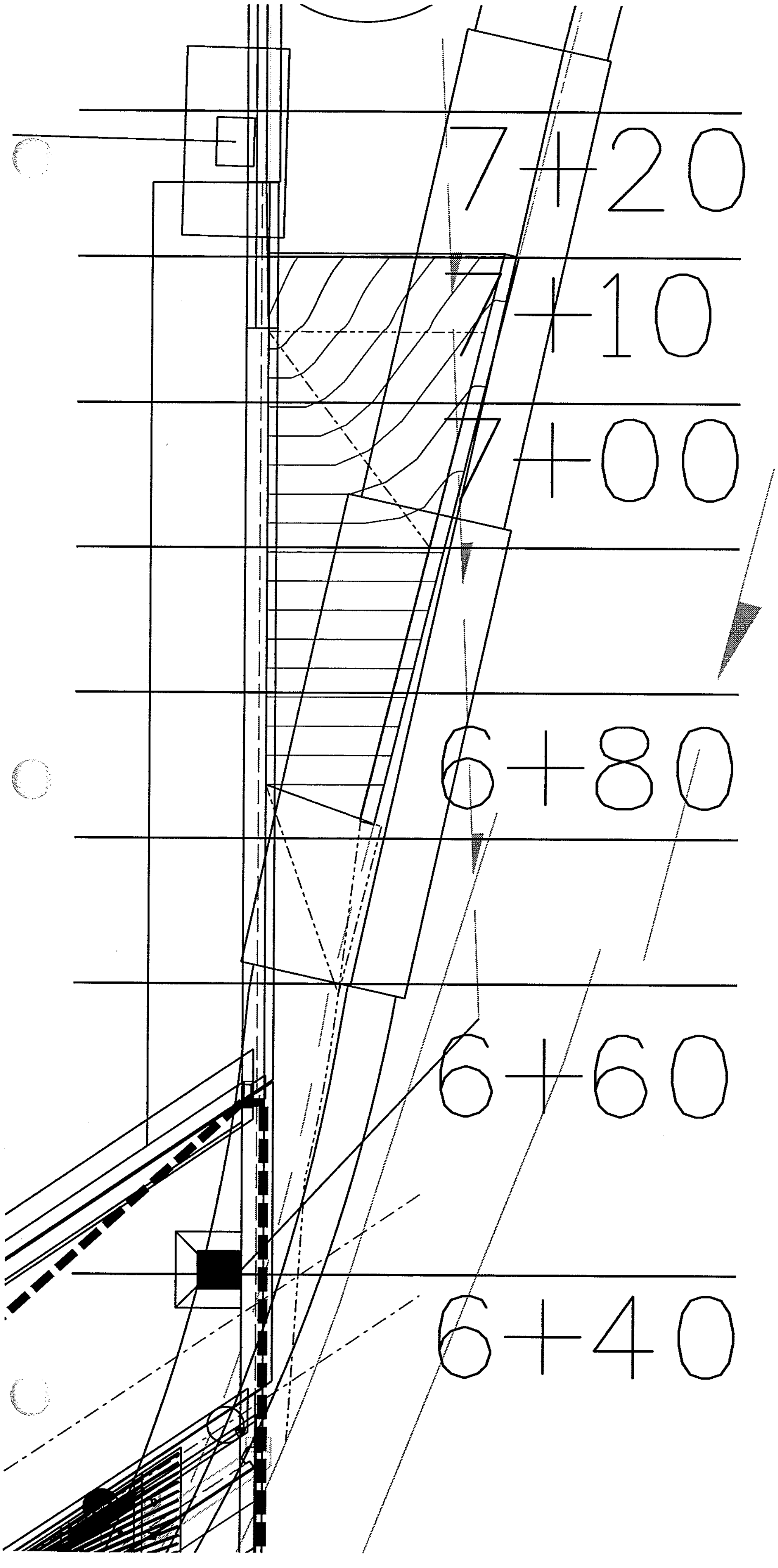
compressive steel:

Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
$d_e=d_s$= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

SouthWest Wall



Note: Potholing Information corroborates assumed wall elevations

WALL LAYOUT - 6+80

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
Ko= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 18.00 degrees (3.1:1 slope)
I= 8.10 degrees
δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.43
Ka= 0.366 $(\delta=\beta)$ 0.325 0.290

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 14.10 ft
Distance to Dead Load Surcharge (a)= 9.20 ft
Live Load - q (LS)= 0.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Live Load Surcharge (b)= 13.00 ft
Distance to broken backslope (c)= 9.20 ft (I=8.1)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 10.50 ft
batter angle (θ)= 90.00 degrees (vertical)
Wwall= 1.43 ft
Ltoe= 3.83 ft γ_{conc}= 0.150 kcf
Htoe= 1.50 ft
Lheel= 6.20 ft
Hheel= 1.50 ft
Lkey= 2.00 ft
Hkey= 1.00 ft
Kdist= 6.67 ft

Assume Depth over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

qult= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 8.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient) (from Geotech Report)
$FS_{sliding}$=	1.5	
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 11.46 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	12.88	12.01	-23.62	47.56	1.86	2.87	OKAY
STRENGTH I-B	18.59	12.01	-25.43	47.56	1.19	2.87	OKAY
STRENGTH IV	19.82	6.05	-24.97	24.57	-0.02	2.87	OKAY
SERVICE I	14.08	7.44	-18.39	30.97	0.89	2.87	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 3.20

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	12.88	1.86	1.66	12.00	OKAY
STRENGTH I-B	18.59	1.19	2.05	12.00	OKAY
STRENGTH IV	19.82	-0.02	1.72	12.00	OKAY
SERVICE I	14.08	0.89	1.46	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 16.49

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	12.88	5.79	4.91	7.09	12.01	[redacted]
STRENGTH I-B	18.59	8.37	4.91	9.15	12.01	[redacted]
STRENGTH IV	19.82	8.92	4.91	9.59	6.05	OKAY
SERVICE I	14.08	6.34	5.45	7.80	7.44	OKAY
ASD	13.13	5.91	5.45	7.45	7.44	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.5

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	0.00	1.14	0.00	0.00	1.14
STRENGTH I-A	0.00	1.71	0.00	0.00	1.71
STRENGTH I-B	0.00	1.71	0.00	0.00	1.71
STRENGTH IV	0.00	1.71	0.00	0.00	1.71
SERVICE I	0.00	1.14	0.00	0.00	1.14

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	0.00	3.42	0.00	0.00	3.42
STRENGTH I-A	0.00	5.13	0.00	0.00	5.13
STRENGTH I-B	0.00	5.13	0.00	0.00	5.13
STRENGTH IV	0.00	5.13	0.00	0.00	5.13
SERVICE I	0.00	3.42	0.00	0.00	3.42

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	3.42
STRENGTH I-A	5.13
STRENGTH I-B	5.13
STRENGTH IV	5.13
SERVICE I	3.42

$AS_{req'd}$
in^2
0.11
0.16
0.16
0.16
0.11

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
3.04	2.85	2.66
4.56	4.28	3.99
4.56	4.28	3.99
4.56	4.28	3.99
3.04	2.85	2.66
	steel okay 1.5' up	steel okay 2' up

d_s= 14.46 inches

per 5.10.8.2 AS_{temp}= 0.31 sq inches
Stem Thickness = 22.50 inches 17.16 inches default thickness
Use # 11 at back face
use spacing= 8.50 inches
As= 2.20 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 3.20
de=ds= 14.46 inches (for no prestressing)
c/de= 0.22 member is not overreinforced
per 5.7.3.3.1-1

Mn= 134.88 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 6.88 inches
fs = 1.07 ksi

Vn = 26.60 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 42.09 ft-K <— Test 1
1.33MhTOT (max.)= 6.82 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.00	0.57	0.57
STRENGTH I-A	0.00	0.86	0.86
STRENGTH I-B	0.00	0.86	0.86
STRENGTH IV	0.00	0.86	0.86
SERVICE I	0.00	0.57	0.57

σ_{LRFD}	e_{LRFD}
Ksf	ft
1.46	0.89
1.66	1.86
2.05	1.19
1.72	-0.02
1.46	0.89

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	5.77	0.00	2.97	3.55	-14.18	4.32	2.44
STRENGTH I-A	0.00	0.00	2.97	5.32	-5.12	5.41	8.58
STRENGTH I-B	10.10	0.00	4.01	5.32	-14.94	5.41	9.90
STRENGTH IV	0.00	0.00	4.01	5.32	-33.56	6.49	-17.74
SERVICE I	5.77	0.00	2.97	3.55	-14.18	4.32	2.44

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mv (UNFACTORED)	2.44
STRENGTH I-A	8.58
STRENGTH I-B	9.90
STRENGTH IV	-17.74
SERVICE I	2.44

$AS_{req'd}$
in^2
0.07
0.25
0.29
-0.50
0.07

d= 15.44 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches

Heel Thickness = 18.00 inches — 18.00 inches default thickness
Use # 9 at top
use spacing= 8.50 inches
As= 1.41 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

Mn= 68.30 ft-K/ft
steel okay

c= 2.05
de=ds= 15.44 inches (for no prestressing)
c/de= 0.13 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.96 inches
fs = 1.50 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.75	0.00	0.96	0.57	-6.42	1.40	-4.14
STRENGTH I-A	0.00	0.00	0.96	0.86	-4.13	1.74	-2.31
STRENGTH I-B	1.31	0.00	1.29	0.86	-7.82	1.74	-4.36
STRENGTH IV	0.00	0.00	1.29	0.86	-10.75	2.09	-8.60
SERVICE I	0.75	0.00	0.96	0.57	-6.42	1.40	-4.14

Vn = 21.28 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 26.94 ft-K <— Test 1
1.33M_{hTOT} (max.)= 13.17 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.96	5.58	0.86
STRENGTH I-A	0.00	6.37	0.78
STRENGTH I-B	1.29	7.85	1.08
STRENGTH IV	1.29	6.61	1.29
SERVICE I	0.96	5.58	0.86

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-1.84	10.69	-1.65	7.20
STRENGTH I-A	0.00	12.21	-1.49	10.73
STRENGTH I-B	-2.48	15.04	-2.07	10.49
STRENGTH IV	-2.48	12.66	-2.48	7.70
SERVICE I	-1.84	10.69	-1.65	7.20

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	7.20	0.22
STRENGTH I-A	10.73	0.33
STRENGTH I-B	10.49	0.33
STRENGTH IV	7.70	0.24
SERVICE I	7.20	0.22

d= 14.44 inches

per 5.10.8.2 AS_{temp}= 0.32 sq inches

Toe Thickness = 18.00 inches 18.00 inches default thickness
Use # 9 at bottom
use spacing= 8.50 inches
As= 1.41 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 68.30 ft-K/ft
steel okay

c= 2.05
de=ds= 14.44 inches (for no prestressing)
c/de= 0.14 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.96 inches
fs = 4.44 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-0.96	5.58	-0.86	3.76
STRENGTH I-A	0.00	6.37	-0.78	5.60
STRENGTH I-B	-2.48	7.85	-1.08	4.29
STRENGTH IV	-2.48	6.61	-1.29	2.83
SERVICE I	-1.84	5.58	-0.86	2.88

Vn = 21.28 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2
1.2*$M_{cracking}$= 26.94 ft-K <— Test 1
1.33M_{hTOT} (max.)= 14.27 ft-K <— Test 2
Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
Mh (UNFACTORED)	3.64
STRENGTH I-A	4.91
STRENGTH I-B	4.91
STRENGTH IV	4.91
SERVICE I	3.64

$AS_{req'd}$
in^2
0.07
0.10
0.10
0.10
0.07

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/de= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 6+90

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 18.00 degrees (3.1:1 slope)
I= 7.10 degrees
δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.43
K_a= 0.366 ($\delta=\beta$) 0.325 0.288

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 8.10 ft
Distance to Dead Load Surcharge (a)= 11.50 ft
Live Load - q (LS)= 2.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Live Load Surcharge (b)= 15.00 ft
Distance to broken backslope (c)= 11.50 ft (I=7.1)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 15.00 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.41 ft
L_{toe}= 3.33 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.33 ft
L_{heel}= 5.31 ft
H_{heel}= 1.33 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft Assume Depth
K_{dist}= 5.72 ft over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 8.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient) (from Geotech Report)
$FS_{sliding}$=	1.5	
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 10.05 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	15.83	12.98	-28.31	59.52	1.97	2.51	OKAY
STRENGTH I-B	22.18	12.98	-31.83	59.52	1.25	2.51	OKAY
STRENGTH IV	23.51	2.32	-29.72	53.64	1.02	2.51	OKAY
SERVICE I	16.69	7.64	-22.72	40.31	1.05	2.51	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.64

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	15.83	1.97	2.59	12.00	OKAY
STRENGTH I-B	22.18	1.25	2.94	12.00	OKAY
STRENGTH IV	23.51	1.02	2.93	12.00	OKAY
SERVICE I	16.69	1.05	2.10	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 11.42

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	15.83	7.12	4.03	7.71	12.98	[redacted]
STRENGTH I-B	22.18	9.98	4.03	10.00	12.98	[redacted]
STRENGTH IV	23.51	10.58	4.03	10.48	2.32	OKAY
SERVICE I	16.69	7.51	4.48	8.25	7.64	OKAY
ASD	15.86	7.14	4.48	7.95	7.64	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.5

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	0.77	2.63	0.00	0.00	3.40
STRENGTH I-A	1.35	3.94	0.00	0.00	5.29
STRENGTH I-B	1.35	3.94	0.00	0.00	5.29
STRENGTH IV	0.00	3.94	0.00	0.00	3.94
SERVICE I	0.77	2.63	0.00	0.00	3.40

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	5.26	11.98	0.00	0.00	17.24
STRENGTH I-A	0.00	17.97	0.00	0.00	17.97
STRENGTH I-B	9.20	17.97	0.00	0.00	27.17
STRENGTH IV	0.00	17.97	0.00	0.00	17.97
SERVICE I	5.26	11.98	0.00	0.00	17.24

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	17.24
STRENGTH I-A	17.97
STRENGTH I-B	27.17
STRENGTH IV	17.97
SERVICE I	17.24

$AS_{req'd}$
in^2
0.55
0.57
0.88
0.57
0.55

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
15.98	15.35	14.71
23.96	23.02	22.07
23.96	23.02	22.07
23.96	23.02	22.07
15.98	15.35	14.71

steel okay 1.5' up steel okay 2' up

ds= 14.29 inches

per 5.10.8.2 AS_{temp}= 0.30 sq inches
Stem Thickness = 21.80 inches 16.92 inches default thickness
Use # 10 at back face
use spacing= 8.00 inches
As= 1.91 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 2.77
de=ds= 14.29 inches (for no prestressing)
c/de= 0.19 member is not overreinforced
per 5.7.3.3.1-1

Mn= 113.85 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 6.37 inches
fs = 6.37 ksi

Vn = 25.77 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 39.51 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 36.14 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.39	1.32	1.71	2.10	1.05
STRENGTH I-A	0.68	1.98	2.66	2.59	1.97
STRENGTH I-B	0.68	1.98	2.66	2.94	1.25
STRENGTH IV	0.68	1.98	2.66	2.93	1.02
SERVICE I	0.39	1.32	1.71	2.10	1.05

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	4.62	2.05	2.21	7.01	-10.76	2.82	7.95
STRENGTH I-A	0.00	3.59	2.21	10.51	-2.42	3.52	17.42
STRENGTH I-B	8.09	3.59	2.99	10.51	-11.62	3.52	17.08
STRENGTH IV	0.00	0.00	2.99	10.51	-15.72	4.23	2.01
SERVICE I	4.62	2.05	2.21	7.01	-10.76	2.82	7.95

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	7.95	0.27
STRENGTH I-A	17.42	0.59
STRENGTH I-B	17.08	0.58
STRENGTH IV	2.01	0.07
SERVICE I	7.95	0.27

d= 13.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Heel Thickness = 16.00 inches — 16.00 inches default thickness
Use # 9 at top
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 2.18
de=ds= 13.44 inches (for no prestressing)
c/de= 0.16 member is not overreinforced per 5.7.3.3.1-1

Mn= 62.29 ft-K/ft
steel okay

n = 10
x = 4.68 inches
fs = 5.35 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSV}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.55	0.39	0.83	1.32	-6.73	1.06	-3.64
STRENGTH I-A	0.00	0.68	0.83	1.98	-3.54	1.33	-0.05
STRENGTH I-B	0.96	0.68	1.12	1.98	-8.26	1.33	-3.52
STRENGTH IV	0.00	0.00	1.12	1.98	-9.60	1.59	-6.50
SERVICE I	0.55	0.39	0.83	1.32	-6.73	1.06	-3.64

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <— Test 1
1.33M_{hTOT} (max.)= 23.16 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.83	7.00	0.67
STRENGTH I-A	0.00	8.64	0.60
STRENGTH I-B	1.12	9.78	0.83
STRENGTH IV	1.12	9.77	1.00
SERVICE I	0.83	7.00	0.67

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-1.39	11.67	-1.11	9.17
STRENGTH I-A	0.00	14.39	-1.00	13.39
STRENGTH I-B	-1.87	16.31	-1.39	13.04
STRENGTH IV	-1.87	16.29	-1.67	12.75
SERVICE I	-1.39	11.67	-1.11	9.17

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mv (UNFACTORED)	9.17
STRENGTH I-A	13.39
STRENGTH I-B	13.04
STRENGTH IV	12.75
SERVICE I	9.17

$AS_{req'd}$
in^2
0.33
0.49
0.48
0.47
0.33

d= 12.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Toe Thickness = 16.00 inches — 16.00 inches default thickness
Use # 9 at bottom
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 62.29 ft-K/ft
steel okay

c= 2.18
de=ds= 12.44 inches (for no prestressing)
c/de= 0.18 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.68 inches
fs = 6.18 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-0.83	7.00	-0.67	5.50
STRENGTH I-A	0.00	8.64	-0.60	8.04
STRENGTH I-B	-1.87	9.78	-0.83	7.08
STRENGTH IV	-1.87	9.77	-1.00	6.90
SERVICE I	-1.39	7.00	-0.67	4.95

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 17.81 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	2.98
STRENGTH I-A	4.03
STRENGTH I-B	4.03
STRENGTH IV	4.03
SERVICE I	2.98

$AS_{req'd}$
in^2
0.06
0.08
0.08
0.08
0.06

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 7+00

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 18.00 degrees (3.1:1 slope)
I= 7.58 degrees
δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.43
K_a= 0.366 $(\delta=\beta)$ 0.325 0.289

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 5.50 ft
Distance to Dead Load Surcharge (a)= 14.00 ft
Live Load - q (LS)= 0.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Live Load Surcharge (b)= 17.50 ft
Distance to broken backslope (c)= 14.00 ft (I=7.58)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 17.10 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.41 ft
L_{toe}= 3.33 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.33 ft
L_{heel}= 5.31 ft
H_{heel}= 1.33 ft
L_{key}= 2.00 ft
H_{key}= 1.00 ft
K_{dist}= 5.72 ft Assume Depth over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f=	0.450	(interface friction coefficient)	(from Geotech Report)
$FS_{sliding}$=	1.5		
$FS_{bearing}$=	2.5	(2.0 allowable if verified by Geotech Report)	
$FS_{overturning}$=	2	(2.0 min. footings on soil, 1.5 min. footings on rock)	

SUMMARY OF ECCENTRICITY CHECK

B= 10.05 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	17.73	14.02	-31.74	75.71	2.48	2.51	OKAY
STRENGTH I-B	24.72	14.02	-36.26	75.71	1.60	2.51	OKAY
STRENGTH IV	26.16	0.52	-35.54	75.27	1.52	2.51	OKAY
SERVICE I	18.62	8.06	-26.07	51.63	1.37	2.51	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.32

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	17.73	2.48	3.48	12.00	OKAY
STRENGTH I-B	24.72	1.60	3.60	12.00	OKAY
STRENGTH IV	26.16	1.52	3.73	12.00	OKAY
SERVICE I	18.62	1.37	2.55	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 9.42

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	17.73	7.98	4.03	8.40	14.02	[redacted]
STRENGTH I-B	24.72	11.13	4.03	10.91	14.02	[redacted]
STRENGTH IV	26.16	11.77	4.03	11.43	0.52	OKAY
SERVICE I	18.62	8.38	4.48	8.94	8.06	OKAY
ASD	17.79	8.01	4.48	8.64	8.06	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.5

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	0.00	3.50	0.00	0.00	3.50
STRENGTH I-A	0.00	5.25	0.00	0.00	5.25
STRENGTH I-B	0.00	5.25	0.00	0.00	5.25
STRENGTH IV	0.00	5.25	0.00	0.00	5.25
SERVICE I	0.00	3.50	0.00	0.00	3.50

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	0.00	18.39	0.00	0.00	18.39
STRENGTH I-A	0.00	27.59	0.00	0.00	27.59
STRENGTH I-B	0.00	27.59	0.00	0.00	27.59
STRENGTH IV	0.00	27.59	0.00	0.00	27.59
SERVICE I	0.00	18.39	0.00	0.00	18.39

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	18.39
STRENGTH I-A	27.59
STRENGTH I-B	27.59
STRENGTH IV	27.59
SERVICE I	18.39

$AS_{req'd}$
in^2
0.59
0.89
0.89
0.89
0.59

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
17.23	16.64	16.06
25.84	24.96	24.09
25.84	24.96	24.09
25.84	24.96	24.09
17.23	16.64	16.06

steel okay 1.5' up   steel okay 2' up

d_s= 14.29 inches

per 5.10.8.2 AS_{temp}= 0.30 sq inches
Stem Thickness = 21.80 inches   16.92 inches default thickness
Use # 10 at back face
use spacing= 8.00 inches
As= 1.91 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

c= 2.77
$d_e=d_s$= 14.29 inches (for no prestressing)
c/d_e= 0.19 member is not overreinforced
per 5.7.3.3.1-1

Mn= 113.85 ft-K/ft
steel okay
steel okay 1' up

n = 10
x = 6.37 inches
fs = 6.80 ksi

Vn = 25.77 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 39.51 ft-K <— Test 1
1.33M_{hTOT} (max.)= 36.69 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔP_{LSv}	P_{av}	V_{TOT}	σ_{LRFD}	e_{LRFD}
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.00	1.76	1.76	2.55	1.37
STRENGTH I-A	0.00	2.63	2.63	3.48	2.48
STRENGTH I-B	0.00	2.63	2.63	3.60	1.60
STRENGTH IV	0.00	2.63	2.63	3.73	1.52
SERVICE I	0.00	1.76	1.76	2.55	1.37

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	5.31	0.00	2.21	9.33	-8.39	2.82	11.28
STRENGTH I-A	0.00	0.00	2.21	13.99	-0.21	3.52	19.52
STRENGTH I-B	9.29	0.00	2.99	13.99	-8.08	3.52	21.71
STRENGTH IV	0.00	0.00	2.99	13.99	-9.63	4.23	11.58
SERVICE I	5.31	0.00	2.21	9.33	-8.39	2.82	11.28

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	11.28	0.38
STRENGTH I-A	19.52	0.67
STRENGTH I-B	21.71	0.74
STRENGTH IV	11.58	0.39
SERVICE I	11.28	0.38

d= 13.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Heel Thickness = 16.00 inches 16.00 inches default thickness
Use # 9 at top
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

Mn= 62.29 ft-K/ft
steel okay

c= 2.18
de=ds= 13.44 inches (for no prestressing)
c/de= 0.16 member is not overreinforced per 5.7.3.3.1-1
n = 10
x = 4.68 inches
fs = 7.60 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	P_{LSv}	ΔP_{LSv}	$P_{EV\text{-}heel}$	P_{av}	σ_{LRFD}	W_{Heel}	V_{hTOT}
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.55	0.00	0.83	1.76	-6.54	1.06	-3.40
STRENGTH I-A	0.00	0.00	0.83	2.63	-1.22	1.33	2.25
STRENGTH I-B	0.96	0.00	1.12	2.63	-7.63	1.33	-2.91
STRENGTH IV	0.00	0.00	1.12	2.63	-8.47	1.59	-4.71
SERVICE I	0.55	0.00	0.83	1.76	-6.54	1.06	-3.40

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <— Test 1
1.33M_{hTOT} (max.)= 28.87 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.83	8.49	0.67
STRENGTH I-A	0.00	11.60	0.60
STRENGTH I-B	1.12	12.01	0.83
STRENGTH IV	1.12	12.43	1.00
SERVICE I	0.83	8.49	0.67

