Final Design Calculations for Abutments & Piling - VE Alternate

In the Town of

Rockingham, Vt.

BRF-0126(12)

Prepared for:

Cold River Bridges

By:

Calderwood Engineering etc

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Cover photo of Cavendish
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BRF-0146(13)

Table of Contents – Abutments & Piling

Table of Contents i
Project notes: 1

Piling Properties, total movement & thermal range 1
Reactions from Superstructure 2-7

Live Load Component applied to single piling distribution 7
Moments in piling for zones 1 & 2; X & Y, & Eff Lengths 8

Flexure weak and strong axes ..... 9 -10
Axial Capacity of piling X & Y Axes 11-12
Combined Axial & Flexural capacities & demands 13

Passive Pressure on Abutments (using log spiral method) 14-15
Concrete Abutment design below construction joint (Hz reinforcing) 15-19
Concrete Abutment Design above construction joint (Hz reinforcing) 20-24
Concrete Abutment Design Vertical Reinforcing 25-29

Appendix A- Risa spreadsheet output for Live Load & SIDL at abutments ..... 2 pages
Appendix B- L-Pile output file for both strong and weak axis ..... 7 pages each axes

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Project: Rockingham VT Bridge
Contractor: Cold River Bridges

Value Engineering Design: Calderwood Engineering
Design Computations by: Eric T. Calderwood, PE
Design Check Computations by: Gregory MacAlister, EIT

Project Notes:

VTrans Structural Design Manual

Integral Abutments

Thermal forces use 0.0000065 in/in/degree F
Either Procedure A or Procedure B is acceptable
Use Procedure B

$T_{min} := -20\ °F$ $\quad T_{max} := 105\ °F$

$T_{avg} := \frac{T_{max} + T_{min}}{2} = 42.5\ °F$

say Girders may be welded down to the steel leveling plates at the abutments when the temperature is between 30 degrees and 60 degrees

$Tc_{min} := 30\ °F$ $\quad Tc_{max} := 60\ °F$ $\quad \varepsilon_{stl} := 0.0000065\ \frac{in}{in \cdot \Delta°F}$ $\quad L_{tot} := 200.00\ ft$

$T_{rangemax} := \max\left(\left\langle Tc_{max} - T_{min}\right\rangle, \left\langle T_{max} - Tc_{min}\right\rangle\right) = 80\ \Delta°F$

$\Delta_{tot1way} := T_{rangemax} \cdot \varepsilon_{stl} \cdot L_{tot} = 1.248\ in$

Total expansion of bridge from neutral position set at deck placement

$\delta_{pile} := \frac{\Delta_{tot1way}}{2} = 0.624\ in$

Total expansion of bridge from neutral position set at deck placement into backfill at each abutment

$n_{pile} := 6$ $\quad A_{pile} := 21.8\ in^2$ $\quad Skew := 30\ °$

$I_{yypile} := 186\ in^4$ $\quad I_{xxpile} := 569\ in^4$ $\quad F_{Yhp} := 50\ ksi$ $\quad B_{ab} := 3\ ft$

$\gamma_{cn} := 150\ pcf$

$r_{yypile} := 2.92\ in$ $\quad r_{xxpile} := 5.11\ in$ $\quad \omega_{pile} := 74\ plf$ $\quad \gamma_{pvt} := 145\ pcf$

$b_{fpile} := 12.215\ in$ $\quad t_{fpile} := 0.610\ in$

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Reactions: from superstructure - Reference steel design computations, uniform load of 1 klf and associated reactions at abutments and piers

Uniform load (spans 1,2 & 3 Loaded)

Distance	Vmax	Mmin	Mmax		No	Rmin	Rmax
0.00	25.9	0.0	0.0		1	0.0	25.9
6.55	19.3	0.0	148.0		2	0.0	74.1
13.10	12.8	0.0	253.1		3	0.0	74.1
19.65	6.2	0.0	315.2		4	0.0	25.9
26.20	0.3	0.0	334.5				
32.75	6.9	0.0	310.9				
39.30	13.4	0.0	244.3				
45.85	20.0	0.0	134.9				
52.40	26.5	-17.4	0.0				
58.95	33.1	-212.7	0.0				
65.50	39.6	-450.8	0.0				
72.40	27.6	-236.6	0.0				
79.30	20.7	-69.9	0.0				
86.20	13.8	0.0	49.1				
93.10	6.9	0.0	120.5				
100.00	0.0	0.0	144.3				
106.90	6.9	0.0	120.5				
113.80	13.8	0.0	49.1				
120.70	20.7	-69.9	0.0				
127.60	27.6	-236.6	0.0				
134.50	39.6	-450.8	0.0				
141.05	33.1	-212.7	0.0				
147.60	26.5	-17.4	0.0				
154.15	20.0	0.0	134.9				
160.70	13.4	0.0	244.3				
167.25	6.9	0.0	310.9				
173.80	0.3	0.0	334.5				
180.35	6.2	0.0	315.2				
186.90	12.8	0.0	253.1				
193.45	19.3	0.0	148.0				
200.00	25.9	0.0	0.0				