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-1.39	14.16	-1.11	11.66
STRENGTH I-A	0.00	19.34	-1.00	18.34
STRENGTH I-B	-1.87	20.02	-1.39	16.75
STRENGTH IV	-1.87	20.72	-1.67	17.17
SERVICE I	-1.39	14.16	-1.11	11.66

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	11.66	0.43
STRENGTH I-A	18.34	0.68
STRENGTH I-B	16.75	0.62
STRENGTH IV	17.17	0.63
SERVICE I	11.66	0.43

d= 12.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Toe Thickness = 16.00 inches 16.00 inches default thickness
Use # 9 at bottom
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

c= 2.18
de=ds= 12.44 inches (for no prestressing)
c/de= 0.18 member is not overreinforced per 5.7.3.3.1-1

Mn= 62.29 ft-K/ft
steel okay

n = 10
x = 4.68 inches
fs = 7.85 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-0.83	8.49	-0.67	6.99
STRENGTH I-A	0.00	11.60	-0.60	11.00
STRENGTH I-B	-1.87	12.01	-0.83	9.30
STRENGTH IV	-1.87	12.43	-1.00	9.55
SERVICE I	-1.39	8.49	-0.67	6.44

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 24.39 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
M_h (UNFACTORED)	2.98
STRENGTH I-A	4.03
STRENGTH I-B	4.03
STRENGTH IV	4.03
SERVICE I	2.98

$AS_{req'd}$
in^2
0.06
0.08
0.08
0.08
0.06

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

WALL LAYOUT - 7+10

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
Ko= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 18.00 degrees (3.1:1 slope)
I= 8.38 degrees
δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.43
Ka= 0.366 ($\delta=\beta$) 0.325 0.291

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Surcharge:

Dead Load - d (ES)= 3.00 ft
Distance to Dead Load Surcharge (a)= 16.50 ft
Live Load - q (LS)= 0.00 ft (Outside of Influence Zone: Zero this Input!)
Distance to Live Load Surcharge (b)= 20.50 ft
Distance to broken backslope (c)= 16.50 ft (I=8.38)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft
Distance Line Load= 0.00 ft

Wall Dimensions:

Design Height (H)= 18.20 ft
batter angle (θ)= 90.00 degrees (vertical)
Wwall= 1.38 ft
Ltoe= 3.33 ft
Htoe= 1.33 ft
Lheel= 5.33 ft
Hheel= 1.33 ft
Lkey= 2.00 ft
Hkey= 1.00 ft
Kdist= 5.72 ft

γ_{conc}= 0.150 kcf

Assume Depth over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 0.00 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 0.00 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 0.65 (Cohesionless (granular) soils) Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 24.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report)
no info f = 0.450 use 2/3*tan(ϕ) if no info
f = μ = 0.450 from Geotech Report

use f= 0.450 (interface friction coefficient) (from Geotech Report)

$FS_{sliding}$= 1.5
$FS_{bearing}$= 2.5 (2.0 allowable if verified by Geotech Report)
$FS_{overturning}$= 2 (2.0 min. footings on soil, 1.5 min. footings on rock)

SUMMARY OF ECCENTRICITY CHECK

B= 10.04 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	e_b	e_{max}	
	K/ft	K/ft	K/ft	K/ft	ft	K/ft	
STRENGTH I-A	19.01	15.10	-35.08	88.67	2.82	2.51	
STRENGTH I-B	26.33	15.10	-40.13	88.67	1.84	2.51	OKAY
STRENGTH IV	27.80	0.00	-39.36	88.67	1.77	2.51	OKAY
SERVICE I	19.80	8.63	-28.79	60.31	1.59	2.51	OKAY

ASD Check ===> OKAY $FS_{overturning}$= 2.13

SUMMARY OF FACTORED BEARING PRESSURES

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 24.00 Ksf

LOAD CASE	V_{TOT}	e_b	σ	q_r	
	K/ft	ft	Ksf	Ksf	
STRENGTH I-A	19.01	2.82	4.32	12.00	OKAY
STRENGTH I-B	26.33	1.84	4.14	12.00	OKAY
STRENGTH IV	27.80	1.77	4.28	12.00	OKAY
SERVICE I	19.80	1.59	2.89	12.00	OKAY

ASD Check ===> OKAY $FS_{bearing}$= 8.31

SUMMARY OF SLIDING RESISTANCE

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)
Φ_{EP}= 0.5 (from Table 10.5.5-1)

LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	19.01	8.55	4.03	8.86	15.10	
STRENGTH I-B	26.33	11.85	4.03	11.49	15.10	
STRENGTH IV	27.80	12.51	4.03	12.02	0.00	OKAY
SERVICE I	19.80	8.91	4.48	9.37	8.63	OKAY
ASD	18.97	8.53	4.48	9.07	8.63	OKAY

ASD Check ===> OKAY $FS_{sliding}$= 1.5

REINFORCING DESIGN

GIVEN:

fy= 40.00 ksi
f'c= 3.00 ksi
COVER= 2.00 inches
$\Phi_{flexure}$= 0.90
b= 12.00 inches

WALL (STEM) REINFORCING

LOAD SUMMARY

HORIZONTAL LOADS

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	H_{TOT}
	K/ft	K/ft	K/ft	K/ft	K/ft
H (UNFACTORED)	0.00	4.00	0.00	0.00	4.00
STRENGTH I-A	0.00	6.01	0.00	0.00	6.01
STRENGTH I-B	0.00	6.01	0.00	0.00	6.01
STRENGTH IV	0.00	6.01	0.00	0.00	6.01
SERVICE I	0.00	4.00	0.00	0.00	4.00

MOMENTS FROM HORIZONTAL FORCES

LOAD TYPE	ΔP_{LSh}	P_{ah}	$T1_{hor}$	$T2_{hor}$	M_{hTOT}
	ft-K/ft	ft-K/ft	K/ft	K/ft	ft-K/ft
Mh (UNFACTORED)	0.00	22.52	0.00	0.00	22.52
STRENGTH I-A	0.00	33.78	0.00	0.00	33.78
STRENGTH I-B	0.00	33.78	0.00	0.00	33.78
STRENGTH IV	0.00	33.78	0.00	0.00	33.78
SERVICE I	0.00	22.52	0.00	0.00	22.52

LOAD TYPE	M_{hTOT}
	ft-K/ft
Mh (UNFACTORED)	22.52
STRENGTH I-A	33.78
STRENGTH I-B	33.78
STRENGTH IV	33.78
SERVICE I	22.52

$AS_{req'd}$
in^2
0.74
1.14
1.14
1.14
0.74

M_{hTOT} (1' up)	M_{hTOT} (1.5' up)	M_{hTOT} (2' up)
ft-K/ft	ft-K/ft	ft-K/ft
21.18	20.51	19.85
31.77	30.77	29.77
31.77	30.77	29.77
31.77	30.77	29.77
21.18	20.51	19.85
	steel okay 1.5' up	steel okay 2' up

d_s= 13.93 inches

per 5.10.8.2 AS_{temp}= 0.30 sq inches
Stem Thickness = 21.80 inches 16.56 inches default thickness
Use # 10 at back face
use spacing= 8.00 inches
As= 1.91 sq. inches

compressive steel:
Use # 0 at front face
As'= 0.00 sq. inches

Mn= 113.85 ft-K/ft
steel okay
steel okay 1' up

c= 2.77
$d_e=d_s$= 13.93 inches (for no prestressing)
c/d_e= 0.20 member is not overreinforced
per 5.7.3.3.1-1
n = 10
x = 6.37 inches
fs = 8.32 ksi

Vn = 25.77 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 39.51 ft-K <— Test 1
1.33M_{hTOT} (max.)= 44.92 ft-K <— Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
HEEL REINFORCING

LOAD SUMMARY

VERTICAL LOADS

LOAD TYPE	ΔPLSv	Pav	VTOT	σLRFD	eLRFD
	K/ft	K/ft	K/ft	Ksf	ft
V (UNFACTORED)	0.00	2.01	2.01	2.89	1.59
STRENGTH I-A	0.00	3.02	3.02	4.32	2.82
STRENGTH I-B	0.00	3.02	3.02	4.14	1.84
STRENGTH IV	0.00	3.02	3.02	4.28	1.77
SERVICE I	0.00	2.01	2.01	2.89	1.59

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	PLSV	ΔPLSv	PEV-heel	Pav	σLRFD	WHeel	MhTOT
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	6.05	0.00	2.22	10.71	-6.65	2.84	15.18
STRENGTH I-A	0.00	0.00	2.22	16.07	-0.20	3.55	21.64
STRENGTH I-B	10.58	0.00	3.00	16.07	-5.59	3.55	27.61
STRENGTH IV	0.00	0.00	3.00	16.07	-6.80	4.26	16.53
SERVICE I	6.05	0.00	2.22	10.71	-6.65	2.84	15.18

LOAD TYPE	MhTOT	ASreq'd
	ft-K/ft	in^2
Mv (UNFACTORED)	15.18	0.51
STRENGTH I-A	21.64	0.74
STRENGTH I-B	27.61	0.96
STRENGTH IV	16.53	0.56
SERVICE I	15.18	0.51

d= 13.44 inches

per 5.10.8.2 AStemp= 0.29 sq inches

Heel Thickness = 16.00 inches 16.00 inches default thickness
Use # 9 at top
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at bottom
As'= 0.00 sq. inches

c= 2.18
de=ds= 13.44 inches (for no prestressing)
c/de= 0.16 member is not overreinforced per 5.7.3.3.1-1

Mn= 62.29 ft-K/ft
steel okay

n = 10
x = 4.68 inches
fs = 10.22 ksi

shear FROM VERTICAL FORCES

LOAD TYPE	PLSV	ΔPLSv	PEV-heel	Pav	σLRFD	WHeel	VhTOT
	K/ft	K/ft	K/ft	K/ft	K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	0.55	0.00	0.83	2.01	-6.19	1.07	-2.80
STRENGTH I-A	0.00	0.00	0.83	3.02	1.33	1.33	5.18
STRENGTH I-B	0.97	0.00	1.12	3.02	-6.80	1.33	-1.69
STRENGTH IV	0.00	0.00	1.12	3.02	-7.63	1.60	-3.49
SERVICE I	0.55	0.00	0.83	2.01	-6.19	1.07	-2.80

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*Mcracking= 21.28 ft-K <--- Test 1
1.33MhTOT (max.)= 36.72 ft-K <--- Test 2

Minimum Reinforcing is provided

REINFORCING (cont.)
TOE REINFORCING
LOAD SUMMARY
VERTICAL LOADS

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}
	K/ft	K/ft	K/ft
V (UNFACTORED)	0.83	9.62	0.67
STRENGTH I-A	0.00	14.38	0.60
STRENGTH I-B	1.12	13.81	0.83
STRENGTH IV	1.12	14.27	1.00
SERVICE I	0.83	9.62	0.67

MOMENTS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	ft-K/ft	ft-K/ft	ft-K/ft	ft-K/ft
Mv (UNFACTORED)	-1.39	16.04	-1.11	13.54
STRENGTH I-A	0.00	23.97	-1.00	22.97
STRENGTH I-B	-1.87	23.01	-1.39	19.75
STRENGTH IV	-1.87	23.78	-1.67	20.24
SERVICE I	-1.39	16.04	-1.11	13.54

LOAD TYPE	M_{hTOT}	$AS_{req'd}$
	ft-K/ft	in^2
Mv (UNFACTORED)	13.54	0.50
STRENGTH I-A	22.97	0.86
STRENGTH I-B	19.75	0.73
STRENGTH IV	20.24	0.75
SERVICE I	13.54	0.50

d= 12.44 inches

per 5.10.8.2 AS_{temp}= 0.29 sq inches

Toe Thickness = 16.00 inches 16.00 inches default thickness
Use # 9 at bottom
use spacing= 8.00 inches
As= 1.50 sq. inches

compressive steel:
Use # 0 at top
As'= 0.00 sq. inches

Mn= 62.29 ft-K/ft
steel okay

c= 2.18
de=ds= 12.44 inches (for no prestressing)
c/de= 0.18 member is not overreinforced
per 5.7.3.3.1-1
n = 10
x = 4.68 inches
fs = 9.12 ksi

SHEARS FROM VERTICAL FORCES

LOAD TYPE	$P_{EV\text{-}toe}$	σ_{LRFD}	W_{Toe}	M_{hTOT}
	K/ft	K/ft	K/ft	ft-K/ft
Mv (UNFACTORED)	-0.83	9.62	-0.67	8.12
STRENGTH I-A	0.00	14.38	-0.60	13.78
STRENGTH I-B	-1.87	13.81	-0.83	11.10
STRENGTH IV	-1.87	14.27	-1.00	11.39
SERVICE I	-1.39	9.62	-0.67	7.57

Vn = 18.92 K/ft
Shear Okay

Minimum Reinforcement per 5.7.3.3.2

1.2*$M_{cracking}$= 21.28 ft-K <--- Test 1
1.33M_{hTOT} (max.)= 30.55 ft-K <--- Test 2
Minimum Reinforcing is provided

REINFORCING (cont.)

KEY REINFORCING

MOMENTS

LOAD TYPE	P_{ph}
	ft-K/ft
Mh (UNFACTORED)	2.98
STRENGTH I-A	4.03
STRENGTH I-B	4.03
STRENGTH IV	4.03
SERVICE I	2.98

$AS_{req'd}$
in^2
0.06
0.08
0.08
0.08
0.06

d= 21.75 inches

per 5.10.8.2 AS_{temp}= 0.43 sq inches

Use # 4 at front face
As= 0.20 sq. inches

compressive steel:
Use # 0 at backface
As'= 0.00 sq. inches

c= 0.29
d_e=d_s= 21.75 inches (for no prestressing)
c/d_e= 0.01 member is not overreinforced
per 5.7.3.3.1-1

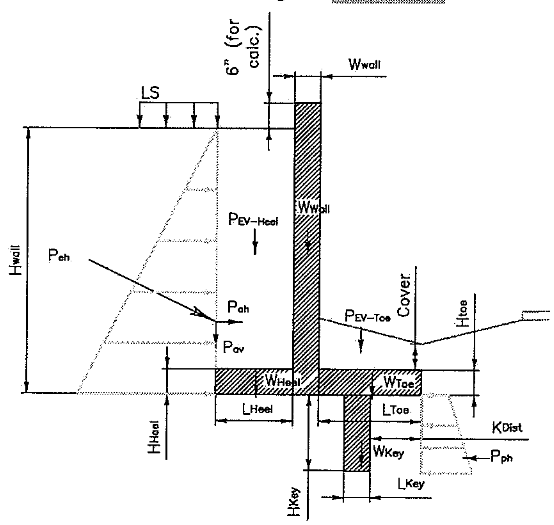
Mn= 14.15 ft-K/ft
steel okay
Vn = 28.37 K/ft
Shear Okay

North West Wall
FAILS

WALL LAYOUT - 4+90 NW

Given:

InSitu Material:	γ=	120	pcf	
Backfill:	γ=	125	pcf	
	Φ=	34.00	degrees	angle of internal friction
	δ=	31.00	degrees	friction angle between fill & wall
	slope of backfill (β) =	18	degrees	
	angle of wall to horizontal (Θ) =	87	degrees	
	Γ =	2.43		
	Ka=	0.366		Ko= 0.44
Wall Dimensions	Hwall=	20.00	ft	
	Wwall=	1.39	ft	γconc= 150 pcf
	Ltoe=	3.08	ft	
	Htoe=	1.50	ft	
	Lheel=	4.75	ft	
	Hheel=	1.50	ft	
	Lkey=	0.00	ft	
	Hkey=	0.00	ft	
	Kdist=	0.00	ft	
Surcharge	LS=	4.90	ft	
	Dist to surcharge=	11.50	ft	



Assume Depth over Toe= 3 ft

LRFD DESIGN:

qult=	9.00	Ksf		(from Geotech Report)
Φbearing=	0.5			(from Geotech Report)
C=	unknown	Ksf	(cohesive soil strength)	
f=	0.45		(interface friction)	(from Geotech Report)
Φsliding=	0.8			(from Table 10.5.5-1)

ASD DESIGN:

Qallow=	3.90	Ksf		(from As-Built Dwgs)
f=	0.45		(interface friction)	(from Geotech Report)
FSsliding=	1.5			
FSbearing=	3			

SUMMARY OF ECCENTRICITY CHECK (NHI)

B= 9.21473 ft

LOAD CASE	V_{TOT}	H_{TOT}	M_{vTOT}	M_{hTOT}	X_O	e_b	e_{max}		e_{LRFD}
	K/ft	K/ft	K/ft	K/ft	ft	ft	ft		ft
STRENGTH I-A	23.06	20.51	158.51	161.58	-0.13	4.74	2.30	[redacted]	[redacted]
STRENGTH I-B	30.57	20.51	195.75	161.58	1.12	3.49	2.30	[redacted]	[redacted]
STRENGTH IV	32.07	13.05	179.54	86.99	2.89	1.72	2.30	OKAY	0.71
SERVICE I	22.36	12.96	140.14	100.62	1.77	2.84	2.30	[redacted]	[redacted]

ASD Check ===> OKAY $FS_{overturning}$= 1.39

SUMMARY OF FACTORED BEARING PRESSURES (NHI)

L'= 1 ft (unit length of wall)

$\Phi_{bearing}$= 0.5
q_{ult}= 9.00 Ksf

LOAD CASE	V_{TOT}	M_{vTOT}	M_{hTOT}	X_O	γq	q_r		σ_{LRFD}
	K/ft	K/ft	K/ft	K/ft	Ksf	Ksf		Ksf
STRENGTH I-A	23.06	158.51	161.58	-0.13	-86.41	4.50	OKAY	[redacted]
STRENGTH I-B	30.57	195.75	161.58	1.12	13.67	4.50	[redacted]	[redacted]
STRENGTH IV	32.07	179.54	86.99	2.89	5.56	4.50	[redacted]	4.11092
SERVICE I	22.36	140.14	100.62	1.77	6.33	4.50	[redacted]	[redacted]

ASD Check ===> [redacted] $FS_{bearing}$= 1.42

SUMMARY OF SLIDING RESISTANCE (NHI)

C= unknown Ksf (cohesive soil strength)
f= 0.45 (interface friction coefficient for ASD design)
Φ_T= 0.8 (from Table 10.5.5-1) δfriction= 24.00 degrees
Φ_{EP}= 0.5 (from Table 10.5.5-1) tanδ= 0.45

LOAD CASE	V_{TOT}	γq_{max}	γq_{min}	$\gamma q_{max}/2$	Q_T	Q_{EP}	$\Phi_T Q_T + \Phi_{EP} Q_{EP}$	H_{TOT}	
	K/ft	Ksf	Ksf	Ksf	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	23.06	10.23	-5.22	5.11	11.53	0.15	9.30	20.51	[redacted]
STRENGTH I-B	30.57	10.85	-4.22	5.43	15.28	0.15	12.30	20.51	NO GOOD
STRENGTH IV	32.07	7.38	-0.42	3.69	16.04	0.15	12.90	13.05	NO GOOD
SERVICE I	22.36	6.92	-2.06	3.46	11.18	0.16	9.03	12.96	NO GOOD
ASD	21.21	2.30	2.30	1.15	10.60	0.16	8.57	12.96	NO GOOD

ASD Check ===> [redacted] $FS_{sliding}$= 0.86
$FS_{sliding}$= 0.7

LRFD LOAD CASE	V_{TOT}	Rt	Rep	Rr	H_{TOT}	
	K/ft	K/ft	K/ft	K/ft	K/ft	
STRENGTH I-A	23.06	10.38	0.15	8.37	20.51	[redacted]
STRENGTH I-B	30.57	13.76	0.15	11.08	20.51	[redacted]
STRENGTH IV	32.07	14.43	0.15	11.62	13.05	[redacted]
SERVICE I	22.36	10.06	0.16	8.13	12.96	[redacted]
ASD	21.21	9.54	0.16	7.72	12.96	[redacted]

WALL LAYOUT - 4+90

Given:

Retained Fill or Foundation:

$\gamma_{retfill}$= 120 pcf (LL & DL surcharge linked to this value)
effective angle of internal friction (ϕ)= 34.00 degrees (use 30 max. if unknown, 34 max. unless backfill is tested)
K_o= 0.4408
C= 0.00 ksf

Slope of Backfill (β)= 18.00 degrees (3.1:1 slope)
I= 12.81 degrees
δ= 31.00 degrees friction angle between fill & wall
angle of wall to horizontal (Θ) = 87 degrees
Γ = 2.43
K_a= 0.366 ($\delta=\beta$) 0.325 0.303

Structure Backfill (Class 1):

$\gamma_{backfill}$= 125 pcf
effective angle of internal friction (ϕ)= 34.00 degrees (34 max. unless backfill is tested)

Works w/ TL
Tiebacks

Surcharge:

Dead Load - d (ES)= 2.90 ft
Distance to Dead Load Surcharge (a)= 11.50 ft
Live Load - q (LS)= 2.00 ft
Distance to Live Load Surcharge (b)= 13.50 ft
Distance to broken backslope (c)= 28.00 ft (I=12.81)
surcharge dissipation slope (x)= 0.50 (1:2 slope)
Line load= 0.00 K/ft Z_2= 0.00 ft
Distance to Line Load= 0.00 ft D_1= 10.00 ft
Width of Line Load (B_f)= 0.00 ft $\sigma_{Bottom\ of\ Wall}$= 0.00 K/ft

Wall Dimensions:

Design Height (H)= 20.00 ft
batter angle (θ)= 90.00 degrees (vertical)
W_{wall}= 1.39 ft
L_{toe}= 3.08 ft γ_{conc}= 0.150 kcf
H_{toe}= 1.50 ft
L_{heel}= 4.75 ft
H_{heel}= 1.50 ft
L_{key}= 0.00 ft
H_{key}= 0.00 ft Assume Depth
K_{dist}= 0.00 ft over Toe= 2 ft

Tieback Anchors:

Tieback 1 (T1)= 2.50 K/ft
Height Below Top of Wall (Y1)= 5.00 ft
Tieback 2 (T2)= 2.50 K/ft
Height Below Top of Wall (Y2)= 10.00 ft
Anchor Inclination= 15.00 degrees (down)
Resistance Factor = 1.00 Proof Tests Required Table 11.5.6-1

LRFD DESIGN:

q_{ult}= 18.00 Ksf (from Geotech Report)
$\Phi_{bearing}$= 0.5 (from Geotech Report)
C= unknown Ksf (cohesive soil strength)
f = μ = 0.450 from Geotech Report
f= 0.45 (interface friction coefficient) (from Geotech Report)
$\Phi_{sliding}$= 0.8 (from Table 10.5.5-1)

LFD DESIGN:

Footing on rock or soil? soil
Qallow= 6.00 Ksf (from Geotech Report) $FS_{sliding}$= 1.5
no info f = 0.450 use 2/3*tan(ϕ) if no info $FS_{bearing}$= 2.5 *
f = μ = 0.450 from Geotech Report $FS_{overturning}$= 2 **
use f= 0.450 (interface friction coefficient) (from Geotech Report)

*(2.0 allowable if verified by Geotech Report) **(2.0 min. footings on soil, 1.5 min. footings on rock)

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

F-16-XB QUANTITIES

203 Embankment Material (Complete In Place)

Abutment 1

use 30 CY

Abutment 4

use 28 CY

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 749 of

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

202 Removal of Slope and Ditch Paving

from U station

Abutment 1 ≈ A ≈ 9609 SF

A ≈ 1068 SY

Abutment 4 = A ≈ 9971 SF with sidewalk
6922 without sidewalk

A ≈ 770 SY

A = 1108 SY if removing sidewalk too

507 Concrete Slope and Ditch Paving (Reinforced)

from U station

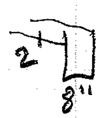
Abutment 1 - 4990 SF
above abut 1 walls - 480 SF
Southwest Grading - 349 SF
} 5819 SF

Abutment 4 - 2841 SF
above abut walls - 469 SF
} 3310 SF

Abut 1 : $5819 \text{ SF} * \frac{4''}{12''} * \frac{1}{27} \approx \underline{72 \text{ CY}}$

Footer walls

$l \approx \underbrace{306' + 306'}_{\text{middle \& bottom}} + \underbrace{8' + 12.5}_{\text{Southwest}} = 632.5'$



$V \approx 632.5' * 1.67' * .67 * \frac{1}{27} \approx 26.2 \text{ CY}$

Cut off walls $l \approx 30' + 17' = 47$

$V = 47 * 1.67 * .67 * 1/27 \approx 2 \text{ CY}$

Use 100 CY

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

507 Concrete Slope and Ditch Paving (Reinforced)

Abut 4: $3310\ SF * \frac{4"}{12"} * \frac{1}{27} \approx 41\ CY$

footerwalls

$l \approx 243' + 243 \approx 486'$

Cut off wall $\approx 20'$

$V \approx (486+20)\ 1.67 * .67 * \frac{1}{27} \approx 21\ CY$

use total $\approx$ 62 CY

202 Structure excavation

Slope Paving

per notes excavation = concrete volume

abutment 1

slope paving $\approx$ 100 CY

pipe - $A_{excav} \approx 166.5\ SF$

$V \approx 166.5(1.5+1.33+1.5)\frac{1}{27} \approx 26.7\ CY$

use total $\approx$ 130 CY

abutment 4

slope paving $\approx$ 62 CY

pipe - $A_{excav} \approx 160.5\ SF$

$V \approx 160.5(1.5+1.33+1.5)\frac{1}{27} \approx 25.8\ CY$

use total $\approx$ 90 CY

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Chk'd:	Date	Structure no.	Sheet 751 of

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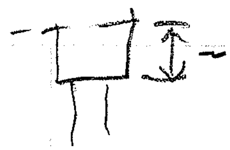
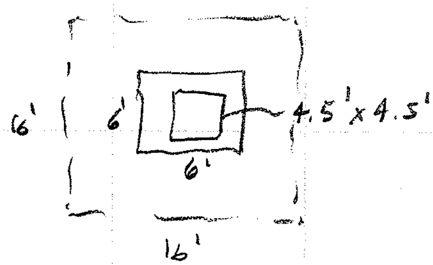
202 Structure Excavation (continued)

Piers

Pier 2 & 3





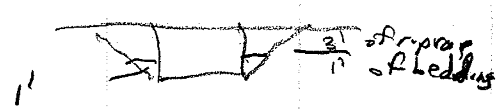
$V \approx 5' * \left(\frac{16*16 + 6*6}{2}\right) \frac{1}{27} \approx 27$ CY per column

total $\approx 8 * 27 \approx 216$ CY

Use 220 CY

206 Structure Backfill (Class 1)

Piers



1' of backfill to bottom of bedding

$V = \left[1 * \left(\frac{8*8 + 6*6}{2}\right) - 4.5 * 4.5 * 1\right] \frac{1}{27}$

$V \approx 1.1$ CY / pier

total $\approx 8 * 1.1 = 8.8$ CY

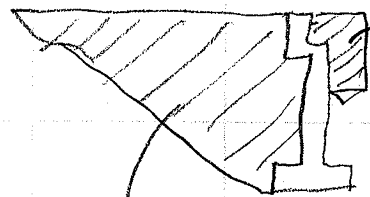
Use 10 CY

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 752 of

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

206 Structure Backfill (Flow-Fill)



A ≈ 50 SF

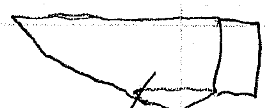
l ≈ 190'

A ≈ 255 SF (assumed)

$V \approx 255 * 190 \frac{1}{27} \approx 1795$ CY

$V = 50$ SF $* 190 * \frac{1}{27} \approx 352$ CY

@ New portions



l ≈ 20'

A ≈ 139 SF

majority of this volume is already accounted for in wall quantities

$V \approx 20 * 139$ SF $* \frac{1}{27} \approx$ 103 CY

total ≈ **2250 CY**

per abutment

could reduce abutment 1 by $20' * 255 SF \frac{1}{27} \approx 188.9$ CY

use 180 CY

would need to replace with MSE

By: Date	Project no.	Project code (SA#):
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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

403- Stone Matrix Asphalt

l = 371.0' BF to BF l = 20' for approach slab
− headers
width = 194' − 2' median = 192'

Superstructure

(371.0) (192') (3"/12") 146.67 lb/CF 1/2000 = 1305.9 tons

additional Ph 4 .907 + .905 SF
(371.0') (.907 + .905 SF) 146.67 1/2000 ≈ 49.3 tons
total ≈ 1355.2 tons
use 1356

Approach Slabs
2(20 ~~21.33~~') (192')(3/12) 146.67 1/2000 ≈ 131.9 tons
− .9

reduction for drains
2 * 5.2 * 4.5' * 3"/12 * 146.67 1/2000 ≈ .9 tons
use 131 tons

512-00101 Bearing Device (Type 1) (EACH)

Bearing Device (Special)
512-00118

2 @ each pier

(2 + 2) 8 ≈ 32 EACH

515-00120 Waterproofing (Membrane)

$(371 - \underbrace{.6 - .6}_{\text{approach slab notch}})$ (194') 1/9 ≈ 7972 SY

Approach Slab - from Ustation: A ≈ [3715 + 3705 − 20(2)(2)] 1/9 ≈ 816 SY

606-10700 Bridge Rail Type 7

2(20 + 371 + 20) ≈ 822 LF

Use 822 LF

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CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

503-00030 Drilled Caisson (30 Inch)

Abutment 1 Tip Elevation ≈ 5148

Abutment 4 Tip Elevation ≈ 5150

Abutment 1

13 @ (5197 − 5148) = 637'
49'

2 @ (5193 − 5148) ≈ 90'
45'

1 @ 5194 − 5148 ≈ 46'

1 @ 5192 − 5148 ≈ 44'

total ≈ 817 LF

Abutment 4

14 @ (5196 − 5150) = 644'
46'

1 @ (5195 − 5150) = 45'
45'

2 @ (5192 − 5150) = 84'
42

total ≈ 773 LF

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

503-00054 Drilled Caisson (54 Inch)

Piers

$l \approx 53'$

8 EA (53') $\approx$ 424 LF / Pier

Pier 2 Tip Elevation = 5131.5

Pier 3 Tip Elevation = 5134.5

G1 = 51.64'
G2 = 51.53'
G3 = 51.41'
G4 = 51.19'
G5 = 50.98
G6 = 50.56
G7 = 50.12
G8 = 49.67

round up to nearest foot

411 LF PIER 2

G1 = 49.33'
G2 = 49.08'
G3 = 48.82'
G4 = 48.51'
G5 = 48.12'
G6 = 47.56
G7 = 46.98
G8 = 46.39

round all to nearest foot

389 LF PIER 3

By:	Date	Project no.	Project code (SA#):
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509 Structural Steel (Galvanized)

W 10x88 @ 3'-6"

wt = (88 lb * 3.5) * 17 caissons ≈ 5236 lb
per abutment

PIERS

JACK PLs 21" SQR x 1½"

wt ≈ 187.6 lbs $= \frac{21}{12} * \frac{21}{12} * \frac{1.5}{12} * 490$

4 - ¾" studs

4 * ¾" stud * $\frac{585}{1000}$ ≈ 2.34 lbs

190 lbs / jackplate

4 jack plates / pier

8 piers * 4/pier * 190 lbs / plate ≈ 6080 lbs

Inspection Bar

1" φ bar - 2.673 lb/ft l ≈ 4.5' ⇒ 12.03 lbs

5" x 5" x ½" PL - 8.51 lb/ft ⇒ 8.51 $\frac{5}{12}$ ≈ 3.55 lbs

3/8" x 5½" Rod - ~.376 lb/ft - ⇒ (.376)($\frac{5.5}{12}$) * 4 ≈ .69 lbs

total wt ≈ 8 piers [12.03 + 2 (3.55 + .69)] ≈ 164 lbs

total ≈ 6244 lbs

use 6244 lbs

By:	Date	Project no.	Project code (SA#):
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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

518-01004 Bridge Expansion Device (0-4 Inch)

$\ell \approx 234'$ along skew

2 reqd.

$L \approx (234)\,2 = \underline{\underline{468'}}$

607-53137 Fence Chain Link (Special) (36 Inch)

Assume both sides on Bridge Rail
above railroad (~50' parallel to rail)
use 100' to either side

$\ell \approx 2(200') \approx \underline{\underline{400\ LF}}$

613-00200 2 Inch Electrical Conduit

$\ell =$ 2sides(371.0) 2 ea = 1484

+ 13.25' @ 3 girders 4 places = 159 LF

lighting & spare

total ≈ 1643 on bridge

weather season - 2 @ 45' = 90'

total = 1733

Approach Slabs

4 corners @ 20' @ 2 EA = 160LF

NE corner 2@20' = 40LF

total = 200LF

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CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

613 - 00150 1½ Inch Electrical Conduit

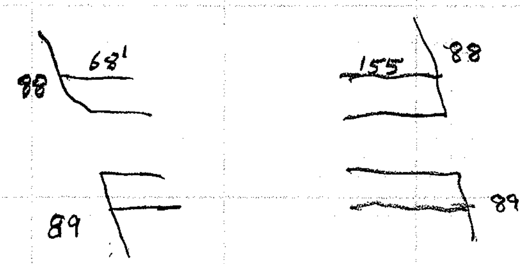
~~2ea (75.83' + 130.83') ≈ 412 LF~~

613-00125 1½ Inch Electrical Conduit

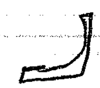
from barrier junction box to junction box & back

assume 20 LF

613-00100 1 Inch Electrical Conduit



+ 2' @ Poles



5*2=10

+2 additional @ sensor

~6' @ verticals

$2(88) + 4(68) + 4(155) + 2(89) + 3(6') + 6(2')$ + 3*10'

↑ weather sensor

total ≈ 1306 LF

By:	Date	Project no.	Project code (SA#):
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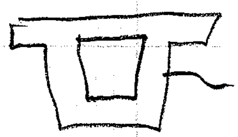
601-03040 Concrete Class D (Bridge)

Girder Closure Pours



$A \approx 28.26$ SF

$V = 4$ closure pours $*$ 8 girder lines $*$ $28.26\ SF/_{27} \approx$ 33.5 CY

Girder Diaphragms

$A \approx 16$ SF

$\ell \approx 3'\text{-}4''$

$\ell = 2'\text{-}9''$ - 20' girders

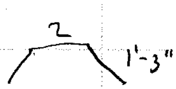
$V = [(3.33) * 4 + 2.75 * 4]\, 16\, \frac{1}{27} \approx$ 14.4 CY

Longitudinal Phasing Pours

2'-10" wide, 8" thick $\ell \approx 367'$ front face to front face

$V = (2.83)(.67)\, 367 * 4$ EA $\frac{1}{27} \approx$ 103.1 CY

Light Pole Pedestals



$\ell \approx 3.57'$

$3.57' * (2 * 1.25 + (1.25)\, 1.25)$ 5 EA $\frac{1}{27} \approx$ 2.7 CY

By:	Date	Project no.	Project code (SA#):
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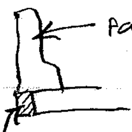
COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

601-03040 Concrete Class D (Bridge) (cont.) (CY)

DECK AT BRIDGE RAIL

Paid for as Bridge Rail Type 7



paid for as Bridge Rail Type 7

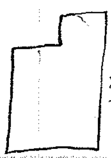
5" 8"

$A \approx .28$ SF

$\ell \approx 385.25' + 366.8'$

$V \approx .28(385.25+366.8)\frac{1}{27} \approx$ ~~7.8 CY~~

Abutment (above bearing seat)



7'

2.5'

abut 1 $\ell \approx 236'$

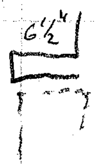
girder area $\approx 12.25 * 2' * \frac{1}{27} \approx .9$ CY

$V \approx 236[2.5*7 - .5*1]\frac{1}{27} - 8(.9) \approx$ 141.4 CY

abut 4 $\ell \approx 234.6'$

$V \approx 234.6[2.5*7 - .5*1]\frac{1}{27} - 8(.9) \approx$ 140.5 CY

Abutment overhangs



6½" 8"

$\ell \approx 3.21'$ NW
$\ell = 4.7'$ NE
$\ell = 3'$ SE
$\ell = 4.7'$ SW

$V = (4.7 + 3 + 4.7 + 3.2)(.54)(.67)\frac{1}{27} \approx$ .2 CY

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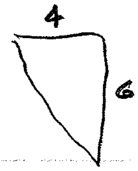
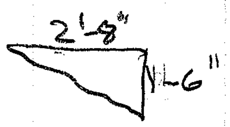
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

extra concrete deck at northwest corner



$\frac{1}{2}(4)(6) * .67 * \frac{1}{27} \approx .3$ CY



$\left[\frac{1}{2}(2.67)(1.5) * 1.79' + \frac{1}{2}(2.67)(1.5) * 3' * \frac{1}{2}\right]\frac{1}{27}$

$\approx .25$ CY

total $\approx$ .6 CY

SUPERSTRUCTURE TOTAL $\approx$ 436.4 CY

use 437 CY

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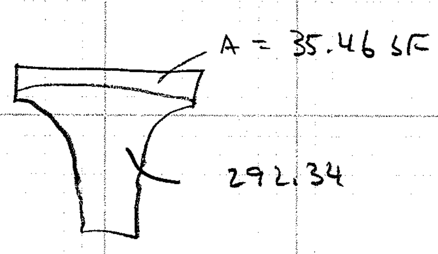
COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

601-03040 Concrete Class D (Bridge)

Piers



$V = 292.34(4.5) + 35.46(4.5+1.17+1.17)$

$V \approx 1557.9\,CY \approx 57.7\ CY/\text{pier}$

$8 * 57.7 \approx$ 462 CY/ pier line

Abutments (below bearing seats)

Abutment 4

Face Area $\approx$ 24 SF + 807.8 SF
from U station

+2.1

$V \approx [(24 + 807.8)2.5 + 2.1 * 3.5]\ 1/27 \approx 77.3\ CY$

Columns

$V = (17.7 + 17.7 + 14.7 + 13.7 + 13.5 + 13.2 + 12.9 + 12.7$
$+ 12.5 + 12.4 + 12.1 + 11.6 + 11.1 + 10.8 + 10.6$
$+ 10.4 + 10.2)\left(\pi \frac{2.5^2}{4}\right)\frac{1}{27} \approx 39.6\ CY$

$l_{tot} \approx 217.8'$

total $\approx$ 116.9 CY

Use 117 CY

By:	Date	Project no.	Project code (SA#):
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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

Abutment 1

Face Area ≈ 860.7 SF

$V \approx (860.7 * 2.5)\ \frac{1}{27} \approx$ 79.7 CY

Columns

$V = [2 * 15.53 + 8 * 11.53 + 11.5 + 11.2 + 11.0 + 10.9$
$+ 10.7 + 13.5 + 15.5] * \pi \frac{2.5^2}{4} \frac{1}{27} \approx$ 37.8 CY

$\ell_{tot} \approx 207.6'$

total ≈ 117.5 CY

Use 118 CY

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CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

601 Concrete Class D (Bridge)

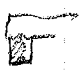
Approach Slabs

Abut 4 – $A \approx 4061.62$ SF

Abut 1 – $A \approx 4081.13$ SF

length of sleeper slab $\approx 235.9'$

$V = (4061.62 + 4081.13)\ 1' \frac{1}{27} \approx 301.6$ CY



$V_{overhang} = (20' * 1' * .5) * 2 * 2 * \frac{1}{27} \approx 1.5$ CY

$V_{sleeper\ slab} \approx 2\ (235.9') * [4 * 1 + 1.25\ (1)] \frac{1}{27} \approx 91.8$ CY

$V_{header} \approx 2 * (.25)(1')\ 234' \frac{1}{27} \approx 4.4$ CY

deduction for drains

$V \approx 4 * (3')\ (2.2')(1')\ \frac{1}{27} \approx -.9$ CY

total ≈ 398.4

Use 399 CY

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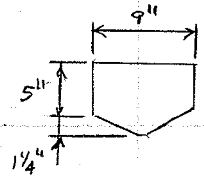
CDOT Form #1034 3/02

601-06400 Grout (Bridge) (special) (CF)

TRANSVERSE CLOSURE POURS



38 slabs ⇒ 37 closure pours

$A \approx .3566$ SF

Phase 1&2 length ≈ 51.97' * 2
Phase 4 length = 48.25'
Phase 3 & 5 length ≈ 35.38'

Phase 1&2: (51.97)(2)(.3566)(37) ≈ 1371.4 CF

Phase 4: (48.25)(.3566)(37) ≈ 636.6 CF

Phase 3: 35.38 (.3566)(37) ≈ 466.9 CF

Phase 5: 35.38 (.3566)(37) ≈ 466.9 CF

TOTAL ≈ 2942 CF

3263

HAUNCH & SHEAR BLOCKOUTS

5.52"
1% 4.35"
5.23"

Avg Haunch ≈ 5.5"

girder lengths = 75.5', 126', 123.5'
haunch width ≈ 64"
use diaphragm to diaphragm lengths

Shear Blockout Area ≈ 1.1 SF
Depth = 8"   Vol ≈ .733 CF

5.52"
2% 5.33"
5.26"

Phase 1&2,4 - 16 per panel
Phase 3&5 - 8 per panel

4.56"
5"
4.97"

Phase 1&2: $\frac{(64")(5.5")}{144}$ (71.5 + 13.5 + 120.5 + 13.5 + 120) 4 girders + (2)(38)(16)(.733)

V ≈ 828.7(4) + 891.3 ≈ 4206 CF

Phase 4: V = 828.7(2) + (38)(16).733 ≈ 2103 CF

Phase 3: V ≈ 828.7(1) + 38(8).733 ≈ 1052 CF

Phase 5: V ≈ 828.7(1) + 38(8).733 ≈ 1052 CF

TOTAL ≈ 8413 CF

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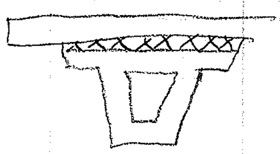
601- 06400 Grout (Bridge) (Special) CONTINUED

HAUNCH AT PIERS (CLOSURE POURS & DIAPHRAGMS

Use 5½" Haunch

length = 2' each width ≈ 131"

+ 2.75+3.33 ≈ 7.1'



PHASE 1 & 2: $2*(7.1)(2 \text{ piers})(2 \text{ girders})\frac{(131")(5.5")}{144} \approx 284.2$ CF

PHASE 4: $(7.1)(2 \text{ piers})(2 \text{ girders})\frac{(131")(5.5")}{144} \approx 142.1$ CF

PHASE 3: $(7.1)(2 \text{ piers})(1 \text{ girder})\frac{(131")(5.5")}{144} \approx 71.1$ CF

PHASE 5: $(7.1)(2 \text{ piers})(1 \text{ girder})\frac{(131")(5.5")}{144} \approx 71.1$ CF

568.5

TOTAL ≈ 569 CF

GRAND TOTAL = 2942 + 8413 + 569

= 11924 CF

Use 11925 CF

11924

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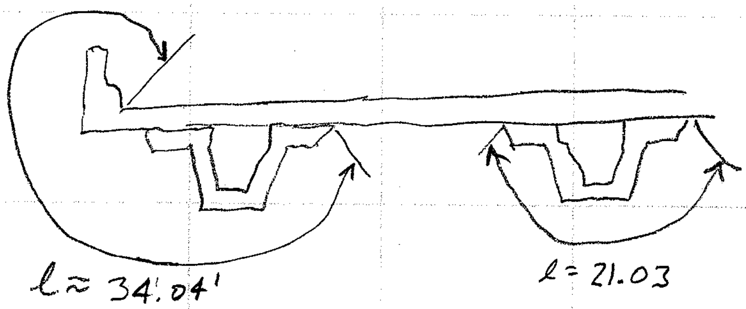
COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

601 - 40300 Structural Concrete Coating (SY)

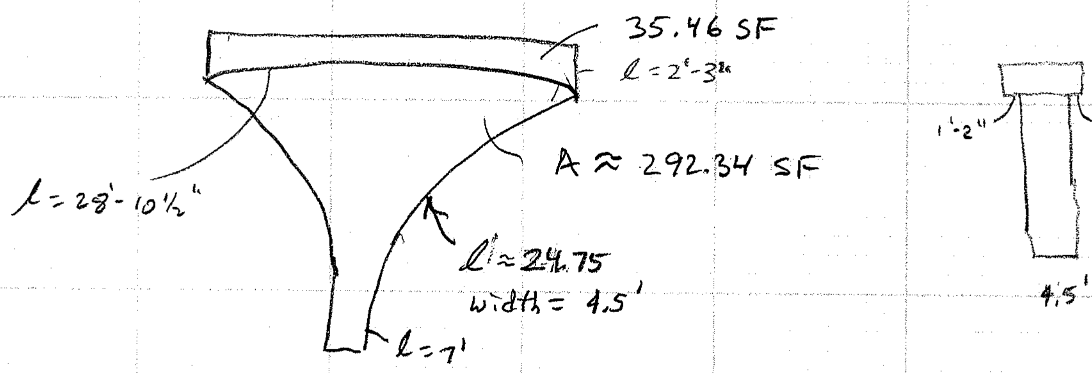


$l \approx$ backface to backface ≈ 371

$$A \approx \left[2(34.04')(371) + 6(21.03)(371')\right]\frac{1}{9}$$

$A \approx$ 8008 SY (super)

$A_{total} = 8008 + 417 \approx$ 8425 SY



$$A \approx \left[2(292.34+35.46) + (24.75+7+2.25)\,4.5(2) + 2(1.167)(28.875+2.25+2.25)\right]\frac{1}{9} \approx 115.5 \text{ SY/pier}$$

$8 * 115.5 \approx$ 924 SY / pier line

Abutments

Abutment 4 $\approx (2063 - 51.4*8)\frac{1}{9} \approx 183.5$ SY
use 190

Abutment 1 $\approx (2089 - 51.4*8) \approx 186.4$ SY
use 190 SY

By:	Date	Project no.	Project code (SA#):
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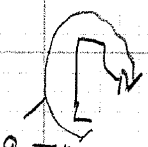
601-40300 Structural Concrete Coating (SY)

Approach Slabs ≈ ~ 

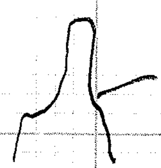
$l \approx 9.7'$

$4 * 20' * 9.7 * \frac{1}{9} \approx 86.2$ SY

total = 86.2 + 44.9 ≈ 131.1 SY

Use 132 SY

Median Barrier

 $l \approx 10.1$ LF

$(371)(10.1)\frac{1}{9} \approx 416.4$ SY superstructure

$2 * 20 * (10.1)\frac{1}{9} = 44.9$ SY approach slabs

By:	Date	Project no.	Project code (SA#):
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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated)

strand ≈ .75 lb/ft

TRANSVERSE CLOSURE POURS

2 - 3/8" strands continuous   37 closure pours
use #4 for weight

Phase 1: (37)(2)(51.97') .668 lb/ft ≈ 2569 lbs

Phase 2: (37)(2) (48.25') .668 lb/ft ≈ 2386 lbs

Phase 3: (37)(2) (35.4') (.668 lb/ft) ≈ 1750 lbs

Phase 4: (37)(2) (35.4) (.668 lb/ft) ≈ 1750 lbs

Total ≈ 8455 lbs

May need mechanical splices for phases 2 & 4

LONGITUDINAL CLOSURE POURS

8 - #5s continuous $\ell \approx$ 366.67' $\frac{366.67}{60}$ ⇒ 7 pieces
splice ≈ 1'-7"
# of splices ⇒ 6

Phase 3: 2(8)[366.67 + 6(1.583)] 1.043 lb/ft ≈ 6278 lbs

Phase 4: 2(8)[366.67 + 6(1.583)] 1.043 lb/ft ≈ 6278 lbs

Total ≈ 12556 lbs

~~DECK AT BRIDGE RAIL~~

~~2 - #5s continuous $\ell \approx$ 385.25 + 366.8~~
~~6 splices~~

~~(2)(2)[385.25 + 366.8 + (2)6(1.583)] 1.043 lb/ft ≈ 3217 lbs~~

Paid for as Bridge Rail Type 7

5s transverse in closure pours

5 per panel × 3'-1"

4 * 2 * 38 panels * 5 * 3.083 * 1.043 ≈ 4888
closure pours   T&B

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated) (cont.)