$$R_{a1klf} = 25.8671 \frac{kip}{klf}$$

$$R_{p1klf} = 74.1329 \frac{kip}{klf}$$

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Dead Load Computations:

$S := 10.5\ ft \qquad OH := 2\ ft + 7\ in = 2.5833\ ft \qquad t_{deck} := 9\ in \qquad N_b := 4$

$W_{stl} := 199.68\ plf = 0.1997\ klf$

$W_{stl} = 0.1997\ klf$

$W_{extdeck} := \left(\frac{S}{2} + OH\right) \cdot t_{deck} \cdot \gamma_{cn} = 881.25\ plf$

$W_{extdeck} = 0.8813\ klf$

$W_{intdeck} := S \cdot t_{deck} \cdot \gamma_{cn} = 1181.25\ plf$

$W_{intdeck} = 1.1813\ klf$

$W_{extpvt} := \left(\frac{S}{2} + OH - 10\ in\right) \cdot 3.25\ in \cdot \gamma_{pvt} = 274.8958\ plf$

$W_{extpvt} = 0.2749\ klf$

$W_{intpvt} := S \cdot 3.25\ in \cdot \gamma_{pvt} = 412.3438\ plf$

$W_{intpvt} = 0.4123\ klf$

60% of railing load applied to exterior girders, steel railing taken at 25plf on top of concrete railing

$W_{extrail} := 60\% \cdot (10\ in \cdot (2\ ft + 3.25\ in) \cdot \gamma_{cn} + 25\ plf) = 185.3125\ plf$

$W_{extrail} = 0.1853\ klf$

$W_{intrail} := 40\% \cdot 2 \cdot \frac{(10\ in \cdot (2\ ft + 3.25\ in) \cdot \gamma_{cn} + 25\ plf)}{(N_b - 2)} = 123.5417\ plf$

$W_{intrail} = 0.1235\ klf$

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$$R_{pDCS} := \frac{R_{a1klf} \cdot \langle 4 \cdot W_{stl} + 2 \cdot W_{extdeck} + 2 \cdot W_{extrail} + 2 \cdot W_{intdeck} + 2 \cdot W_{intrail} \rangle}{n_{pile}} = 23.8901\ kip$$

Pile Reaction due to DC from Superstructure

$$R_{pDWS} := \frac{R_{a1klf} \cdot \langle 2 \cdot W_{extpvt} + 2 \cdot W_{intpvt} \rangle}{n_{pile}} = 5.9256\ kip$$

Pile Reaction due to DW from Super

$$R_{pAC} := \frac{\left(54\ ft \cdot B_{ab} \cdot 5.25\ ft + \frac{54\ ft + 50.48\ ft}{2} \cdot B_{ab} \cdot 2.8733\ ft + \frac{4.60\ ft + 10.964\ ft}{2} \cdot B_{ab} \cdot 0.9167\ ft\right) \cdot \gamma_{cn}}{n_{pile}}$$

$$R_{pAC} = 33.0551\ kip$$

Pile Reaction due to Self Weight of Abutment concrete

$$R_{pAS} := \frac{20\ ft \cdot \cos(30\ °) \cdot \frac{35\ ft}{\cos(30\ °)} \cdot (1\ ft + 3\ in) \cdot \gamma_{cn}}{2 \cdot n_{pile}} = 10.9375\ kip$$

Pile Reaction due to Self Weight of Approach slab (1/2 the slab bearing on the abutment 1/2 bearing on soil)

$$R_{pASDW} := \frac{20\ ft \cdot \cos(30\ °) \cdot \frac{35\ ft}{\cos(30\ °)} \cdot (3\ in) \cdot \gamma_{pvt}}{2 \cdot n_{pile}} = 2.1146\ kip$$