Extended Deck @ Abutments

2 #5s continuous
#5s @ 6" top & bottom

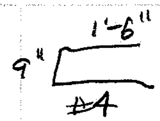
concrete lengths,
l = 3.21, 4.7', 3' & 4.7'
width ≈

long. wt ≈ $(2.88 + 4.37 + 2.67 + 4.37)\ 2 * 1.043\ \text{lb} \approx$ **30 lbs**

transverse = $.79' * \Big(\underbrace{\frac{2.88}{.5} + \frac{4.37}{.5} + \frac{2.67}{.5} + \frac{4.37}{.5}}_{7\text{ bars} + 10\text{ bars} + 7\text{ bars} + 10\text{ bars}}\Big) * 2 * 1.043 \approx$ **56 lbs**

total ≈ **86 lbs**

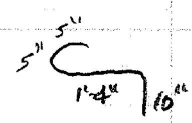
Bars at Post tensioning Anchors



79 ducts - 2 abutments

$2\ (79) * 2 * (1.5 + 1.5 + .75)\ \ .668\ \text{lbs/ft} \approx$ **792 lbs**

Hook Bars

#5 

$l \approx 3'$

26 in phase 1 & 2
22 in phase 4
20 in phase 3 & 5

2 abutments $[26(2) + 22 + 20(2)] * 3' * 1.043 \approx$ **714 lbs**

#5 @ top of abutment in deck

total ≈ 238'

40' stock lengths
6 bars - 5 laps @ 1'-7"
4 phase laps @ 1'-7"

$2 * 2\ [238 + 5(1.583) + 4(1.583)]\ 1.043\ \text{lbs} \approx$ **1053 lbs**

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing steel (Epoxy coated) (cont.)

Bars for NorthWest corner Abut 4

11 - #8 2'6 ⌐ 2'0 min 5-4" max 3'-8" avg

11 * (2.5+3.67) * 2.67 lb/ft ≈ 182 lbs

#5 @ 6" 8'-4" max 1'-2" min

Top transverse 10 bars 4'-9" avg.

10 * 4.75 * 1.043 lbs ≈ 50 lbs

bottom transverse
#5 @ 6" 8" min, 4'-4" max

6 bars 2'-6 avg

6 * 2.5 * 1.043 lbs ≈ 16 lbs

long. ~12 bars

6' max

1' min

avg ≈ 3.5'

12 * 3.5' * 1.043 ≈ 44 lbs

total ≈ 292 lbs

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602-00020 Reinforcing Steel (Epoxy Coated) (cont.)

Abutment Reinforcing above Bearing seat

11 continuous @ top

℄ abut length ≈ 234.6 Abut 4
use for both → 235.92 Abut 1

phase 1 & 2 - 51.7' + 7'-3"
phase 4 - 48.25' + 7'-3" ×2
phase 3 & 5 - 42.25'

$$wt = 2\left[(51.7+7.25)+(51.7+7.25)+42.25+42.25+(48.25+7.25+7.25)\right] * 5.313$$

$wt \approx$ 2818 lbs

7 Transverse 5 EA

30.2' girder to girder + 2'-7" splice
reduce 2.7' for end girders

assume splices moved to match phasing

12.4

$$wt = \left[6(30.2+2.583) + 2(30.2+2.583-2.7)\right] * 5 * 2.044 \text{ lb/ft}$$

$\approx$ 2625 lbs

5 @ Front face

1 @ 24.75
1 @ 17 between each girder

1 @ 9'
1 @ 5' @ end girders

$7 * (24.75+17) + 2(9+5) * 1.043 \text{ lb/ft} \approx$ 334 lbs

4 splices $4 * 2 * 1.583 * 1.043$ lb/ft $\approx$ 14 lbs

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated) (Cont.)

#5 @ back face

#5s @ 1'-0"

7 bars

$$wt = 7\left[51.7 + 1.75 + 51.7 + 1.75 + 42.25 + 42.25 + 48.25 + 1.75 + 1.75\right] 1.043$$

$wt \approx$ **1776 lbs**

mechanical anchors

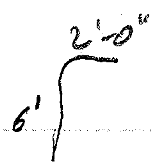
#11 anchor length = 5'-4"

assume 2 per phase

$2(4) * 5.33 * 5.313$ lb $\approx$ **227 lbs**

stirrups

#5



phase 1 = $\frac{51.7}{1} \approx 52$ spaces ⇒ 53 bars + 1 at phase

phase 2 = phase 1

phase 3 = $42.25/1$ = 43 spaces ⇒ 44 bars + 2 at either end

phase 5 = phase 3

phase 4 = $\frac{48.25}{1}$ = 49 spaces ⇒ 50 bars + 2 @ either end

$8' * (54 + 54 + 46 + 46 + 52)$ 1.043 lb/ft $\approx$ **2103 lbs**

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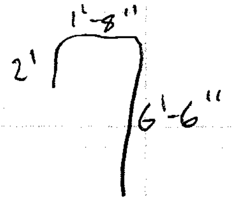
602-00020 Reinforcing Steel (Epoxy Coated) (Cont.)

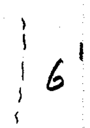
Stirrups



$(6.5+1.67+2)\left[54+54+46+46+52\right]1.043$

$\approx$ 2673 lbs

delete @ girders



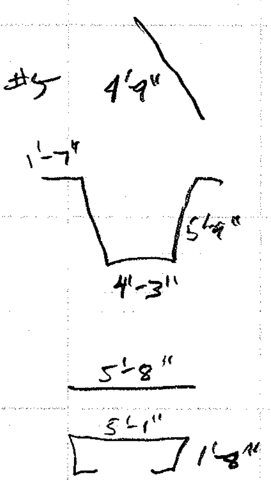
8 @ each girder

$8 * 8 * 6' * 1.043 =$ -400 lbs

Total per abutment = 12170 lbs

GIRDER CLOSURE POURS

#5



$l \approx 2(4.75) + 18.92 + 5.67 + 9.923$

$l_{tot} \approx 44.0'$ (2 per closure)

4 closures per girder

$wt = 4 * 8 * 44 * 2 * 1.043$

wt $\approx$ 2938 lbs

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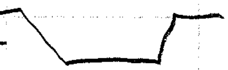
COLORADO DEPARTMENT OF TRANSPORTATION

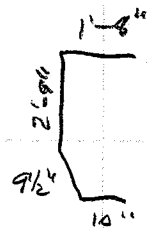
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing (Epoxy Coated)

Light Pedestals

6- #4  $l \approx 7.42'$ wt= 6*7.42*.668 = 29.8 lbs

2- #4 x2'-8" wt= 2*2.67*.668= 3.6 lbs

6- #4 

$l = 6.04$ wt = 6*6.04*.668 = 24.2 lbs

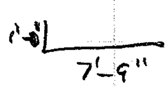
total = 57.6 lbs

use 58 lbs/pedestal

5 pedstals on bridge

5 * 58 = 290 lbs

Barrier Steel in Closure Pours

#6 

2side(37) * 8.75' * 1.502 lbs ≈ 973 lbs

#4  $l \approx 4.74'$ 2 sides(37) 4.74 * .668 lbs ≈ 235 lbs

total ≈ 1208

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated) (cont)

Deck shear steel not in girders

end anchors ~ .8 lbs

#6 x 2'-10"

assume 4 per location
~ 9 locations per girder

8 * 4 * [9 * 2.83 * 1.502 + .8] ≈ 1250 lbs

Total super structure ≈

4888 + 8455 + 12556 + 86 + 792 + 714 + 1053 + 292
+ (2 * 12170) + 2938 + 1250 + 290 + 1208

subtotal ≈ 58862 lbs

187

Phase I steel Connected to panels

#5 24 per panel l ≈ 3'-4" + 1"

38 panels

wt = 3.42' * 1.043 * 24 * 38 ≈ 3254 lbs

Superstructure Total = 62116 lbs

Use 62120 lbs

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602 - 00020 Reinforcing Steel (Epoxy Coated)

Abutment

3' 425" + 16.42 ≈ 22"
use 2'

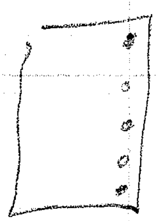
$\# 11$ continuous

$$wt = 4 * [(51.7 + 3 + 2) + (51.7 + 3 + 2) + (42.25 + 2 + 2') + (42.25 + 2' + 2') + (48.25 + 3' + 3' + 2 + 2)]\ 5.313$$

$$wt \approx 5614\ lbs$$

$\# 5$ each face

use 3 each face



$$wt = 6 * [(51.7 + 1.75) + (51.7 + 1.75) + 42.25 + 42.25 + (48.25 + 1.75 + 1.75)]\ 1.043$$

$$wt \approx 1522\ lbs$$

$\# 5$ 5.583 max, 2'-2"

$$(5.58 + 2.16 + 5.58) * (54 + 54 + 46 + 46 + 52)\ 1.043 \approx 3505\ lbs$$ (E)

$\# 5$ 1-7 2'-2"

$$(2.17 + 1.58 + 1.58) * (54 + 54 + 46 + 46 + 52)(1.043) \approx 1403\ lbs$$

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated)

Abutment

#6 x 4' each side of girder

4' * 2 * 8 * 1.502 lb/ft ≈ 96 lbs (E)

Bar INTO approach slab

#5 1' 2'-9" @ 1'-0" = 240 bars * 3.75 * 1.043 ≈ 940 lbs

Total Abutment ≈ 13080 lbs

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

602-00020 Reinforcing Steel (Epoxy Coated)

Phase 2 = Phase 1 = 52.47' Phase 4 = 48.65
Phase 3 = 38.8' Phase 5 = 45.23'

Approach Slabs

Bottom Longitudinal

#6 @ 6" $\ell = 20.5 - 2" - 2" = 20.17'$

Phase 1 & 2 105 spaces → 106 bars
Phase 3 77 spaces → 78 bars
Phase 4 97 spaces → 98 bars
Phase 5 90 spaces → 91 bars

$wt = 20.17'(106+106+78+98+91)\ 1.502 \approx$ 14512 lbs

Top Longitudinal

#4 @ 1'-6" 20.17 7" 1'-0" $\ell = 23.34'$

Phase 1 & 2 - 35 spaces → 36 bars
Phase 3 - 26 spaces → 27 bars
Phase 4 - 33 spaces → 34 bars
Phase 5 - 30 spaces → 31 bars

$wt = 23.34'(36+36+27+34+31)\ .668 \approx$ 2557 lbs

Transverse

#5 @ 1' 4 splices
21 spaces - 22 bars

$2 * 22 [52.3' + 52.3 + 38.8 + 48.65 + 45.23 + 4(1.75)]\ 1.043 \approx$ 11211 lbs

1'-7" $2 * 22 * 1.583 * 1.043 \approx$ 73 lbs

2 #5 : $2 * 2 * 20' * 1.043 \approx$ 84 lbs

By: Date	Project no.	Project code (SA#):
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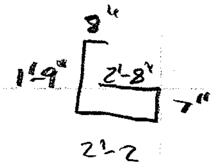
602-00020 Reinforcing Steel (Epoxy Coated)

Sleeper Slab

$\ell \simeq 235.8'$

12 - #5 cont.

#5 

@ 1'-0" = ℓ = 7.84'

longitudinal

12 * [235.5 + 5(1.583)] 1.043 ≈ 3047 lbs

stirrups

2(7.84') * 240 bars * 1.043 ≈ 3925 lbs

DRAINS

5 - #8 x 8'	5 * 8' * 2.67 ≈	107 lbs	251 lbs per drain
1 - #8 x 6'	1 * 6' * 2.67 ≈	16 lbs	
4 - #8 x 9.583'	4 * 9.583' * 2.67 ≈	103 lbs	
4 - #5 @ 6'-0"	4 * 6' * 1.043 ≈	25 lbs	

total ≈ 502 lbs

Deductions

#6	7 * 2.75' * 1.502 lbs ≈ 29 lbs	52 lbs per drain
#4	3 * 2.75 * .668 lbs ≈ 6 lbs	
#5	6 * 2.83 * 1.043 lbs ≈ 17 lbs	

total ≈ − 100 lbs

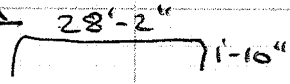
TOTAL APPROACH SLAB ≈ 35 811 lbs
USE 2 APPROACH SLABS = 71 625

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid) 602-00000 Reinforcing Steel

PIER REINFORCING

TOP Horizontal

#10

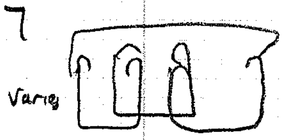


11 ea

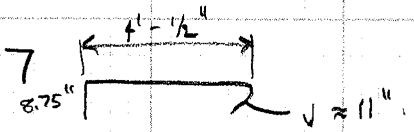
11 * (28.167 + 1.833 + 1.833) = 350.167' * 4.303 lb/ft ≈ 1507 lbs

Transverse Ties

5 - #7



#7

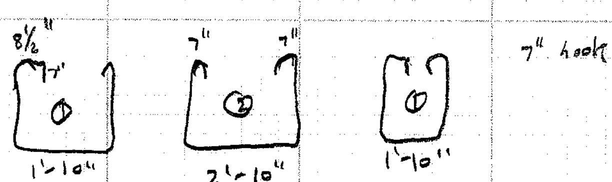


$\ell \approx 5.6875$"

from U station true length ≈ 5.5'

5 * 5.6875" = 28.44' * 2.044 lb/ft ≈ 58.1 lbs

2 sides ⇒ 117 lbs



1 set @ 2'-1" $\ell_1 \approx 8.58'$ $\ell_2 \approx 9.33'$

1 set @ 2'-4½" $\ell_1 \approx 9.17'$ $\ell_2 \approx 9.92'$

1 set @ 2'-8" $\ell_1 \approx 9.75'$ $\ell_2 \approx 10.5'$

1 set @ 3'-0" $\ell_1 \approx 10.42'$ $\ell_2 = 11.17'$

1 set @ 3'-4" $\ell_1 \approx 11.08'$ $\ell_2 \approx 11.83'$

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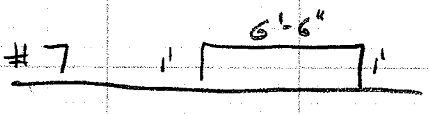
PIER REINFORCING (cont.)

#7 ties  (continued)

Length = 2 (8.58) + 9.33 + 2 (9.17) + 9.92 + 2 (9.75) + 10.5
+ 2 (10.42) + 11.17 + 2 (11.08) + 11.83

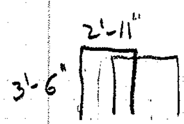
≈ 150.75' * 2.044 lb/ft ≈ 308.1 lbs

2 sides ⇒ 617 lbs

#7 

10 total * (6.5 + 1 + 1) ≈ 85' * 2.044 lb/ft ≈ 174 lbs

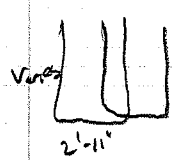
2 sides ⇒ 348 lbs ✓

#7 

7 * (3.5 + 2.92 + 3.5) * 2 ≈ 138.9' * 2.044 lb/ft = 284 lbs

2 sides ⇒ 568 lbs

#7

7 – 

1 @ 3'-2½"
1 @ 3'-8¾"
1 @ 4'-3"
1 @ 4'-10½"
1 @ 5'-10¼"
1 @ 6'-11½"
1 @ 8'-3"

$\ell = 2_{\text{legs}} * 2 * (3.21 + 3.73 + 4.25 + 4.88 + 5.86 + 6.96 + 8.25) + 7 * 2 * 2.92$

$\ell \approx$ 189.44' * 2.044 lb/ft ≈ 387.2 lbs

2 sides ⇒ 775 lbs

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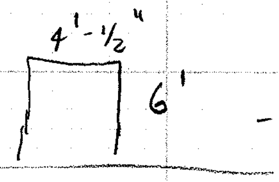
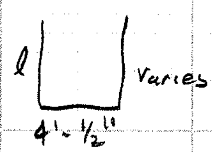
PIER REINFORCING (cont)

#5 

total both sides = 8 + 8 + 9 = 25 EA

25(6'+6'+ 4.042') ≈ 401.05' * 1.043 lb/ft ≈ **419 lbs**

#5 

8 ~~total~~ (1 side)

1 @ 5'-7½" l = 2(5.625) + 4.042 ≈ 15.292'
1 @ 6'-4¾" l ≈ 2(6.396) + 4.042 ≈ 16.834'
1 @ 7'-3" l ≈ 2(7.25) + 4.042 ≈ 18.542'
1 @ 8'-2½" l ≈ 2(8.208) + 4.042 ≈ 20.458'
1 @ 9'-3½" l ≈ 2(9.292) + 4.042 ≈ 22.626'
1 @ 10'-6½" l ≈ 2(10.542) + 4.042 ≈ 25.126'
1 @ 12'-¾" l ≈ 2(12.063) + 4.042 ≈ 28.168'
1 @ 14'-1¾" l ≈ 2(14.146) + 4.042 ≈ 32.334'

Total length ≈ (15.292 + 16.834 + 18.542 + 20.458 + 22.626
+ 25.126 + 28.168 + 32.334) ≈ 179.38' * 1.043
≈ 187.2 lbs

2 sides ⇒ **375 lbs**

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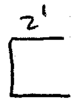
COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

PIER REINFORCING (cont.)

PIER OVERHANG

#4 Varies  @ 1'-6" ±

1 @ 1'-10 1/4"

1 @ 1'-6 3/4"

1 @ 1'-3 1/2"

1 @ 1'- 3/4"

1 @ 11"

1 @ 9 1/2"

1 @ 7 3/4"

1 @ 6 1/2"

1 @ 5 1/2"

1 @ 5"

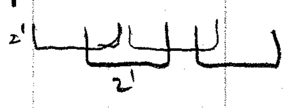
1 @ center @ 4 3/4"

total length ≈ 2 [1.854+4+1.563+4+1.292+4+1.063+4+.917+4
+ .792+4 + .646+4 + .542+4 + .458+4
+ .417+4] + .396+4

length ≈ 103.484' * .668 lb/ft ≈ 69.1 lbs

two sides ⟹ 139 lbs

#4



4 @ 6' each 4 places

length ≈ 4(6') 4 = 96' * .668 lb/ft ≈ 65 lbs

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

PIER REINFORCING (cont.)

PIER OVERHANG (cont.)

#5 $\ell \approx 28'-6\frac{1}{2}''$

length = 4 * 28.542' ≈ 114.168' * 1.043 ≈ 119 lbs

#5 28'-4"

length = 4 * 28.333 ≈ 113.33' * 1.043 ≈ 119 lbs

#4 2' length = 4 * 2 ≈ 8 * .668 ≈ 6 lbs

PIER ENDS

#6 $7\frac{1}{2}''$ $3'-10\frac{1}{2}''$ 3 each

2 sides * 3 each * (.625 + 3.875 + .625) ≈ 30.75' * 1.502 lb/ft ≈ 47 lbs

9-#5 $\ell \approx 22'-8''$

2 sides * 9 each * 22.667' ≈ 408' * 1.043 lb/ft ≈ 426 lbs

#6 $7\frac{1}{2}''$ $3'-10\frac{1}{2}''$ 4 each

2 sides * 4 each (.625 + 3.875 + .625) ≈ 41' * 1.502 ≈ 62 lbs

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COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

PIER REINFORCING (cont.)

PIER FACE

#6 @ 6" each face

3 @ 28'-4"
1 @ 27'-8 3/4"
1 @ 26'-1/2"
1 @ 24'-6"
1 @ 23'-1"
1 @ 21'-9 1/4"
1 @ 20'-6 1/2"
1 @ 19'-4 3/4"
1 @ 18'-4"
1 @ 17'-4"
1 @ 16'-4 1/2"
1 @ 15'-5 3/4"
1 @ 14'-7 3/4"
1 @ 13'-10 1/4"
1 @ 13'-1"
1 @ 12'-4 1/2"
1 @ 11'-8 1/2"
1 @ 11'-3/4"
1 @ 10'-5 3/4"
1 @ 9'-10 3/4"

total length ≈ 412.7' one side

(2 sides) * 412.7' * 1.502 lb/ft ≈ 1240 lbs

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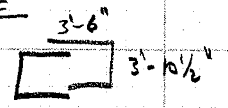
PIER REINFORCING (cont.)

PIER FACE

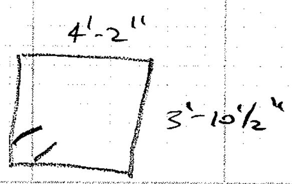
11 - #5 

@ 9"

$11 * 2 * (3.5 + 3.875 + 3.5) \approx 239.25' * 1.043\ lb/ft \approx \underline{250\ lbs}$

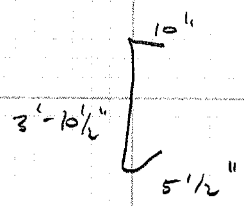
COLUMN CORE

TRANSVERSE

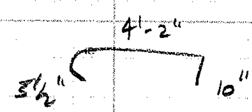
#5 

bar extension ≈ 5½"

$l = 2(4.167 + 3.875) + .458 + .458 \approx 17'$



$l \approx 3.875 + .458 + .833 \approx 5.167'$



$l \approx 4.167 + .458 + .833 \approx 5.458'$

total ≈ 27.625' per tie set

tie sets = 25 @ 9"
11 @ 12"

total length = $(25 + 11)\ 27.625' \approx 994.5' * 1.043\ lb/ft \approx \underline{1038\ lbs}$

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COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

PIER REINFORCING

COLUMN CORE

LONGITUDINAL

24 - #10 $\quad l \approx 29'\text{-}6'' - 4''_{\text{top}} - 2''_{\text{bottom}} \approx 29'$

$24 * 29' \approx 696' * 4.303 \approx$ 2995 lbs

7 - #5 $\quad l \approx 29'$ each side

$14 * 29' \approx 406' * 1.043$ lb/ft $\approx$ 424 lbs

+6

PIER TOTAL $\approx$ 11656 lbs

8 piers/pierline

$8\,(11656) \approx$ 93248 lbs

use 93250 lbs

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 789 of

CDOT Form #1034 3/02

618-10072 Pre cast Concrete U Girder (U72) (Pre-Tensioned)

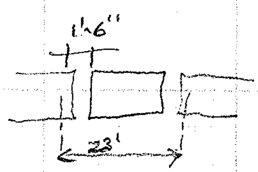
Span lengths - 76' + 19' + 127' + 19' + 124' = 365'

Length = 8 EA (76 + 19 + 127 + 19 + 124)

= 2920 LF

optional pier girders $\ell \approx 20'$

(8 EA)(2 piers)(20') ≈ 320 LF



618-00002 Prestressing Steel Wire or Strand MKFT

million / 1000

$\ell \approx 366.25$

NB & SB phase 1 & 2 18 ducts * 4 strands/duct * 2 = 144 strands

phase 4 - 17 ducts * 4 strands/duct = 68 strands

phase 3 & 5 - 13 ducts * 4 strands/duct = 52 strands each phase

Phase 1 : NB & SB (144 strands) * .75 (270)(.217) * $\frac{366.25}{1000}$ = 2317.5 MKFT

Phase 2 : (68 strands) * .75 (270)(.217) * $\frac{366.25}{1000}$ ≈ 1094.4 MKFT

Phase 3 & Phase 3A : (52) strands * .75 (270)(.217) * $\frac{366.25}{1000}$ ≈ 836.9 MKFT

Phase 4 & Phase 4A : (52) strands * .75 (270)(.217) * $\frac{366.25}{1000}$ ≈ 836.9 MKFT

TOTAL ≈ 5085.7 MKFT

USE 5086 MKFT

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 790 of

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

618-06034 Prestressed Concrete Slab (Depth 6" Through 13") SF

Phase 1&2 - Slab Area ≈ 414.66 SF

366.67/9.625 # of slabs = 38 × 2 = 76 EA

PHASE 1&2 AREA = 76 (414.66) = 31514. SF

Phase 4 - slab Area ≈ 384.99 SF

# of slabs = 38 EA

PHASE 4 AREA = 38(384.99) = 14630 SF

Phase 3 - slab Area ≈ 282.32 SF

# of slabs = 38 EA

PHASE 3 AREA = 38 (282.32) = 10729 SF

Phase 5 - slab Area ≈ 282.32 SF

# of slabs = 38 EA

PHASE 5 AREA = 38(282.32) = 10729 SF

TOTAL = 67602 SF

By: Date	Project no.	Project code (SA#):
Chk'd: Date	Structure no.	Sheet 791 of

CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION
DESIGN COMPUTATIONS (Grid)

INFO ONLY

618 - 00007 Grout (strand) (Duct) (CF)

$A_{duct} \approx .0128$ SF

$\ell = 366.5'$

phase 1 : 2 (18) (366.5)(.0128) ≈ 168.9 CF

phase 2 : 15 (366.5)(.0128) ≈ 70.4 CF

phase 3 : 13 (366.5) (.0128) ≈ 61.0 CF

phase 4 : 13 (366.5)(.0128) ≈ 61.0 CF

TOTAL ≈ 361.3 CF

USE 361 CF

FOR INFO ONLY
INCLUDE IN COST OF
POST - TENSIONING STRAND

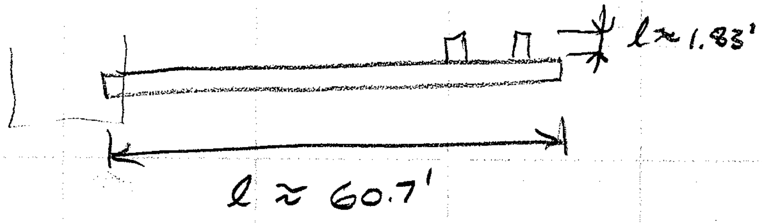
By: Date		Project no.	Project code (SA#):
Chk'd: Date		Structure no.	Sheet 792 of

CDOT Form #1034 3/02

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)

619 8" Plastic Pipe

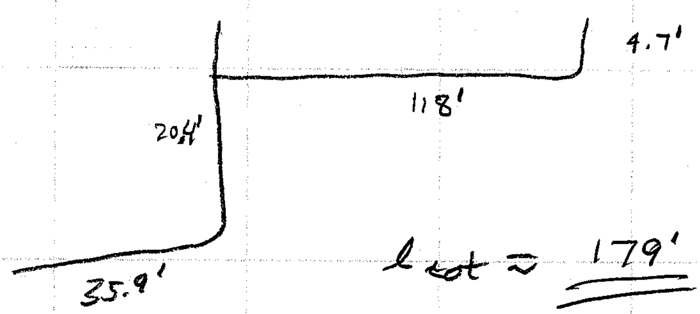


$l_{tot} = 60.7 + 1.83 + 1.83 \approx 64.4'$

2 systems use 65 ft per abutment

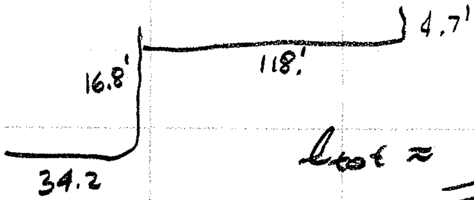
619 16" Plastic Pipe

near abut 1



$l_{tot} \approx 179'$

near abut 4



$l_{tot} \approx 174'$

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 793 of ___

	BR R600-297, SA 16212					
	Str No. WALL-F-16-EC through WALL-F-16-EG					
	By AJP					
Item No.	Description	Unit	WALL-F-16-EC	WALL-F-16-EE	WALL-F-16-EF	Total
202	Removal of Portions of Present Structure	EA		1	1	2
206	Structure Excavation	Cu Yd	4350		5570	9920
206	Structure Backfill (Class 1)	Cu Yd	6489		8336	14825
206	Mechanical Reinforcement of Soil	Cu Yd	3579		4592	8171
420	Geomembrane	Sq. Yd.	1570		2782	4352
504	Precast Panel Facing	Sq. Ft.	7627		12855	20482
601	Concrete (Class D)	Cu Yd	106.0		378.0	484.0
601	Structural Concrete Coating	Sq. Yd.	1121		2483	3604
602	Reinforcing Steel	Lbs	12829		50070	62899
606	Bridge Rail Type 7	LF	226		913	1139
613	1 Inch Electrical Conduit	LF	4		12	16
613	2 Inch Electrical Conduit	LF	452		~~1826~~ 2317	~~2278~~ 2769

Wall F-16-EF
Add 1-2"ø conduit to end of wall → 461'
Add 1-2"ø conduit to pull box & down wall → 30'

794

Project No. ***BR R600-297***
Subaccount No. ***16212***
By: ***A. Pott***

DESCRIPTION: ***WALL-F-16-EC***

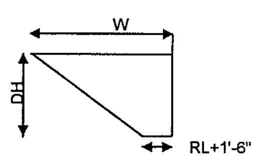
Panel or Block Thickness (D)= 0.625 FT
Height of Leveling Pad= 0.5 FT
Width of Leveling Block= 0.5 ft (either side of panel or block)

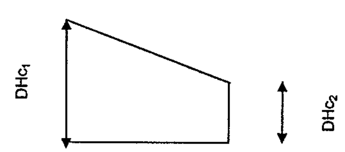
	Section Length	DHc_1	DHc_2	AVERAGE DH	Excavation Depth	RL	STRUCTURE EXCAVATION	STRUCTURE BACKFILL (CLASS 1&2)	MECHANICAL REINFORCEMENT OF SOIL	GEOMEMBRANE	PANEL FACING	STEP HEIGHT	CONCRETE CLASS D	STRUCTURAL CONCRETE COATING	REINFORCING	STRUCTURE BACKFILL (CLASS 2)
	FT	FT	FT	FT	FT	FT	CY	CY	CY	SY	SF	FT	CY	SY	LB	CY
slb	72.00	5.30	6.60	5.95	5.95	6.00	167.00	141.00	80.00	108.00	429.00	2.00	2.30	48.00	18.00	9.00
slb	8.00	7.60	7.80	7.70	7.70	7.70	30.00	26.00	16.00	16.00	62.00	4.00	0.40	7.00	20.00	1.00
slb	4.00	9.80	9.90	9.85	9.85	8.00	22.00	19.00	11.00	9.00	40.00	4.00	0.30	5.00	20.00	1.00
slb	8.00	10.90	12.00	11.45	11.45	8.02	52.00	47.00	25.00	19.00	92.00	4.00	0.40	11.00	20.00	1.00
slb	8.00	14.00	14.00	14.00	14.00	9.80	76.00	70.00	38.00	23.00	112.00	4.00	0.40	13.00	20.00	1.00
slb	4.00	16.00	16.10	16.05	16.05	11.24	50.00	46.00	26.00	13.00	65.00	4.00	0.30	8.00	20.00	1.00
slb	8.00	18.10	18.00	18.05	18.05	12.64	124.00	116.00	64.00	29.00	145.00	4.00	0.40	17.00	20.00	1.00
slb	4.00	20.00	20.00	20.00	20.00	14.00	76.00	71.00	40.00	16.00	80.00	4.00	0.30	9.00	20.00	1.00
slb	8.00	22.00	22.00	22.00	22.00	15.40	182.00	172.00	96.00	35.00	176.00	4.00	0.40	20.00	20.00	1.00
slb	4.00	24.00	24.00	24.00	24.00	16.80	108.00	103.00	58.00	19.00	96.00	4.00	0.30	11.00	20.00	1.00
slb	8.00	26.00	26.10	26.05	26.05	18.24	253.00	241.00	136.00	41.00	209.00	4.00	0.40	24.00	20.00	1.00
	5.63	26.10	26.40	26.25	26.25	18.38	181.00	182.00	101.00	29.00	148.00	4.00	0.30	17.00	20.00	1.00
	4.00	24.40	24.40	24.40	24.40	17.08	112.00	112.00	62.00	20.00	98.00	4.00	0.30	11.00	20.00	1.00
	8.00	22.40	22.50	22.45	22.45	15.72	190.00	191.00	105.00	36.00	180.00	4.00	0.40	20.00	20.00	1.00
	13.00	21.50	21.70	21.60	21.60	15.12	286.00	287.00	158.00	56.00	281.00	4.00	0.60	32.00	20.00	2.00
	161.30	12.10	12.50	12.30	12.30	8.61	1195.00	1215.00	633.00	402.00	1984.00	4.00	5.00	221.00	20.00	21.00
	26.10	22.90	22.85	22.88	22.88	16.01	641.00	644.00	355.00	118.00	598.00	4.00	1.00	67.00	20.00	4.00
	4.00	23.85	23.80	23.83	23.83	16.68	107.00	107.00	59.00	19.00	96.00	4.00	0.30	11.00	20.00	1.00
	4.00	24.80	24.80	24.80	24.80	17.36	115.00	116.00	64.00	20.00	100.00	2.00	0.20	12.00	18.00	1.00
	12.00	26.80	26.80	26.80	26.80	18.76	401.00	403.00	224.00	63.00	322.00	2.00	0.50	36.00	18.00	2.00
slb	56.00	26.80	25.10	25.95	25.95	18.17	1757.00	1671.00	941.00	284.00	1454.00	2.00	1.80	162.00	18.00	7.00
slb	4.00	23.10	22.90	23.00	23.00	16.10	100.00	94.00	53.00	19.00	92.00	2.00	0.20	11.00	18.00	1.00
slb	4.00	20.90	20.70	20.80	20.80	14.56	82.00	77.00	43.00	17.00	84.00	2.00	0.20	10.00	18.00	1.00
slb	4.00	18.70	18.60	18.65	18.65	13.06	66.00	62.00	35.00	15.00	75.00	2.00	0.20	9.00	18.00	1.00
slb	4.00	16.60	16.40	16.50	16.50	11.55	53.00	49.00	27.00	14.00	66.00	2.00	0.20	8.00	18.00	1.00
slb	4.00	14.40	14.20	14.30	14.30	10.01	40.00	37.00	20.00	12.00	58.00	2.00	0.20	7.00	18.00	1.00
slb	4.00	12.20	12.10	12.15	12.15	8.51	29.00	27.00	15.00	10.00	49.00	2.00	0.20	6.00	18.00	1.00
slb	4.00	10.10	9.90	10.00	10.00	8.00	22.00	20.00	11.00	9.00	40.00	2.00	0.20	5.00	18.00	1.00
slb	52.00	7.90	5.40	6.65	6.65	6.65	147.00	126.00	73.00	86.00	346.00	2.00	1.70	39.00	18.00	7.00
slb	8.00	6.40	6.00	6.20	6.20	6.20	20.00	17.00	10.00	13.00	50.00	2.00	0.40	6.00	18.00	1.00

Wall Length w/moment slab~	225.84	ft
Bridge Rail Type 7=	226.00	ft
Moment Slab Concrete Class D=	80.00	CY
Moment Slab Reinforcing=	12138.46	LB
Structural Concrete Coating=	258.00	SY
2 Inch Electrical Conduit	452.00	ft
pedestal concrete =	6.00	CY
pedestal reinforcing =	116.00	LB

Quantities reduced by 1' for slab

	STRUCTURE EXCAVATION	STRUCTURE BACKFILL (CLASS 1&2)	MECHANICAL REINFORCEMENT OF SOIL	GEOMEMBRANE	PANEL FACING	CONCRETE CLASS D	STRUCTURAL CONCRETE COATING	REINFORCING	STRUCTURE BACKFILL (CLASS 2)
TOTALS	***6684 CY***	***6489 CY***	***3579 CY***	***1570 SY***	***7627 SF***	***106 CY***	***1121 SY***	***12829 LB***	***75 CY***
	6684.00	6489.00	3579.00	1570.00	7627.00	106.00	1121.00	12829.00	75.00
	CLASS 2 ONLY =	***75 CY***							





SECTION

L

ELEVATION

AVERAGE DH = (DHc_1 + DHc_2)/2
W = RL+1.5+(DH x 1)

Project No.	BR R600-297	
Subaccount No.	16212	
By:	A. Pott	
DESCRIPTION:	WALL-F-16-EF	

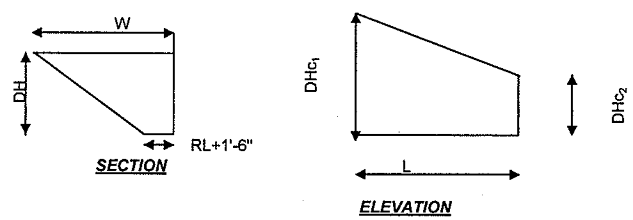
Panel or Block Thickness (D)=	0.625	FT
Height of Leveling Pad=	0.5	FT
Width of Leveling Block=	0.5	ft (either side of panel or block)

	Section Length	DHc_1	DHc_2	AVERAGE DH	Excavation Depth	RL	STRUCTURE EXCAVATION	STRUCTURE BACKFILL (CLASS 1&2)	MECHANICAL REINFORCEMENT OF SOIL	GEOMEMBRANE	PANEL FACING	STEP HEIGHT	CONCRETE CLASS D	STRUCTURAL CONCRETE COATING	REINFORCING	STRUCTURE BACKFILL (CLASS 2)
	FT	FT	FT	FT	FT	FT	CY	CY	CY	SY	SF	FT	CY	SY	LB	CY
slb	8.00	4.50	4.70	4.60	4.60	6.00	14.00	11.00	7.00	11.00	37.00	2.00	0.40	5.00	18.00	1.00
slb	180.00	4.70	7.54	6.12	6.12	6.12	436.00	370.00	209.00	275.00	1102.00	2.00	5.50	123.00	18.00	23.00
slb	20.00	8.54	8.65	8.60	8.60	8.00	88.00	78.00	46.00	41.00	172.00	2.00	0.70	20.00	18.00	3.00
slb	12.00	9.65	9.70	9.68	9.68	8.00	62.00	55.00	31.00	26.00	117.00	2.00	0.50	13.00	18.00	2.00
slb	4.00	10.70	10.70	10.70	10.70	8.00	24.00	22.00	12.00	9.00	43.00	2.00	0.20	5.00	18.00	1.00
slb	4.00	11.70	11.70	11.70	11.70	8.19	27.00	25.00	13.00	10.00	47.00	2.00	0.20	6.00	18.00	1.00
slb	4.00	12.70	12.80	12.75	12.75	8.93	32.00	29.00	16.00	11.00	51.00	2.00	0.20	6.00	18.00	1.00
slb	192.00	13.80	14.40	14.10	14.10	9.87	1847.00	1694.00	920.00	544.00	2708.00	2.00	5.90	301.00	18.00	24.00
slb	4.00	16.58	16.58	16.58	16.58	11.61	53.00	49.00	27.00	14.00	67.00	2.00	0.20	8.00	18.00	1.00
slb	8.00	18.58	18.58	18.58	18.58	13.01	132.00	123.00	68.00	30.00	149.00	2.00	0.40	17.00	18.00	1.00
slb	4.00	20.58	20.58	20.58	20.58	14.41	80.00	76.00	42.00	17.00	83.00	2.00	0.20	10.00	18.00	1.00
slb	8.00	22.58	22.58	22.58	22.58	15.81	192.00	181.00	102.00	36.00	181.00	2.00	0.40	21.00	18.00	1.00
slb	4.00	24.58	24.58	24.58	24.58	17.21	113.00	108.00	61.00	20.00	99.00	2.00	0.20	11.00	18.00	1.00
slb	8.00	26.58	27.00	26.79	26.79	18.75	268.00	255.00	144.00	42.00	215.00	2.00	0.40	24.00	18.00	1.00
slb	12.00	29.00	29.00	29.00	29.00	20.30	468.00	448.00	253.00	68.00	348.00	2.00	0.50	39.00	18.00	2.00
	10.15	29.00	29.00	29.00	29.00	20.30	396.00	398.00	222.00	58.00	295.00	2.00	0.40	33.00	18.00	2.00
	8.00	29.00	29.00	29.00	29.00	20.30	312.00	313.00	175.00	46.00	232.00	2.00	0.40	26.00	18.00	1.00
	4.00	27.00	26.90	26.95	26.95	18.87	136.00	136.00	76.00	22.00	108.00	2.00	0.20	12.00	18.00	1.00
	7.50	25.90	25.90	25.90	25.90	18.13	235.00	236.00	131.00	38.00	195.00	2.00	0.30	22.00	18.00	1.00
	4.50	16.00	16.00	16.00	16.00	11.20	56.00	56.00	30.00	15.00	72.00	2.00	0.20	8.00	18.00	1.00
	185.00	15.00	12.00	13.50	13.50	9.45	1638.00	1661.00	875.00	503.00	2498.00	2.00	5.70	278.00	18.00	24.00
	15.88	21.40	21.00	21.20	21.20	14.84	336.00	338.00	186.00	67.00	337.00	2.00	0.60	38.00	18.00	2.00
slb	12.00	21.00	20.80	20.90	20.90	14.63	247.00	233.00	130.00	50.00	251.00	2.00	0.50	28.00	18.00	2.00
slb	8.00	18.80	18.70	18.75	18.75	13.13	134.00	125.00	70.00	30.00	150.00	2.00	0.40	17.00	18.00	1.00
slb	8.00	16.70	16.60	16.65	16.65	11.66	106.00	99.00	55.00	27.00	134.00	2.00	0.40	15.00	18.00	1.00
slb	8.00	14.60	14.40	14.50	14.50	10.15	82.00	75.00	41.00	24.00	116.00	2.00	0.40	13.00	18.00	1.00
slb	12.00	12.40	12.10	12.25	12.25	8.58	89.00	80.00	43.00	30.00	147.00	2.00	0.50	17.00	18.00	2.00
slb	16.00	11.10	10.70	10.90	10.90	8.00	97.00	87.00	47.00	37.00	175.00	2.00	0.60	20.00	18.00	2.00
slb	12.00	9.70	9.40	9.55	9.55	8.00	61.00	54.00	31.00	26.00	115.00	2.00	0.50	13.00	18.00	2.00
slb	80.00	8.40	6.20	7.30	7.30	7.30	270.00	234.00	137.00	144.00	584.00	2.00	2.50	65.00	18.00	10.00
slb	64.00	7.20	5.20	6.20	6.20	6.20	159.00	135.00	77.00	99.00	397.00	2.00	2.00	45.00	18.00	8.00
slb	32.00	6.20	5.40	5.80	5.80	6.00	72.00	61.00	35.00	48.00	186.00	2.00	1.10	21.00	18.00	4.00
slb	24.00	6.40	5.70	6.05	6.05	6.05	57.00	49.00	28.00	37.00	146.00	2.00	0.80	17.00	18.00	3.00
slb	24.00	6.70	5.90	6.30	6.30	6.30	62.00	53.00	30.00	38.00	152.00	2.00	0.80	17.00	18.00	3.00
slb	20.00	6.90	6.30	6.60	6.60	6.60	56.00	48.00	28.00	33.00	132.00	2.00	0.70	15.00	18.00	3.00
slb	64.00	7.30	5.50	6.40	6.40	6.40	169.00	144.00	82.00	102.00	410.00	2.00	2.00	46.00	18.00	8.00
slb	44.00	6.50	5.40	5.95	5.95	6.00	102.00	86.00	49.00	66.00	262.00	2.00	1.40	30.00	18.00	6.00
slb	60.00	6.40	5.00	5.70	5.70	6.00	132.00	111.00	63.00	88.00	342.00	2.00	1.90	38.00	18.00	8.00
						TOTALS	8840 CY	8336 CY	4592 CY	2782 SY	12855 SF		378 CY	2483 SY	50070 LB	160 CY
							8840.00	8336.00	4592.00	2782.00	12855.00		378.00	2483.00	50070.00	160.00
						CLASS 2 ONLY =		160 CY								

Quantities reduced by 1' for slab

Wall Length w/moment slab~	912.72	ft
Bridge Rail Type 7=	913.00	ft
Moment Slab Concrete Class D=	320.00	CY
Moment Slab Reinforcing=	49037.23	LB
Structural Concrete Coating=	1040.00	SY
2 Inch Electrical Conduit	1826.00	ft
pedestal concrete =	17.00	CY
pedestal reinforcing =	348.00	LB

796



AVERAGE DH = $(DHc_1 + DHc_2)/2$
W = RL+1.5+(DH x 1)

Triangle Volume Report

Report Created: 12/27/2010
Time: 8:51am

Mode: Entire Surface
Input Grid Factor: 1.000000

Original Surface: 16212SURRURVSurface01
Design Surface: SouthWallExcav

Cut Factor:	1.00
Fill Factor:	1.00
Cut:	117243.8 cu ft
Fill:	8.6 cu ft
Net:	117235.2 cu ft
Cut:	4342.4 cu yd
Fill:	0.3 cu yd
Net:	4342.0 cu yd

Use 4350 CY

797

Triangle Volume Report

Report Created: 12/27/2010
Time: 9:40am

Mode: Entire Surface
Input Grid Factor: 1.000000

Original Surface:	**16212SURRURVSurface01**
Design Surface:	**NorthWallExcav**
Cut Factor:	1.00
Fill Factor:	1.00
Cut:	150252.6 cu ft
Fill:	11.1 cu ft
Net:	150241.5 cu ft
Cut:	5564.9 cu yd
Fill:	0.4 cu yd
Net:	5564.5 cu yd

Use 5570 CY

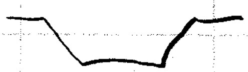
798

Light Pole Pedestals

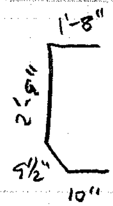
601 Concrete ≈ 2.7 CY/ pedestal

WALL-F-16-EC → 2EA ⇒ 5.4 CY use 6
WALL-F-16-EF → 6 EA ⇒ 16.2 CY use 17

602-00020 Reinforcing (Epoxy Coated)

6-#4  $\ell \approx 7.42'$ wt = 6*7.42*.668 = 29.8 lbs

2-#4 x 2'-8" wt = 2*2.67*.668 = 3.6 lbs

6-#4 

$\ell = 6.04'$ wt = 6*6.04*.668 = 24.2 lbs

total ≈ 57.6 lbs

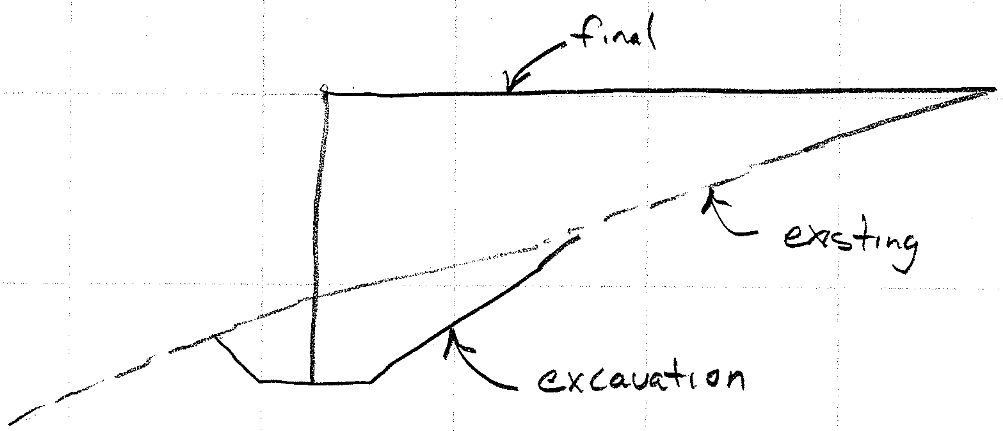
Use 58 lbs/pedestal

WALL-F-16-EC → 2EA ⇒ 116 lbs
WALL-F-16-EF → 6 EA ⇒ 348 lbs

By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 799 of

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS (Grid)



By:	Date	Project no.	Project code (SA#):
Chk'd:	Date	Structure no.	Sheet 800 of ____

CDOT Form #1034 3/02

Triangle Volume Report

Report Created: 12/22/2010
Time: 8:47am

Mode: Entire Surface
Input Grid Factor: 1.000000

Original Surface: **SouthWallExcav**
Design Surface: **Southwest Grading**
Cut Factor: 1.00
Fill Factor: 1.00

Cut: 461.4 cu ft
Fill: 698.5 cu ft
Net: -237.1 cu ft

Cut: 17.1 cu yd
Fill: 25.9 cu yd
Net: -8.8 cu yd

801

Triangle Volume Report

Report Created: 12/22/2010
Time: 8:59am

Mode: Selected Shapes
Input Grid Factor: 1.000000

Original Surface: 16212SURRURVSurface01
Design Surface: Southwest Grading

Cut Factor:	1.00
Fill Factor:	1.00
Cut:	117.7 cu ft
Fill:	0.0 cu ft
Net:	117.7 cu ft
Cut:	4.4 cu yd
Fill:	0.0 cu yd
Net:	4.4 cu yd

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COMPARI[illegible]ON

Quantities

1/11/2011

Item No.	Description	Unit	Super-Structure	Abut. 1	Pier 2	Pier 3	Abut. 4	Appr. Slabs	Total	Andy	% difference		
	BR R600-297												
	Str No. F-16-XB												
	By JOE VIRDI												
202	Removal of Slope and Ditch Paving	SY		1068			772		1,839	1838	-0.07%		
202	Removal of Bridge	Each	1						1	1	0.00%		
203	Embankment Material (Complete in Place)	Cu Yd		31			29		60	58	[redacted]	okay	
206	Structure Excavation	Cu Yd		135	225	225	94		679	660	-2.87%		
206	Structure Backfill (Flow-Fill)	Cu Yd		2326			2326		4,653	4500	[redacted]	okay	
206	Structure Backfill (Class 1)	Cu Yd			13	13			25	20	[redacted]	okay	
403	Stone Matrix Asphalt (Fibers) (Asphalt)	Ton	1350					139	1,489	1487	-0.10%		
503	Drilled Caisson (30 Inch)	Lin Ft		815			773		1,588	1590	0.13%		
503	Drilled Caisson (54 Inch)	Lin Ft			405	383	0		788	800	1.49%		
507	Concrete Slope and Ditch Paving	Cu Yd		96			66		162	162	0.01%		
509	Structural Steel (Galvanized)	Lbs		5236	6256	6256	5236		22,985	22960	-0.11%		
509	Bearing Device (Type I)	Each	0		16	16		0	32	32	0.00%		
513	Bridge Drain (Special)	Each		2			2		4	4	0.00%		
515	Waterproofing (Membrane)	Sq Yd	7893					812	8,705	8788	0.94%		
518	Bridge Expansion Device (0 to 4 Inches)	Lin Ft						469	469	468	-0.19%		
601	Concrete Class D (Bridge)	Cu Yd	421.2	119.5	462.1	462.1	120.1	388.9	1,974	1995	1.06%		
601	Grout (Bridge) (Special)	Cu Ft	11553.9						11,554	11925	[redacted]		
601	Structural Concrete Coating	Sq Yd	8353.6	93.7	923.0	923.0	93.7	90.8	10,478	10785	2.85%		
602	Reinforcing Steel	Lbs	0		95,945	95,945	0		191,891	186500	-2.89%		
			[redacted]	[redacted]			[redacted]	[redacted]					
602	Reinforcing Steel (Epoxy Coated)	Lbs	56,162 62,120	12,552 13,080			12,552 13,080	69,098 71,625	150,365	159905	[redacted]		$ < 10k okay
606	Bridge Rail Type 7	Lin Ft	822						822	822	0.00%		
606	Guard Rail Type 7 CC (Special)	Lin Ft	411						411	411	0.00%		
607	Fence Chain Link (Special) (60 Inch)	Lin Ft	400						400	400	0.00%		
613	1 1/2" Electrical Conduit	Lin Ft	20						20	20	0.00%		
613	2" Electrical Conduit	Lin Ft	1,744						1,744	1933	[redacted]		extra for weather sensor & girders
613	Luminaire High Pressure Sodium (150 Watt)	Each	12						12	12	0.00%		
614	Weather Sensor	LS	1						1	1	0.00%		
618	Prestressing Steel Strand	MKFT	5,088						5,088	5086	-0.03%		
618	Prestressed Concrete Slab (Depth 6" Through 13")	Sq Ft	67,976						67,976	67602	-0.55%		
618	Prestressed Concrete U Girder (U72) (Pre-Tensioned)	Lin Ft	2,920						2,920	2920	0.00%		
619	8" Plastic Pipe	Lin Ft	126						126	129	2.33%		
619	16" Plastic Pipe	Lin Ft	346						346	353	1.98%		

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CHECK Quantities

1/12/2011

Item No.	Description	Unit	Super-Structure	Abut. 1	Pier 2	Pier 3	Abut. 4	Appr. Slabs	Total
	BR R600-297								
	Str No. F-16-XB								
	By JOE VIRDI								
202	Removal of Slope and Ditch Paving	SY		1068			772		1,839
202	Removal of Bridge	Each	1						1
203	Embankment Material (Complete in Place)	Cu Yd		31			29		60
206	Structure Excavation	Cu Yd		135	225	225	94		679
206	Structure Backfill (Flow-Fill)	Cu Yd		2326			2326		4,653
206	Structure Backfill (Class 1)	Cu Yd			13	13			25
403	Stone Matrix Asphalt (Fibers) (Asphalt)	Ton	1350					139	1,489
503	Drilled Caisson (30 Inch)	Lin Ft		815			773		1,588
503	Drilled Caisson (54 Inch)	Lin Ft			405	383	0		788
507	Concrete Slope and Ditch Paving	Cu Yd		96			66		162
509	Structural Steel (Galvanized)	Lbs		5236	6256	6256	5236		22,985
509	Bearing Device (Type I)	Each	0		16	16		0	32
513	Bridge Drain (Special)	Each		2			2		4
515	Waterproofing (Membrane)	Sq Yd	7893					708	8,600
518	Bridge Expansion Device (0 to 4 Inches)	Lin Ft						469	469
601	Concrete Class D (Bridge)	Cu Yd	421.2	119.5	462.1	462.1	120.1	388.9	1,974
601	Grout (Bridge) (Special)	Cu Ft	11553.9						11,554
601	Structural Concrete Coating	Sq Yd	8353.6	93.7	923.0	923.0	93.7	90.8	10,478
602	Reinforcing Steel	Lbs	0		95,945	95,945	0		191,891
602	Reinforcing Steel (Epoxy Coated)	Lbs	56,162	12,552			12,552	69,098	150,365
606	Bridge Rail Type 7	Lin Ft	822						822
606	Guard Rail Type 7 CC (Special)	Lin Ft	411						411
607	Fence Chain Link (Special) (60 Inch)	Lin Ft	400						400
613	1 1/2" Electrical Conduit	Lin Ft	20						20
613	2" Electrical Conduit	Lin Ft	1,744						1,744
613	Luminaire High Pressure Sodium (150 Watt)	Each	12						12
614	Weather Sensor	LS	1						1
618	Prestressing Steel Strand	MKFT	5,088						5,088

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618	Prestressed Concrete Slab (Depth 6" Through 13")	Sq Ft	67,976						67,976
618	Prestressed Concrete U Girder (U72)' (Pre-Tensioned)	Lin Ft	2,920						2,920
619	8" Plastic Pipe	Lin Ft	126						126
619	16" Plastic Pipe	Lin Ft	346						346

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BR R600-297
Str No. F-16-XB
By JOE VIRDI

Item No. 202
Removal of Slope and Ditch Paving
Abutment 1

Height	No.	Width	Length	Sq. Ft.	Sq Yd
Slope 9609.0000	1.0	1.0000	1.0000	9609.00	1067.67
			Total		**1068 Sq. Yd.**

Abutment 4

Height	No.	Width	Length	Sq. Ft.	Sq Yd
Slope 6944.0000	1.0	1.0000	1.0000	6944.0	771.56
			Total		**772 Sq. Yd.**

Item No. 202
Embankment Material (Complete in Place)
Abutment 1

Height	No.	Width	Length	Sq. Ft.	Sq Yd
Abut.1 0.4200	1.0	4.3300	153.0000	278.25	30.92
			Total		**31 Sq. Yd.**

Embankment Material (Complete in Place)
Abutment 4

Height	No.	Width	Length	Sq. Ft.	Sq Yd
Abut.1 0.4000	1.0	4.3300	153.0000	265.00	29.44
			Total		**29 Sq. Yd.**

Item No. 206
Structure Excavation
Abutment 1

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd
Abutment 5.5000	1.0	4.3330	153.0000	3646.22	135.05
					135.05 Cu Yd

Abutment 4

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd
Area = 4.5000	1.0	4.3330	130.0000	2534.81	93.88
					93.88 Cu Yd

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Item No. 206
Structure Excavation
Pier 2

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier						
13.5000	1.0	7.5000	7.5000	759.38	28.13	
					225.00	**Cu Yd**

Item No. 206
Structure Excavation
Pier 3

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier						
13.5000	1.0	7.5000	7.5000	759.38	28.13	
					225.00	**Cu Yd**

Item No. 206
Structure Backfill (Class 1)
Pier 2

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier						
0.7500	1.0	7.5000	7.5000	42.19	1.56	
					12.50	**Cu Yd**

Item No. 206
Structure Backfill (Class 1)
Pier 3

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier						
0.7500	1.0	7.5000	7.5000	42.19	1.56	
					12.50	**Cu Yd**

Item No. 206
Structure Backfill (Flow-Fill)
Abutment 1

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Bk Abut. Area =						
348.7000	1	1.0000	237.0000	82641.90	3060.81	
Deduct						
-83.6700	1	1.0000	237.0000	-19829.79	-734.44	
				62812.11	**2326.37**	**Cu Yd**

Structure Backfill (Flow-Fill)
Abutment 4

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Bk Abut.						
Area =						
348.7000	1	1.0000	237.0000	82641.90	3060.81	
Deduct						
-83.6700	1	1.0000	237.0000	-19829.79	-734.44	
				62812.11	**2326.37**	**Cu Yd**

BR R600-297
Str No. F-16-XB
By JOE VIRDI

Item No. 403
Stone Matrix Asphalt
Superstructure

Width	Length	Thickness Inch			
71036.00	1.00	3		213108	Sq. Ft.
			/	9	
				23679	Sq. Yd.
			*	114	
				2699368	Lbs.
			/	2000	
				1350	***Tons***

Approach Slabs:

Approach Slab Abut.1 and 2

Width	No.	Length	Thickness Inch			
7309.00	1	1.00	3		21927	Sq. Ft.
				/	9	
					2436	Sq. Yd.
				*	114	
					277742	Lbs.
				/	2000	
					139	***Tons***

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Item No. 503
Drilled Caisson 30 Inch

	Top Elev	Tip Elev	Length	No. of Caisson	Total Lin Ft
Abut. 1	5197.0	5148.0	49.0	13	637.0
Abut. 1	5192.5	5148.0	44.5	4	178.0

Total Lin. Ft. Abut. 1 = 815.00

	Top Elev	Tip Elev	Length	No. of Caisson	Total Lin Ft
Abut. 4	5196.0	5150.0	46.0	14	644.0
Abut. 4	5192.0	5150.0	42.0	2	84.0
Abut. 4	5195.0	5150.0	45.0	1	45.0

Total Lin. Ft. Abut. 4 = 773.00

Item No. 503
Drilled Caisson 54 Inch

	Top Elev	Tip Elev	Length	No. of Caisson	Total Lin Ft
Pier 2	5182.2	5131.5	50.7	8	405.2

Total Lin. Ft. Abut. 1 = 405.24

	Top Elev	Tip Elev	Length	No. of Caisson	Total Lin Ft
Pier 3	5182.4	5134.5	47.9	8	382.9

Total Lin. Ft. Abut. 4 = 382.88

809

Item No. 503
Structural Steel (Galvanized)
Pier 2

Width	Thick	Length	Wt/Inch in Lbs	No.	Total Weight	Total Weight Lb	
Plate							
21.00	1.5"	21.0000	9	32	5994.2	**5994.2**	**Lbs.**
Stud in In.							
4.00	1	4.0000	1	128	93.4	**93.4**	**.**
Bar							
1.00	1	57.5000	0	8	102.4	**102.4**	**.**
Stud in In.							
4.00	0	4.0000	0	64	9.6	**9.6**	**.**
Plate							
5.00	1	5.0000	1	16	56.7	**56.7**	
					Total	**6256.29**	**Lbs.**

Structural Steel (Galvanized)
Pier 3

Width	Thick	Length	Wt/Inch in Lbs	No.	Total Weight	Total Weight Lb	
Plate							
21.00	1.5"	21.0000	9	32	5994.2	**5994.2**	**Lbs.**
Stud in In.							
4.00	1	4.0000	1	128	93.4	**93.4**	**.**
Bar							
1.00	1	57.5000	0	8	102.4	**102.4**	**.**
Stud in In.							
4.00	0	4.0000	0	64	9.6	**9.6**	**.**
Plate							
5.00	1	5.0000	1	16	56.7	**56.7**	
					Total	**6256.29**	**Lbs.**

810

Structural Steel (Galvanized)
Abut. 1

Width	Thick	Length	Wt/Ft in Lbs	No.	Total Weight	Total Weight Lb	
W10X88 88.00	1	3.5000	88	17	5236.0	**5236.0**	**Lbs.**
					Total	**5236.00**	**Lbs.**

Structural Steel (Galvanized)
Abut. 4

Width	Thick	Length	Wt/Ft in Lbs	No.	Total Weight	Total Weight Lb	
W10X88 88.00	1	3.5000	88	17	5236.0	**5236.0**	**Lbs.**
					Total	**5236.00**	**Lbs.**

Item No. 507
Concrete Slope and Ditch Paving
Abut.1

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
5820.0000	1	0.3330	1.0000	1938.06	71.78	
Toe Walls 2.0000	2	0.6667	250.0000	666.70	24.69	
			Total =		**96.47**	**Cu. Yd.**

Abut.4

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
3310.0000	1	0.3330	1.0000	1102.23	40.82	
Toe Walls 2.0000	2	0.6667	250.0000	666.70	24.69	
			Total =		**65.52**	**Cu. Yd.**

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BR R600-297
Str No. F-16-XB
By JOE VIRDI

Item No. 515
Waterproofing (Membrane)
Superstructure

Width	Length	Total Sq. Ft.	Total Sq. Yd.
71036.00	1.0000	71036.0	**7892.9 Sq. Yd.**

Total Waterproofing Superstructure = 7892.89 Sq. Yd.

Approach Slabs

Width	No.	Length	Total Sq. Ft.	Total Sq. Yd.
App Slab 3184.00		2.0000	6368.0	**707.6 Sq. Yd.**

Total Waterproofing at App. Slabs = 707.56 Sq. Yd.

Item No. 518
Bridge Expansion Device (0-4 Inch)

Width	Length	Total Lin Ft	Total Lin Ft
1.00	234.4500	468.9	**468.9 Lin Ft**

Total Bridge Expansion Device 468.90 Lin Ft

812

Item No. 601
Structural Concrete Coating
Superstructure

Width	No.	Length	Total Sq. Ft.	Total Sq. Yd.
Rail on Dk.				
6.80	2	371.0000	5044.2	560.5
Sb overhang				
6.13	2	371.0000	4544.8	505.0
Girder U				
21.68	8	73.0000	12661.1	1406.8
Girder U				
21.68	16	23.0000	7978.2	886.5
Girder U				
21.68	8	125.0000	21680.0	2408.9
Girder U				
21.68	8	121.0000	20986.2	2331.8
Abut. 1				
143.00	8	1.0000	1144.0	127.1
Abut. 4				
143.00	8	1.0000	1144.0	127.1

Total Concrete Coating -Superstructure = 8353.62 Sq. Yd.

Structural Concrete Coating
Abut. 1

Width	No.	Length	Total Sq. Ft.	Total Sq. Yd.
Abut. Front				
3.50	1	241.0000	843.5	93.7

Total Concrete Coating = 93.72 Sq. Yd.

813

Structural Concrete Coating
Abut. 4

Abut. Front

3.50	1	241.0000	843.5	93.7
Side				

Total Waterproofing Superstructure = 93.72 Sq. Yd.

Structural Concrete Coating
Pier 2

Column
Cornice

35.82	16	1.0000	573.1	63.7
1.17	16	28.5000	532.0	59.1
292.30	16	1.0000	4676.8	519.6
101.25	16	1.0000	1620.0	180.0
14.14	8	8.0000	905.0	100.6

Total Structural Concrete Coating = 922.99 Sq. Yd.

814

Structural Concrete Coating
Pier 3

Column Cornice				
35.82	16	1.0000	573.1	63.7
1.17	16	28.5000	532.0	59.1
292.30	16	1.0000	4676.8	519.6
101.25	16	1.0000	1620.0	180.0
14.14	8	8.0000	905.0	100.6

Total Structural Concrete Coating = 922.99 Sq. Yd.

Structural Concrete Coating
Approach Slabs

Rail				
6.80	4	20.5000	557.4	61.9
0.67	4	20.5000	54.9	6.1
Lip				
2.50	4	20.5000	205.0	22.8

Total Structural Concrete Coating = 90.82 Sq. Yd.

815

Item No. 606
Bridge Rail Type 7
Superstructure

Height	No.	Total
North Side		
411.00	1	411.0 Lin. Ft.
South side		
411.00	1	411.0 Lin. Ft.
Total Bridge Rail Type Type 7		**822 Lin. Ft.**

Item No. 606
Guard Rail Type 7 CC (Special)
Superstructure

Height	No.	Total
Median		
411.00	1	411.0 Lin. Ft.
Total Guard Rail Type 7 CC (Special)		**411 Lin. Ft.**

Item No. 607
Fence Chain Link (Special) (60 Inch)
Superstructure

Location	No.	Total
North+South		
400.00	1	400.0 Lin. Ft.
Total Fence Chain Link (Special)		**400 Lin. Ft.**

Item No. 613
1 1/2 Inch Electrical Conduit
Superstructure

Location	No.	Total
North Abut.		
10.00	2	20.0 Lin. Ft.
Total 1 1/2 Inch Electrical Conduit		**20 Lin. Ft.**

Item No. 613
2 Inch Electrical Conduit
Superstructure

Location	No.	Total
N&S of Deck		
50.00	2	100.0 Lin. Ft.
411.00	4	1644.0 Lin. Ft.
Total 2 Inch Electrical Conduit		**1744 Lin. Ft.**

Item No. 613
Luminaire High Pressure Sodium (150 Watt)
Superstructure

Location	No.	Total
12.00	1	12.0 Lin. Ft.
Luminaire High Pressure Sodium (150 Watt)		**12 Lin. Ft.**

817

Item No. 618
Prestressing Steel Wire or Strand

Percent	Sq. Inch	Str. Length	Kip/Strand	Total	Total MKFT	
0.7500	17.150	366.25	270.0000	1271940.47	5087.76	**MKFT**
0	0					
			Total =		**5087.76**	**MKFT**

Item No. 618
Precast Concrete U Girder (U72) (Pre-Tensioned)

Span	Width	No.	Length	Total	
1.00	1	8	76	608.0	Lin. Ft.
2.00	1	8	127	1016.0	Lin. Ft.
3.00	1	8	124	992.0	Lin. Ft.
Bet.Girders	1	16	19	304.0	Lin. Ft.
	Total =			**2920**	**Lin. Ft.**

Item No. 619
8 Inch Plastic Pipe

Superstr.	Width	No.	Length	Total	
Abut. 1	1	1	63	63.0	Lin. Ft.
Abut. 4	1	1	63	63.0	Lin. Ft.
	Total =			**126**	**Lin. Ft.**

818

Item No. 619
16 Inch Plastic Pipe

Superstr.	Width	No.	Length	Total	
Abut. 1	1	1	174	174.0	Lin. Ft.
Abut. 4	1	1	172	172.0	Lin. Ft.
Total =				**346**	**Lin. Ft.**

BR R600-297
Str No. F-16-XB
By JOE VIRDI

Item No. 601
Concrete Class D (Bridge)
Superstructure

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Closure P.						
0.67	4.0	2.8333	371.0000	2803.22	103.82	
			Total =		103.82	Cu. Yd.

Item No. 601
Concrete Class D (Bridge)
Abutment 1 (Superstructure)

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Abutment						
7.0833	1	2.5000	237.6250	4207.92	155.85	
Void for Approach Slab Notch						
1.0000	1	0.5417	237.6250	-128.71	-4.77	
Void Girder Area =						
12.5800	8		2.7500	-276.76	-10.25	
			Total =		140.83	Cu. Yd.

Item No. 601
Concrete Class D (Bridge)
Abutment 4 (Superstructure)

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Abutment						
7.1870	1	2.5000	237.6250	4269.53	158.13	
Void for Approach Slab Notch						
1.0000	1	0.5417	237.6250	-128.71	-4.77	
Void Girder Area =						
12.5800	8		2.7500	-276.76	-10.25	
			Total =		143.11	Cu. Yd.

Item No. 601
Concrete Class D (Bridge)
Pier 2 (Superstructure)

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Bet. Gidrs						
28.2200	8	1.0000	2.0000	451.52	16.72	
			Total =		16.72	Cu. Yd.

Item No. 601
Concrete Class D (Bridge)
Pier 3 (Superstructure)

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Bet. Gidrs						
28.2200	8	1.0000	2.0000	451.52	16.72	
			Total =		16.72	Cu. Yd.
		Grand Total			**421.21**	**Cu. Yd.**

Item No. 601
Concrete Class D (Bridge)
Abutment 1

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Abutment						
3.6700	1	2.5000	237.6250	2180.21	80.75	
Column						
4.9080	13.0	1.0000	11.6000	740.13	27.41	
4.9080	4.0	1.0000	15.5700	305.67	11.32	
			Total =		119.48	Cu. Yd.

Abutment 4

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Abutment						
3.6700	1	2.5000	237.6250	2180.21	80.75	
Column						
4.9080	14.0	1.0000	11.9900	823.86	30.51	
4.9080	3.0	1.0000	16.2000	238.53	8.83	
			Total =		120.10	Cu. Yd.

Item No. 601
Concrete Class D (Bridge)
Pier 2

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier Area						
292.20	8.0	4.5000	1.0000	10519.20	389.60	
Overhang						
35.80	8.0	6.8333	1.0000	1957.06	72.48	
			Total =		462.08	Cu. Yd.

Pier 3

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
Pier Area						
292.20	8.0	4.5000	1.0000	10519.20	389.60	
Overhang						
35.80	8.0	6.8333	1.0000	1957.06	72.48	
			Total =		462.08	Cu. Yd.

821

BR R600-297
Str No. F-16-XB
By JOE VIRDI

Item No. 601
Concrete Class D (Bridge)
Abutment 1 Approach Slab

Height	No.	Width	Length	Total Cu Ft	Total Cu Yd	
4300.0000	2	1.0000	1.0000	8600.00	318.52	
Sleeper						
1.0000	2	4.0000	237.6200	1900.96	70.41	
Stem						
1.0000	2.0	1.2500	237.6200	594.05	22.00	
			Total =		388.92	Cu. Yd.
Grand Total - 2 App. Slabs					**388.92**	**Cu. Yd.**

Item No. 601
Grout (Bridge) (Special)
Superstructure

Height	No.	Width	Length	Total Cu Ft	Total Cu Ft	
0.5000	37	0.7500	365.0000	5064.38	5064.38	
Grdr Ovhg						
0.4167	16	2.6667	365.0000	6489.49	6489.49	
Top ofGrdr						
0.4167	16	1.0000	6.0000	40.00	40.00	
			Total =		**11553.86**	**Cu. Ft.**

822

Item No. 618
Prestressed Concrete Slab (Depth 6" to 13")
Superstructure

Height	Length	Width	No	Total Sq Ft	Total Sq Ft	
Phase 1 Panel	52.0	7.9220	38.0000	15644.84	15644.84	
Phase 2 Panel	52.0	7.9220	38.0000	15644.84	15644.84	
Phase 3 Panel	39.4	7.9220	38.0000	11860.82	11860.82	
Phase 4 Panel	48.3	7.9220	38.0000	14524.99	14524.99	
Phase 5 Panel	39.4	7.9220	38.0000	11860.82	11860.82	
Block Out	-1.2	10.0000	48.0000	-576.00	-576.00	
Block Out	-1.2	10.0000	38.0000	-456.00	-456.00	
Block Out	-1.2	10.0000	44.0000	-528.00	-528.00	
			Total =		**67976.31**	**Sq. Ft.**

Project No. BR R600-297													
Str. No. F-16-XB													
Prepared by Joe Virdi													
					4	5	6	7	8	9	10	11	
BAR LIST - SUPERSTRUCTURE					0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	
Slab:----													
				No.									
		Bar	No.	Req'd				Dimensions		Epoxy Total	Black Total	Epoxy Total	Black Total
Description		Size	Req'd	B.E.I	Length	Type	A	B	C	Length	Length	Weight	Weight
Closure pour (long.)	E	5	32		359'-7"	Str.				11,506.67	-	12,001	-
Closure pour (Tran)	E	4	54		237'-0"	Str.				12,798.00	-	8,549	-
Betw. Gider ends Stirrup	E	5	64		9'-5"	Str.				602.67	-	629	-
Betw. Gider ends Stirrup	E	5	64		18'-11"	Str.				1,210.67	-	1,263	-
Betw. Gider ends Str.	E	5	64		5'-8"	Str.				362.67	-	378	-
Betw. Gider ends Str.	E	5	912		3'-3"	Str.				2,964.00	-	3,091	-
Stirrup for Bridge Rail	E	4	24		4'-9"	Str.				114.00	-	76	-
Stirrup for Bridge Rail	E	4	24		2'-9"	Str.				66.00	-	44	-
Abutment 1:----													
For App Slab	E	5	238		3'-6"	Str.				833.00	-	869	-
Thru Gider Webs Str.	E	7	5		255'-3"	Str.				1,276.25	-	2,609	-
At Top in Notch Str.	E	5	2		247'-0"	Str.				494.00	-	515	-
At Top in Notch Stirrup	E	5	690		2'-9"	Str.				1,897.50	-	1,979	-
Abut. Top	E	11	2		283'-8"	Str.				567.33	-	3,014	-
Abut. Back	E	5	6		247'-0"	Str.				1,482.00	-	1,546	-
Abut. Stirrup	E	5	236		8'-4"	Str.				1,966.67	-	2,051	-
Abut. Stirrup	E	5	236		10'-0"	Str.				2,360.00	-	2,461	-
Abutment 4:----													
For App Slab	E	5	238		3'-6"	Str.				833.00	-	869	-
Thru Gider Webs Str.	E	7	5		255'-3"	Str.				1,276.25	-	2,609	-
At Top in Notch Str.	E	5	2		247'-0"	Str.				494.00	-	515	-
At Top in Notch Stirrup	E	5	690		2'-9"	Str.				1,897.50	-	1,979	-
Abut. Top	E	11	2		283'-8"	Str.				567.33	-	3,014	-
Abut. Back	E	5	6		247'-0"	Str.				1,482.00	-	1,546	-
Abut. Stirrup	E	5	236		8'-5"	Str.				1,986.33	-	2,072	-
Abut. Stirrup	E	5	236		10'-1"	Str.				2,379.67	-	2,482	-

828

824-827 not used

BAR SUMMARY - SUPERSTRUCTURE

EPOXY COATED					NON-EPOXY COATED			
Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight		Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
12,978.00	#4	0.668	8,669		-	#4	0.668	-
34,752.33	#5	1.043	36,247		-	#5	1.043	-
-	#6	1.502	-		-	#6	1.502	-
2,552.50	#7	2.044	5,217		-	#7	2.044	-
-	#8	2.670	-		-	#8	2.670	-
-	#9	3.400	-		-	#9	3.400	-
-	#10	4.303	-		-	#10	4.303	-
1,134.67	#11	5.313	6,028		-	#11	5.313	-
		Total =	56,162				Total =	-

Project No. BR R600-297
Str. No. F-16-XB
Prepared by Joe Virdi

BAR LIST - ABUTMENT 1					4	5	6	7	8	9	10	11	
					0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	
Description		Bar Size	No. Req'd	No. Req'd B.E.I	Length	Type	Dimensions A	Dimensions B	Dimensions C	Epoxy Total Length	Black Total Length	Epoxy Total Weight	Black Total Weight
Abut. (Trans)	E	11	4		283'-8"					1,134.67	-	6,028	-
Abut. (Trans)	E	5	6		247'-0"					1,482.00	-	1,546	-
Abut. (Vert.)	E	6	16		4'-0"					64.00	-	96	-
Abut. Stirrups	E	5	236		14'-1"					3,323.67	-	3,467	-
Abut. Stirrups	E	5	236		5'-9"					1,357.00	-	1,415	-

BAR SUMMARY - ABUTMENT 1

EPOXY COATED Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight	NON-EPOXY COATED Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
-	#4	0.668	-	-	#4	0.668	-
6,162.67	#5	1.043	6,428	-	#5	1.043	-
64.00	#6	1.502	96	-	#6	1.502	-
-	#7	2.044	-	-	#7	2.044	-
-	#8	2.670	-	-	#8	2.670	-
-	#9	3.400	-	-	#9	3.400	-
-	#10	4.303	-	-	#10	4.303	-
1,134.67	#11	5.313	6,028	-	#11	5.313	-
		Total =	12,552			Total =	-

BAR LIST - ABUTMENT 4					4	5	6	7	8	9	10	11	
					0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	
Description		Bar Size	No. Req'd	No. Req'd B.E.I	Length	Type	Dimensions A	Dimensions B	Dimensions C	Epoxy Total Length	Black Total Length	Epoxy Total Weight	Black Total Weight
Abut. (Trans)	E	11	4		283'-8"					1,134.67	-	6,028	-
Abut. (Trans)	E	5	6		247'-0"					1,482.00	-	1,546	-
Abut. (Vert.)	E	6	16		4'-0"					64.00	-	96	-
Abut. Stirrups	E	5	236		14'-1"					3,323.67	-	3,467	-
Abut. Stirrups	E	5	236		5'-9"					1,357.00	-	1,415	-

BAR SUMMARY - ABUTMENT 4

EPOXY COATED					NON-EPOXY COATED			
Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight		Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
-	#4	0.668	-		-	#4	0.668	-
6,162.67	#5	1.043	6,428		-	#5	1.043	-
64.00	#6	1.502	96		-	#6	1.502	-
-	#7	2.044	-		-	#7	2.044	-
-	#8	2.670	-		-	#8	2.670	-
-	#9	3.400	-		-	#9	3.400	-
-	#10	4.303	-		-	#10	4.303	-
1,134.67	#11	5.313	6,028		-	#11	5.313	-
		Total =	12,552				Total =	-

Project No. BR R600-297
Str. No. F-16-XB
Prepared by Joe Virdi

BAR LIST - PIER 2 or 3

				4	5	6	7	8	9	10	11	
				0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	

Description	Bar Size	No. Req'd	No. Req'd B.E.I	Length	Type	Dimensions A	Dimensions B	Dimensions C	Epoxy Total Length	Black Total Length	Epoxy Total Weight	Black Total Weight
Pier Column Curved	5	18		23'-3"	Curv				-	418.50	-	436
Pier Column Stirrup	7	10		38'-1"	Stirr				-	380.83	-	778
Pier Column Stirrup	7	14		48'-6"	Stirr				-	679.00	-	1,388
Pier Column Stirrup	5	16		40'-10"	Stirr				-	653.33	-	681
Pier Column Stir	10	24		28'-2"	Str.				-	676.00	-	2,909
Pier Column Stir	5	14		28'-2"	Str.				-	394.33	-	411
Pier Column Top Tie Bar	5	9		16'-1"	Stirr				-	144.75	-	151
Pier Cap Hor Tie @ End	6	6		5'-8"	Stirr				-	34.00	-	51
Pier Cap Vert Tie Bar	7	20		8'-6"	Stirr				-	170.00	-	347
Pier Cap Stirrups @ Top	10	11		30'-10"	Stirr				-	339.17	-	1,459
Hori. Tie Bars	6	8		5'-11"	Stirr				-	47.33	-	71
Hori. Tie Bars	5	11		20'-9"	Stirr				-	228.25	-	238
Horizontal Cont.	6	46		18'-6"	Str.				-	851.00	-	1,278
Cage in Column	5	35		28'-0"	Stirr				-	980.00	-	1,022
Cont. In Cap	5	4		28'-4"	Stirr				-	113.33	-	118
Stirrup in Cap	4	29		6'-10"	Stirr				-	198.17	-	132
Stirrup in Cap	4	4		24'-0"	Stirr				-	96.00	-	64
Stirrup in Corbel	4	18		5'-2"	Stirr				-	93.00	-	62
Pier Cap Curved	5	2		28'-6"	Curv				-	57.00	-	59
Pier Cap End	4	4		2'-0"	Str.				-	8.00	-	5
Pier Cap End	5	4		4'-0"	Str.				-	16.00	-	17
Pier Cap Stirrups	7	4		38'-2"	Stirr				-	152.67	-	312

BAR SUMMARY - PIER 2

EPOXY COATED				NON-EPOXY COATED			
Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight	Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
-	#4	0.668	-	395.17	#4	0.668	264
-	#5	1.043	-	3,005.50	#5	1.043	3,135
-	#6	1.502	-	932.33	#6	1.502	1,400
-	#7	2.044	-	1,382.50	#7	2.044	2,826
-	#8	2.670	-	-	#8	2.670	-
-	#9	3.400	-	-	#9	3.400	-
-	#10	4.303	-	1,015.17	#10	4.303	4,368
-	#11	5.313	-	-	#11	5.313	-
		Total =	-			Total =	95,945

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Project No. BR R600-297
Str. No. F-16-XB
Prepared by Joe Virdi

					4	5	6	7	8	9	10	11	
BAR LIST - APPROACH SLAB ABUTMENT 1					0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	
				No.						Epoxy	Black	Epoxy	Black
		Bar	No.	Req'd				Dimensions		Total	Total	Total	Total
Description		Size	Req'd	B.E.I	Length	Type	A	B	C	Length	Length	Weight	Weight
Slab (Top Long)	E	4	132		23'-3"					3,069.00	-	2,050	-
Slab (Bottom Long)	E	6	394		20'-2"					7,945.67	-	11,934	-
Slab (Top & Bot Trans)	E	5	42		244'-11"					10,286.50	-	10,729	-
Sleeper Trans	E	5	10		244'-11"					2,449.17	-	2,554	-
Sleeper Stirrups	E	5	394		17'-8"					6,960.67	-	7,260	-
Dowel for Guard Rail	E	8	2		4'-0"					8.00	-	21	-

BAR SUMMARY - APPROACH SLAB ABUTMENT 1

EPOXY COATED				NON-EPOXY COATED			
Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight	Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
3,069.00	#4	0.668	2,050	-	#4	0.668	-
19,696.33	#5	1.043	20,543	-	#5	1.043	-
7,945.67	#6	1.502	11,934	-	#6	1.502	-
-	#7	2.044	-	-	#7	2.044	-
8.00	#8	2.670	21	-	#8	2.670	-
-	#9	3.400	-	-	#9	3.400	-
-	#10	4.303	-	-	#10	4.303	-
-	#11	5.313	-	-	#11	5.313	-
		Total =	34,549			Total =	-

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BAR LIST - APPROACH SLAB ABUTMENT 4					4	5	6	7	8	9	10	11	
					0.668	1.043	1.502	2.044	2.670	3.400	4.303	5.313	
				No.						Epoxy	Black	Epoxy	Black
		Bar	No.	Req'd				Dimensions		Total	Total	Total	Total
Description		Size	Req'd	B.E.I	Length	Type	A	B	C	Length	Length	Weight	Weight
Slab (Top Long)	E	4	132		23'-3"					3,069.00	-	2,050	-
Slab (Bottom Long)	E	6	394		20'-2"					7,945.67	-	11,934	-
Slab (Top & Bot Trans)	E	5	42		244'-11"					10,286.50	-	10,729	-
Sleeper Trans	E	5	10		244'-11"					2,449.17	-	2,554	-
Sleeper Stirrups	E	5	394		17'-8"					6,960.67	-	7,260	-
Dowel for Guard Rail	E	8	2		4'-0"					8.00	-	21	-

BAR SUMMARY - APPROACH SLAB ABUTMENT 4

EPOXY COATED				NON-EPOXY COATED			
Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight	Number of Lin. Ft.	Bar Size	Lbs. / Lin. Ft.	Weight
3,069.00	#4	0.668	2,050	-	#4	0.668	-
19,696.33	#5	1.043	20,543	-	#5	1.043	-
7,945.67	#6	1.502	11,934	-	#6	1.502	-
-	#7	2.044	-	-	#7	2.044	-
8.00	#8	2.670	21	-	#8	2.670	-
-	#9	3.400	-	-	#9	3.400	-
-	#10	4.303	-	-	#10	4.303	-
-	#11	5.313	-	-	#11	5.313	-
		Total =	34,549			Total =	-
	Total=		69098				

COLORADO DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS

Check Quantities

202-Removal of Bridge Railing

Median Guardrail = 132.40'

North West Guardrail = 51.0'

Total 183.4

606 Bridge Rail Type 7 (Special)

Total = 51'

606 Bridge Rail Type 7 CC (Special)

Total = 132.4'

By: SJF Date 1/6/11	Project no. BR R600-297	Project code (SA#): 16212
Chk'd: Date	Structure no. F-16-DQ	Sheet 1 of 1

CDOT Form #1034 1/95

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Check Quantities

By: Stephen Fussnecker 1/4/2011

Summary of Quantities

F-16-XB Walls

ITEM NO.	DESCRIPTION	UNIT	F-16-EC	F-16-EE	F-16-EF	TOTAL
202	Removal of Portions of Present Structure	L S	0	1	1	2
206	Structure Excavation	CY	4235		5667	9902
206	Structure Backfill (Class 1)	CY	6082		8740	14822
206	Mechanical Reinforcement of Soil	CY	3415		4897	8312
420	Geomembrane	SY	1548		2794	4342
504	Precast Panel Facing	SF	7558		12809	20368
601	Concrete Class D	CY	102		372	474
601	Structural Concrete Coating	SY	1161		2426	3588
602	Reinforcing Steel (Epoxy Coated)	LB	12609		49770	62379
606	Bridge Rail Type 7	LF	226		913	1139
613	1 Inch Electrical Conduit	LF	4		12	16
613	2 Inch Electrical Conduit	LF	452		1825	2277

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Exc, Bck fill, + MSE

WALL-F-16-EC ~~Geomembrane~~

Assumptions: Assumes 10:1 slope of membrane in PLG calc

	Effective Elev Dist	Top Wall Start Elev	Top Wall End Elev	[redacted]			Top Wall gain ft/ft	Avg Top Wall Elev	Avg Ht	PLG	[redacted]			Excavation Volume	MSE Volume	Backfill Class 1 Volume
1	72.00	5216.23	5217.59	5211	56.00	6.00	0.0189	5216.76	5.76	13.27	42.08	49.29		2558.36	1934.987	130.9758
2	72.00	5216.23	5217.59	5211	16.00	6.67	0.0189	5217.44	6.44	14.62	50.30	46.04		770.72	687.1582	45.458
3	35.71	5217.59	5218.09	5210	8.00	8.00	0.0140	5217.65	7.65	17.15	36.42	78.4		459.28	489.3447	31.18307
4	35.71	5217.59	5218.09	5208	4.00	8.00	0.0140	5217.73	9.73	19.24	46.92	73.82		241.4794	311.3609	21.20696
5	35.71	5217.59	5218.09	5206	8.00	8.40	0.0140	5217.81	11.81	21.72	44.00	92		544	793.9037	56.33181
6	35.71	5217.59	5218.09	5204	8.00	9.80	0.0140	5217.93	13.93	25.24	60.00	109		676	1091.804	76.35769
7	35.71	5217.59	5218.09	5202	4.00	11.67	0.0140	5218.01	16.01	29.19	117.00	146.84		527.68	747.3506	50.72432
8	20.88	5218.09	5218.24	5200	8.00	13.07	0.0072	5218.12	18.12	32.71	49.53	209.23		1035.04	1894.495	127.8546
9	20.88	5218.09	5218.24	5198	4.00	14.47	0.0072	5218.16	20.16	36.15	65.81	100.79		333.2	1166.967	78.31253
10	20.88	5218.09	5218.24	5196	8.00	15.40	0.0072	5218.20	22.20	39.12	48.05	119.54		670.36	2735.649	185.2351
11	20.88	5218.09	5218.24	5194	4.00	17.27	0.0072	5218.25	24.25	43.04	64.78	102.45	-9.869	334.46	1435.751	53.67598
12	37.58	5218.24	5218.65	5192	7.00	18.67	0.0109	5218.28	26.28	46.47	43.70	122.36	-101.918	581.21	756.0759	28.87781
13	37.58	5218.24	5218.65	5192	4.63	18.38	0.0109	5218.34	26.34	46.24	154.87	104.29	-79.9323	599.3075	133.6889	5.529565
14	37.58	5218.24	5218.65	5194	4.00	17.08	0.0109	5218.39	24.39	42.99	115.67	83.38	-23.1023	398.1	1102.799	41.34439
15	37.58	5218.24	5218.65	5196	8.00	15.75	0.0109	5218.45	22.45	39.72	108.20	52.15	-0.7392	641.4	2812.62	105.1711
16	37.58	5218.24	5218.65	5197	13.00	15.19	0.0109	5218.57	21.57	38.28	172.52	76.89		1621.165	4259.164	286.9435
17	237.6250	5209.00	5209.50	5197	161.458	8.00	0.0021	5209.23	12.23	21.74	245.37	272.12	74.74	31873.48952	15801.4	1162.614
18	26.01	5219.84	5219.87	5197	26.1	16.03	0.0012	5219.86	22.86	40.41	394.59	411.24		10516.0815	9562.165	643.0274
19	1.00	5219.87	5219.87	5196	4.00	16.63	0.0000	5219.87	23.87	42.02	390.96	447.42		1676.76	1587.832	106.8187
20	4.00	5219.87	5219.87	5195	4.00	17.50	0.0000	5219.87	24.87	43.89	383.74	452.82		1673.12	1740.9	116.3205
21	12.00	5219.87	5219.87	5193	12.00	19.25	0.0000	5219.87	26.87	47.64	367.09	515.12		5293.26	6206.97	409.7449
22	27.08	5219.71	5219.04	5193	8.00	19.13	-0.0247	5219.61	26.61	47.26	507.85	441.79		3798.56	4072.555	268.5733
23	27.08	5219.71	5219.04	5193	20.00	18.67	-0.0247	5219.26	26.26	46.46	684.24	496.55		11807.9	9807.242	650.4086
24	28.91	5219.04	5217.88	5193	20.00	18.20	-0.0401	5218.64	25.64	45.36	1056.83	672.89		17297.2	9332.524	620.0989
25	28.91	5219.04	5217.88	5193	8.00	17.73	-0.0401	5218.08	25.08	44.33	790.20	772.67		6251.48	3556.94	237.0487
26	28.00	5217.88	5216.15	5195	4.00	16.33	-0.0618	5217.76	22.76	40.61	661.34	655.81		2634.3	1486.45	98.97032
27	28.00	5217.88	5216.15	5197	4.00	14.93	-0.0618	5217.51	20.51	36.96	538.10	535.48		2147.16	1224.815	81.57896
28	28.00	5217.88	5216.15	5199	4.00	13.53	-0.0618	5217.26	18.26	33.31	424.79	422.61		1694.8	988.3472	65.86784
29	28.00	5217.88	5216.15	5201	4.00	11.67	-0.0618	5217.02	16.01	29.20	315.81	315.77		1263.16	747.5802	50.74558
30	28.00	5217.88	5216.15	5203	4.00	10.27	-0.0618	5216.77	13.77	25.55	226.94	227.89		909.66	565.5836	38.54809
31	28.00	5217.88	5216.15	5205	4.00	8.87	-0.0618	5216.52	11.52	21.90	151.75	152.49		608.48	408.7549	28.03085
32	28.00	5217.88	5216.15	5207	4.00	8.00	-0.0618	5216.27	9.27	18.78	94.40	94.99		378.78	296.7543	19.922
33	60.00	5216.15	5213.48	5209	12.00	8.00	-0.0445	5215.88	6.88	16.39	51.86	53.78		633.84	660.768	41.08949
34	60.00	5216.15	5213.48	5209	12.00	7.33	-0.0445	5215.35	6.35	15.19	45.92	48.2		564.72	558.458	35.37403
35	60.00	5216.15	5213.48	5209	12.00	6.67	-0.0445	5214.82	5.81	13.99	40.34	42.65		497.94	465.4326	30.12918
36	60.00	5216.15	5213.48	5209	16.00	6.00	-0.0445	5214.19	5.19	12.70	30.10	32.86		503.68	498.432	33.06277
37	60.00	5216.15	5213.48	5208	4.00	6.67	-0.0445	5213.75	5.75	13.92	35.14	47.87		166.02	153.33	9.902518
38	60.00	5216.15	5213.48	5208	4.00	6.00	-0.0445	5213.57	5.57	13.08	40.17	42.54		165.42	133.656	8.985093
					516.18								Total Volume =	114347.5729	92210.01	CF
													Total Volume =	4235.10	3415.19	6082.044 CY

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Exc, Bckfill, & MSE

WALL-F-16-EF ~~Geomembrane~~

	Effective Elev Dist	Top Wall Start Elev	Top Wall End Elev	[redacted]			Top Wall gain ft/ft	Avg Top Wall Elev	Avg Ht	PLG	[redacted]			Volume	MSE Volume	Backfill Class 1 Volume
1	87.81	5217.04	5218.86	5212.5	7.98	6.15	0.0207	5217.12	4.62	12.28	25.34	23.53		194.966865	226.8395	14.60548
2	87.81	5217.04	5218.86	5212.5	72.12	6.40	0.0207	5217.95	5.45	13.36	41.18	25.88		2418.1836	2516.849	163.7897
3	87.81	5217.04	5218.86	5212.5	7.81	7.53	0.0207	5218.78	6.28	15.31	53.03	45.33		1770.637376	368.9325	23.06638
	94.35	5218.86	5220.01	5212.5	28.20	7.36	0.0122	5219.03	6.53	15.40					1355.586	86.24272
4	94.35	5218.86	5220.01	5212.5	72.00	7.50	0.0122	5219.64	7.14	16.15	51.28	52.3		3728.88	3856.923	248.4383
5	20.00	5220.04	5220.14	5211.5	20.00	8.00	0.0050	5220.09	8.59	18.10	97.98	113.94		3814.56	1374.4	90.27707
	24.00	5220.14	5220.24	5210.5	12.00	8.00	0.0042	5220.17	9.66	19.17					927.84	63.06605
	24.00	5220.14	5220.24	5209.5	4.00	8.00	0.0042	5220.20	10.70	20.21					342.3467	24.03501
6	24.00	5220.14	5220.24	5208.5	4.00	8.40	0.0042	5220.22	11.72	21.63	124.48	124.41		497.78	393.624	27.84802
7	24.00	5220.14	5220.24	5207.5	4.00	8.87	0.0042	5220.23	12.73	23.11	154.09	154.1		616.38	451.7195	32.06667
8	192.00	5220.24	5220.79	5206.5	160.00	9.80	0.0029	5220.47	13.97	25.28	150.42	168.46	191.47	27218.66667	21903.65	1533.602
9	192.00	5220.24	5220.79	5206.5	32.00	10.27	0.0029	5220.74	14.24	26.03	86.96	155.84		3884.8	4681.203	322.9357
10	24.00	5220.79	5220.99	5204.5	4.00	11.61	0.0083	5220.81	16.31	29.43	139.21	170.08		618.58	757.2816	51.86801
11	24.00	5220.79	5220.99	5202.5	8.00	13.01	0.0083	5220.86	18.36	32.88	158.07	228.67		1546.96	1910.562	129.8411
12	24.00	5220.79	5220.99	5200.5	4.00	14.41	0.0083	5220.91	20.41	36.33	221.99	258.36		960.7	1176.24	79.44609
13	24.00	5220.79	5220.99	5198.5	8.00	15.81	0.0083	5220.96	22.46	39.79	185.41	333.41		2075.28	2840.319	190.8891
14	53.00	5220.99	5220.80	5196.5	4.00	17.21	-0.0036	5220.98	24.48	43.21	140.42	260.45	-36.3236	801.74	796.0935	29.98494
15	53.00	5220.99	5220.80	5194.5	8.00	18.61	-0.0036	5220.96	26.46	46.59	109.75	213.55		1293.2	3939.561	262.4038
16	53.00	5220.99	5220.80	5192.0	12.00	20.53	-0.0036	5220.93	28.93	50.98	53.91	200.54	-1.5522	1526.7	7081.181	263.766
17	53.00	5220.99	5220.80	5192.0	10.15	20.30	-0.0036	5220.89	28.89	50.71	232.07	202.93		2208.03825	5952.882	394.9008
18	53.00	5220.99	5220.80	5192.0	8.00	20.30	-0.0036	5220.85	28.85	50.68	253.61	161.9		1662.04	4685.764	310.7051
19	53.00	5220.99	5220.80	5194.0	4.00	18.90	-0.0036	5220.83	26.83	47.26	200.31	161.03		722.68	2028.478	134.9204
20	53.00	5220.99	5220.80	5195.0	8.00	18.20	-0.0036	5220.81	25.81	45.53	212.28	157.43		1478.84	3757.967	250.3466
21	235.54	5211.40	5207.60	5195.0	4.00	11.20	-0.0161	5210.90	15.90	28.61	220.12	127.76		695.76	712.1338	49.12461
22	235.54	5211.40	5207.60	5196.0	40.00	10.50	-0.0161	5210.54	14.54	26.55	332.23	184.4128		10332.856	6107.185	420.1261
23	235.54	5211.40	5207.60	5196.0	40.00	10.00	-0.0161	5209.90	13.90	25.41	322.66	320.9		12871.2	5558.238	384.7676
24	235.54	5211.40	5207.60	5196.0	44.00	9.50	-0.0161	5209.22	13.22	24.23	301.97	311.25		13490.84	5525.127	384.806
25	235.54	5211.40	5207.60	5196.0	36.00	8.75	-0.0161	5208.57	12.57	22.83	271.76	284.96		10020.96	3960.396	281.7084
26	235.54	5211.40	5207.60	5196.0	25.50	8.40	-0.0161	5208.08	12.08	21.99	223.77	264.07		6219.833162	2586.755	184.9709
27	44.16666	5217.34	5216.58	5196.0	15.88	14.88	-0.0172	5217.20	21.20	37.60	124.91	360.96		3857.8078	5010.238	338.4662
28	44.16666	5217.34	5216.58	5196.0	12.00	14.93	-0.0172	5216.96	20.96	37.41	311.54	223.41		3209.7	3755.82	252.2397
29	44.16666	5217.34	5216.58	5198.0	8.00	13.53	-0.0172	5216.79	18.79	33.84	249.34	202.77		1808.44	2033.984	136.9982
30	44.16666	5217.34	5216.58	5200.0	8.00	11.67	-0.0172	5216.65	16.65	29.84	200.88	143.195		1376.3	1554.795	107.0752
31	20.00	5216.58	5216.07	5202.0	8.00	10.27	-0.0255	5216.48	14.48	26.26	175.60	114.08		1158.72	1189.512	82.54439
32	20.00	5216.58	5216.07	5204.0	12.00	8.87	-0.0255	5216.22	12.22	22.60	142.18	100.39		1455.42	1301.016	91.03483
33	28.00	5216.07	5215.33	5205.0	16.00	8.00	-0.0264	5215.86	10.86	20.37	72.57	107.12		5750.08	1389.897	98.06561
	28.00	5216.07	5215.33	5206.0	12.00	8.00	-0.0264	5215.49	9.49	19.00					910.9029	61.57019
	80.00	5215.33	5213.02	5207.0	36.00	8.00	-0.0289	5214.81	7.81	17.32					2249.352	144.0965
34	80.00	5215.33	5213.02	5207.0	24.00	7.33	-0.0289	5213.94	6.94	15.78	62.61	67.41		1560.2448	1221.588	78.93341
35	80.00	5215.33	5213.02	5207.0	20.00	6.67	-0.0289	5213.31	6.31	14.49	52.88	58.06		1109.4	841.5873	55.42048
36	64.00	5213.02	5211.18	5206.0	12.00	7.33	-0.0288	5212.85	6.85	15.69	72.09	76.36		890.7	602.3061	38.79225
37	64.00	5213.02	5211.18	5206.0	24.00	6.67	-0.0288	5212.33	6.33	14.51	58.99	67.31		1515.6	1013.306	66.77827
38	64.00	5213.02	5211.18	5206.0	28.00	6.00	-0.0288	5211.58	5.58	13.09	46.22	54.57		1411.06	937.86	63.07871
39	32.00	5211.18	5210.33	5205.0	8.00	6.67	-0.0266	5211.07	6.07	14.25	64.34	68.41		531	324.0953	21.16822
40	32.00	5211.18	5210.33	5205.0	24.00	6.00	-0.0266	5210.65	5.65	13.16	45.30	59.48		1257.36	813.42	54.83983
41	24.00	5210.33	5209.65	5204.0	12.00	6.67	-0.0283	5210.16	6.16	14.34	58.71	66.64		752.1	493.0464	32.3
42	24.00	5210.33	5209.65	5204.0	12.00	6.00	-0.0283	5209.82	5.82	13.33	46.26	54.31		603.42	419.04	28.4272
43	24.00	5209.65	5208.92	5203.0	24.00	6.67	-0.0304	5209.29	6.28	14.46	50.68	68.15		1425.96	1006.103	66.19917
44	20.00	5208.92	5208.30	5202.0	8.00	7.33	-0.0310	5208.80	6.80	15.63	68.39	73.39		567.12	398.5174	25.62266
45	20.00	5208.92	5208.30	5202.0	12.00	6.67	-0.0310	5208.49	6.49	14.66	55.16	63.55		712.26	519.1394	34.39987
46	64.00	5208.30	5206.44	5201.0	20.00	7.33	-0.0291	5208.01	7.01	15.85	64.83	79.14		1439.7	1027.574	66.5433
47	64.00	5208.30	5206.44	5201.0	24.00	6.67	-0.0291	5207.37	6.37	14.55	56.24	59.98		1394.64	1019.71	67.29453
48	64.00	5208.30	5206.44	5201.0	20.00	6.00	-0.0291	5206.73	5.73	13.24	47.89	51.73		996.2	687.675	46.49979
49	44.00	5206.44	5205.30	5200.0	16.00	6.67	-0.0259	5206.23	6.23	14.41	59.73	70.29		1040.16	665.1567	43.68582
50	44.00	5206.44	5205.30	5200.0	28.00	6.00	-0.0259	5205.66	5.66	13.17	43.04	55		1372.56	951.3382	64.1705
51	60.00	5205.30	5203.97	5199.0	16.00	6.67	-0.0222	5205.12	6.12	14.30	58.11	65.26		986.96	653.411	42.75005
52	60.00	5205.30	5203.97	5199.0	44.00	6.00	-0.0222	5204.46	5.46	12.96	43.91	53.55		2144.12	1440.824	96.47499
					1195.63								Total Volume =	152998.09	132207.3	CF
													Total Volume =	5666.60	4896.57	8740.019 CY

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WALL-F-16-EC Geomembrane & Precast Panel

Assumptions: Assumes 10:1 slope of membrane in PLG calc

	Effective Elev Dist	Top Wall Start Elev	Top Wall End Elev	[redacted]			Top Wall gain ft/ft	Avg Top Wall Elev	Avg Ht	PLG	Area SF	Precast Panel Area
1	72.00	5216.23	5217.59	5211	56.00	6.00	0.0189	5216.76	5.76	13.27	742.87	322.50
2	72.00	5216.23	5217.59	5211	16.00	6.67	0.0189	5217.44	6.44	14.62	233.86	103.02
3	35.71	5217.59	5218.09	5210	8.00	8.00	0.0140	5217.65	7.65	17.15	137.24	61.17
4	35.71	5217.59	5218.09	5208	4.00	8.00	0.0140	5217.73	9.73	19.24	76.96	38.92
5	35.71	5217.59	5218.09	5206	8.00	8.40	0.0140	5217.81	11.81	21.72	173.80	94.51
6	35.71	5217.59	5218.09	5204	8.00	9.80	0.0140	5217.93	13.93	25.24	201.91	111.41
7	35.71	5217.59	5218.09	5202	4.00	11.67	0.0140	5218.01	16.01	29.19	116.78	64.04
8	20.88	5218.09	5218.24	5200	8.00	13.07	0.0072	5218.12	18.12	32.71	261.64	144.95
9	20.88	5218.09	5218.24	5198	4.00	14.47	0.0072	5218.16	20.16	36.15	144.60	80.65
10	20.88	5218.09	5218.24	5196	8.00	15.40	0.0072	5218.20	22.20	39.12	313.00	177.64
11	20.88	5218.09	5218.24	5194	4.00	17.27	0.0072	5218.25	24.25	43.04	172.16	96.99
12	37.58	5218.24	5218.65	5192	8.00	18.67	0.0109	5218.28	26.28	46.48	371.81	210.27
13	37.58	5218.24	5218.65	5192	5.63	18.38	0.0109	5218.36	26.36	46.26	260.22	148.26
14	37.58	5218.24	5218.65	5194	4.00	17.08	0.0109	5218.41	24.41	43.01	172.05	97.64
15	37.58	5218.24	5218.65	5196	8.00	15.75	0.0109	5218.48	22.48	39.75	317.97	179.81
16	37.58	5218.24	5218.65	5197	13.00	15.19	0.0109	5218.59	21.59	38.30	497.90	280.68
												0.00
17	237.6250	5209.00	5209.50	5197	161.458	8.00	0.0021	5209.23	12.23	21.74	3510.78	1975.17
												0.00
18	26.01	5219.84	5219.87	5197	26.1	16.03	0.0012	5219.86	22.86	40.41	1054.58	596.52
19	1.00	5219.87	5219.87	5196	4.00	16.63	0.0000	5219.87	23.87	42.02	168.08	95.48
20	4.00	5219.87	5219.87	5195	4.00	17.50	0.0000	5219.87	24.87	43.89	175.57	99.48
21	12.00	5219.87	5219.87	5193	12.00	19.25	0.0000	5219.87	26.87	47.64	571.73	322.44
22	27.08	5219.71	5219.04	5193	8.00	19.13	-0.0247	5219.61	26.61	47.26	378.12	212.89
23	27.08	5219.71	5219.04	5193	20.00	18.67	-0.0247	5219.26	26.26	46.46	929.16	525.29
24	28.91	5219.04	5217.88	5193	20.00	18.20	-0.0401	5218.64	25.64	45.36	907.23	512.78
25	28.91	5219.04	5217.88	5193	8.00	17.73	-0.0401	5218.08	25.08	44.33	354.63	200.62
26	28.00	5217.88	5216.15	5195	4.00	16.33	-0.0618	5217.76	22.76	40.61	162.43	91.03
27	28.00	5217.88	5216.15	5197	4.00	14.93	-0.0618	5217.51	20.51	36.96	147.83	82.04
28	28.00	5217.88	5216.15	5199	4.00	13.53	-0.0618	5217.26	18.26	33.31	133.24	73.05
29	28.00	5217.88	5216.15	5201	4.00	11.67	-0.0618	5217.02	16.01	29.20	116.80	64.06
30	28.00	5217.88	5216.15	5203	4.00	10.27	-0.0618	5216.77	13.77	25.55	102.20	55.07
31	28.00	5217.88	5216.15	5205	4.00	8.87	-0.0618	5216.52	11.52	21.90	87.61	46.08
32	28.00	5217.88	5216.15	5207	4.00	8.00	-0.0618	5216.27	9.27	18.78	75.13	37.09
33	60.00	5216.15	5213.48	5209	12.00	8.00	-0.0445	5215.88	6.88	16.39	196.69	82.60
34	60.00	5216.15	5213.48	5209	12.00	7.33	-0.0445	5215.35	6.35	15.19	182.24	76.19
35	60.00	5216.15	5213.48	5209	12.00	6.67	-0.0445	5214.82	5.81	13.99	167.90	69.78
36	60.00	5216.15	5213.48	5209	16.00	6.00	-0.0445	5214.19	5.19	12.70	203.17	83.07
37	60.00	5216.15	5213.48	5208	4.00	6.67	-0.0445	5213.75	5.75	13.92	55.70	22.99
38	60.00	5216.15	5213.48	5208	4.00	6.00	-0.0445	5213.57	5.57	13.08	52.30	22.28

Total Geomembrane Area for F-16-EC = 13927.87 7558.44 SF
Total Geomembrane Area for F-16-EC = 1547.54 SY

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WALL-F-16-EF Geomembrane & Precast Panel

	Effective Elev Dist	Top Wall Start Elev	Top Wall End Elev	[redacted]			Top Wall gain ft/ft	Avg Top Wall Elev	Avg Ht	PLG	Area SF	Precast Panel Area
1	87.81	5217.04	5218.86	5212.5	7.98	6.15	0.0207	5217.12	4.62	12.28	97.97	36.88
2	87.81	5217.04	5218.86	5212.5	72.12	6.40	0.0207	5217.95	5.45	13.36	963.49	393.26
3	87.81	5217.04	5218.86	5212.5	7.81	7.53	0.0207	5218.78	6.28	15.31	119.53	49.03
	94.35	5218.86	5220.01	5212.5	28.20	7.36	0.0122	5219.03	6.53	15.40	434.23	184.18
4	94.35	5218.86	5220.01	5212.5	72.00	7.50	0.0122	5219.64	7.14	16.15	1162.84	514.26
5	20.00	5220.04	5220.14	5211.5	20.00	8.00	0.0050	5220.09	8.59	18.10	361.98	171.80
	24.00	5220.14	5220.24	5210.5	12.00	8.00	0.0042	5220.17	9.66	19.17	230.09	115.98
	24.00	5220.14	5220.24	5209.5	4.00	8.00	0.0042	5220.20	10.70	20.21	80.83	42.79
6	24.00	5220.14	5220.24	5208.5	4.00	8.40	0.0042	5220.22	11.72	21.63	86.50	46.86
7	24.00	5220.14	5220.24	5207.5	4.00	8.87	0.0042	5220.23	12.73	23.11	92.45	50.93
8	192.00	5220.24	5220.79	5206.5	160.00	9.80	0.0029	5220.47	13.97	25.28	4045.09	2235.07
9	192.00	5220.24	5220.79	5206.5	32.00	10.27	0.0029	5220.74	14.24	26.03	832.87	455.81
10	24.00	5220.79	5220.99	5204.5	4.00	11.61	0.0083	5220.81	16.31	29.43	117.73	65.23
11	24.00	5220.79	5220.99	5202.5	8.00	13.01	0.0083	5220.86	18.36	32.88	263.06	146.85
12	24.00	5220.79	5220.99	5200.5	4.00	14.41	0.0083	5220.91	20.41	36.33	145.34	81.63
13	24.00	5220.79	5220.99	5198.5	8.00	15.81	0.0083	5220.96	22.46	39.79	318.29	179.65
14	53.00	5220.99	5220.80	5196.5	4.00	17.21	-0.0036	5220.98	24.48	43.21	172.86	97.93
15	53.00	5220.99	5220.80	5194.5	8.00	18.61	-0.0036	5220.96	26.46	46.59	372.76	211.69
16	53.00	5220.99	5220.80	5192.0	12.00	20.53	-0.0036	5220.93	28.93	50.98	611.77	347.11
17	53.00	5220.99	5220.80	5192.0	10.15	20.30	-0.0036	5220.89	28.89	50.71	514.81	293.25
18	53.00	5220.99	5220.80	5192.0	8.00	20.30	-0.0036	5220.85	28.85	50.68	405.43	230.83
19	53.00	5220.99	5220.80	5194.0	4.00	18.90	-0.0036	5220.83	26.83	47.26	189.02	107.33
20	53.00	5220.99	5220.80	5195.0	8.00	18.20	-0.0036	5220.81	25.81	45.53	364.26	206.48
												0.00
21	235.54	5211.40	5207.60	5195.0	4.00	11.20	-0.0161	5210.90	15.90	28.61	114.44	63.58
22	235.54	5211.40	5207.60	5196.0	40.00	10.50	-0.0161	5210.54	14.54	26.55	1062.17	581.64
23	235.54	5211.40	5207.60	5196.0	40.00	10.00	-0.0161	5209.90	13.90	25.41	1016.33	555.82
24	235.54	5211.40	5207.60	5196.0	44.00	9.50	-0.0161	5209.22	13.22	24.23	1066.13	581.59
25	235.54	5211.40	5207.60	5196.0	36.00	8.75	-0.0161	5208.57	12.57	22.83	822.03	452.62
26	235.54	5211.40	5207.60	5196.0	25.50	8.40	-0.0161	5208.08	12.08	21.99	560.67	307.95
												0.00
27	44.16666	5217.34	5216.58	5196.0	15.88	14.88	-0.0172	5217.20	21.20	37.60	597.12	336.71
28	44.16666	5217.34	5216.58	5196.0	12.00	14.93	-0.0172	5216.96	20.96	37.41	448.95	251.56
29	44.16666	5217.34	5216.58	5198.0	8.00	13.53	-0.0172	5216.79	18.79	33.84	270.71	150.33
30	44.16666	5217.34	5216.58	5200.0	8.00	11.67	-0.0172	5216.65	16.65	29.84	238.71	133.23
31	20.00	5216.58	5216.07	5202.0	8.00	10.27	-0.0255	5216.48	14.48	26.26	210.09	115.82
32	20.00	5216.58	5216.07	5204.0	12.00	8.87	-0.0255	5216.22	12.22	22.60	271.25	146.68
33	28.00	5216.07	5215.33	5205.0	16.00	8.00	-0.0264	5215.86	10.86	20.37	325.90	173.74
	28.00	5216.07	5215.33	5206.0	12.00	8.00	-0.0264	5215.49	9.49	19.00	227.98	113.86
	80.00	5215.33	5213.02	5207.0	36.00	8.00	-0.0289	5214.81	7.81	17.32	623.48	281.17
34	80.00	5215.33	5213.02	5207.0	24.00	7.33	-0.0289	5213.94	6.94	15.78	378.77	166.66
35	80.00	5215.33	5213.02	5207.0	20.00	6.67	-0.0289	5213.31	6.31	14.49	289.72	126.18
36	64.00	5213.02	5211.18	5206.0	12.00	7.33	-0.0288	5212.85	6.85	15.69	188.22	82.17
37	64.00	5213.02	5211.18	5206.0	24.00	6.67	-0.0288	5212.33	6.33	14.51	348.17	151.92
38	64.00	5213.02	5211.18	5206.0	28.00	6.00	-0.0288	5211.58	5.58	13.09	366.49	156.31
39	32.00	5211.18	5210.33	5205.0	8.00	6.67	-0.0266	5211.07	6.07	14.25	114.01	48.59
40	32.00	5211.18	5210.33	5205.0	24.00	6.00	-0.0266	5210.65	5.65	13.16	315.73	135.57
41	24.00	5210.33	5209.65	5204.0	12.00	6.67	-0.0283	5210.16	6.16	14.34	172.05	73.92
42	24.00	5210.33	5209.65	5204.0	12.00	6.00	-0.0283	5209.82	5.82	13.33	159.92	69.84
43	24.00	5209.65	5208.92	5203.0	24.00	6.67	-0.0304	5209.29	6.28	14.46	347.09	150.84
44	20.00	5208.92	5208.30	5202.0	8.00	7.33	-0.0310	5208.80	6.80	15.63	125.07	54.37
45	20.00	5208.92	5208.30	5202.0	12.00	6.67	-0.0310	5208.49	6.49	14.66	175.96	77.83
46	64.00	5208.30	5206.44	5201.0	20.00	7.33	-0.0291	5208.01	7.01	15.85	316.95	140.19
47	64.00	5208.30	5206.44	5201.0	24.00	6.67	-0.0291	5207.37	6.37	14.55	349.13	152.88
48	64.00	5208.30	5206.44	5201.0	20.00	6.00	-0.0291	5206.73	5.73	13.24	264.74	114.61
49	44.00	5206.44	5205.30	5200.0	16.00	6.67	-0.0259	5206.23	6.23	14.41	230.56	99.72
50	44.00	5206.44	5205.30	5200.0	28.00	6.00	-0.0259	5205.66	5.66	13.17	368.74	158.56
51	60.00	5205.30	5203.97	5199.0	16.00	6.67	-0.0222	5205.12	6.12	14.30	228.80	97.96
52	60.00	5205.30	5203.97	5199.0	44.00	6.00	-0.0222	5204.46	5.46	12.96	570.42	240.14

Total Geomembrane Area for F-16-EF = 25149.71 12809.37 SF
Total Geomembrane Area for F-16-EF = 2794.41 SY

Project # BR R600-297
Sub-Account # 16212
Str No. WALL-F-16-EC,EE,EF
By Stephen Fussnecker

202 Removal of Portions of Present Structure [Each]

	F-16-EC	F-16-EE	F-16-EF
		1	1
Total =	0	1	1

606 Bridge Rail Type 7 [Lin. Ft.]

Assumptions:

Rail Length on F-16-EC =	225.822	Ft
Rail Length on F-16-EF =	912.682	Ft

613 1 Inch Electrical Conduit [Lin. Ft.]

Assumptions: 2 ft per pedestal

Length on F-16-EC =	4.000	Ft
Length on F-16-EF =	12.000	Ft

613 2 Inch Electrical Conduit [Lin. Ft.]

Assumptions:

Length on F-16-EC =	451.645	Ft
Length on F-16-EF =	1825.365	Ft

Project # BR R600-297
Sub-Account # 16212
Str No. WALL-F-16-EC,EE,EF
By Stephen Fussnecker

601 Concrete Class D [Yd^3]

Assumptions:

0.037	CY/Ft Leveling pad
0.086	CY/ 1 Ft step
0.037	CY/Ft of >1' steps
0.101	CY/Ft of horizontal coping
0.106	CY/Ft of vertical coping
0.051	CY/Ft of horizontal coping under deck
0.482	CY per pedestal
0.353	CY/Ft anchoring slab

WALL-F-16-EC

Leveling Pad:

length of wall =	517.625		
# of 1' steps	5		
Other steps:	Ht	#	
	2	20	
Volume w/out steps =	19.17	CY	
Volume of 1' steps =	0.43	CY	
Volume of other steps =	1.48	CY	
Total Leveling Pad =	21.08	CY	

Coping:

Total Length of Vert =	16.24	Ft
Volume of vertical =	1.72	CY
Total Length of Hor =	242.62	Ft
Volume of Horizontal =	24.55	CY
Total Length under deck =	6.56	Ft
Volume under deck =	0.34	CY
Total Volume of Coping =	26.61	CY

Pedestal:

# of pedestals =	2.00	
Volume of Pedestals =	0.96	CY

Anchoring slab:

Total length =	225.82	Ft
Total volume =	79.78	CY

Total concrete = 101.82 CY

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WALL-F-16-EF

Leveling Pad:			
length of wall =	1195.531		
# of 1' steps	19		
Other steps:	Ht	#	
	2	11	
	2.5	1	
Volume w/out steps =	44.28	CY	
Volume of 1' steps =	1.64	CY	
Volume of other steps =	0.91	CY	
Total Leveling Pad =	46.83	CY	
Coping:			
Total Length of Vert =	19.17	Ft	
Volume of vertical =	2.04	CY	
Total Length of Hor =	238.36	Ft	
Volume of Horizontal =	24.12	CY	
Total Length under deck =	6.86	Ft	
Volume under deck =	0.35	CY	
Total Volume of Coping =	26.51	CY	
Pedestal:			
# of pedestals =	6.00		
Volume of Pedestals =	2.89	CY	
Anchoring slab:			
Total length =	912.68	Ft	
Total volume =	322.42	CY	
Total concrete =	**372.14**	**CY**	

601 Structural Concrete Coating [SY]

Assumptions:

WALL-F-16-EC

Area of wall =	7558.44	SF
Perimeter of Rail =	8.9345	Ft
Length of rail =	225.8224	Ft
Area of Rail =	2017.6102	SF
Coping:		
Total Length of Vert =	16.24	Ft
area/ft of vertical =	3.29	SF
Total Length of Hor =	242.62	Ft
Area/ft of Horizontal =	3.32	SF
Total Length under deck =	6.56	Ft
Area/ft under deck =	2.68	SF
Total Area of Coping =	876.56	SF
Total Area =	10452.6139	SF
Total Area =	**1161.4015**	**SY**

WALL-F-16-EF

Area of wall =	12809.37	SF
Perimeter of Rail =	8.9345	Ft
Length of rail =	912.6824	Ft
Area of Rail =	8154.3609	SF
Coping:		
Total Length of Vert =	19.17	Ft
area/ft of vertical =	3.29	SF
Total Length of Hor =	238.36	Ft
Area/ft of Horizontal =	3.32	SF
Total Length under deck =	6.86	Ft
Area/ft under deck =	2.68	SF
Total Area of Coping =	872.83	SF
Total Area =	21836.5600	SF
Total Area =	**2426.2844**	**SY**

Project # BR R600-297
Sub-Account # 16212
Str No. WALL-F-16-EC,EE,EF
By Stephen Fussnecker

602 Reinforcing Steel (Epoxy Coated [Lb]

Assumptions:		
	0.000	Lb/Ft Leveling pad
	16.370	Lb/2.5' steps
	15.360	Lb/2' steps
	11.520	Lb/1' steps
	53.710	Lb/Ft anchoring slab
	57.670	Lb/Pedestal

WALL-F-16-EC

Length of anchoring Slabs =	225.8224	Ft
Steel in Anchor slab =	12128.92	Lb
Length of Leveling pad =	517.625	Ft
Steel in Leveling pad =	0	Lb
# of 1' steps =	5	Ft
Steel in 1' steps =	57.6	Lb
# of 2' steps =	20	Ft
Steel in 2' steps =	307.2	Lb
# of 2.5' steps =		Ft
Steel in 2.5' steps =	0	Lb
# of pedestals =	2	Ft
Steel in pedestals =	115.34	Lb
Total Steel =	**12609.06**	**Lb**

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WALL-F-16-EF

Length of anchoring Slabs =	912.6824	Ft
Steel in Anchor slab =	49020.17	Lb
Length of Leveling pad =	1195.531	Ft
Steel in Leveling pad =	0	Lb
# of 1' steps =	19	Ft
Steel in 1' steps =	218.88	Lb
# of 2' steps =	11	Ft
Steel in 2' steps =	168.96	Lb
# of 2.5' steps =	1	Ft
Steel in 2.5' steps =	16.37	Lb
# of pedestals =	6	Ft
Steel in pedestals =	346.02	Lb
Total Steel =	**49770.4**	**Lb**