Pile Reaction due to Pavement on Approach slab (1/2 the slab bearing on the abutment 1/2 bearing on soil)

$$R_{pP} := 80\ ft \cdot \omega_{pile} = 5.92\ kip$$

$$R_{pDC} := R_{pDCS} + R_{pAC} + R_{pAS} + R_{pP} = 73.8028\ kip$$

Total Reaction at pile due to DC loads

$$R_{pDW} := R_{pDWS} + R_{pASDW} = 8.0402\ kip$$

Total Reaction at pile due to DW loads

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$R_{LaneAbut} := 0.64\ klf \cdot R_{a1klf} = 16.555\ kip$ *Abutment Reaction due to Lane Load Unfactored Undistributed*

$R_{LanePier} := 0.64\ klf \cdot R_{p1klf} = 47.445\ kip$ *Pier Reaction due to Lane Load Unfactored Undistributed*

HS20 Truck

Distance	Vmax	Mmin	Mmax	No	Rmin	Rmax
0.00	52.9	0.0	0.0	1	-5.6	52.9
6.55	44.4	-37.0	290.7	2	-9.5	69.8
13.10	36.2	-74.0	523.8	3	-9.4	69.8
19.65	28.4	-111.0	653.2	4	-5.6	54.4
26.20	21.3	-148.0	687.4			
32.75	27.8	-185.0	679.1			
39.30	35.6	-222.0	611.8			
45.85	42.7	-259.0	469.8			
52.40	49.2	-296.0	270.2			
58.95	55.1	-333.0	99.5			
65.50	60.1	-424.8	110.5			
72.40	49.7	-371.4	81.9			
79.30	41.8	-318.0	314.8			
86.20	33.7	-264.7	482.6			
93.10	25.6	-211.3	572.6			
100.00	22.2	-160.2	611.5			
106.90	30.3	-214.4	589.1			
113.80	38.4	-268.5	485.8			
120.70	46.1	-322.7	309.7			
127.60	52.8	-376.8	71.0			
134.50	61.3	-431.0	109.0			
141.05	55.4	-328.0	98.1			
147.60	48.7	-291.6	270.6			
154.15	41.4	-255.1	468.4			
160.70	33.5	-218.7	604.4			
167.25	25.1	-182.2	665.7			
173.80	22.6	-145.8	698.2			
180.35	30.5	-109.3	652.5			
186.90	38.9	-72.9	509.1			
193.45	46.5	-36.4	304.6			
200.00	54.4	0.0	0.0			

$R_{ahs20} = 52.9436\ kip$ $R_{phs20} = 69.831\ kip$

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Neg Moment Truck Train

Distance	Vmax	Mmin	Mmax		No	Rmin	Rmax	
0.00	47.6	0.0	0.0		1	-3.7	47.6	
6.55	39.9	-24.3	261.6		2	-8.6	70.1	
13.10	32.6	-48.6	471.4		3	-8.5	70.0	
19.65	25.6	-73.0	587.9		4	-3.7	49.6	
26.20	19.1	-97.3	618.6					
32.75	26.3	-121.6	611.2					
39.30	33.1	-145.9	550.7					
45.85	39.1	-170.3	422.8					
52.40	44.3	-251.0	243.2					
58.95	49.6	-414.9	89.7					
65.50	54.3	-634.1	99.7					
72.40	47.6	-427.3	97.2					
79.30	42.2	-286.2	261.8					
86.20	36.2	-238.2	371.9					
93.10	29.8	-190.2	429.3					
100.00	23.1	-144.5	457.3					
106.90	27.9	-193.3	450.2					
113.80	33.6	-242.1	392.3					
120.70	39.2	-290.9	280.7					
127.60	44.1	-429.0	110.8					
134.50	56.1	-636.9	98.1					
141.05	51.7	-417.7	88.3					
147.60	46.7	-253.3	249.4					
154.15	41.0	-167.7	426.6					
160.70	34.7	-143.8	547.2					
167.25	27.6	-119.8	607.2		Vsp=		38.8	kips
173.80	21.0	-95.8	640.6		Mminsp=		-263.0	k-ft
180.35	28.2	-71.9	596.1		Mmaxsp=		344.6	k-ft
186.90	35.6	-47.9	466.3		Mmax=		640.6	k-ft
193.45	42.5	-24.0	278.4		Mmin=		-636.9	k-ft
200.00	49.6	0.0	0.0		Vmax=		56.1	kips

$R_{anmtt} = 47.6492$ kip $\qquad R_{pnmtt} = 70.0848$ kip

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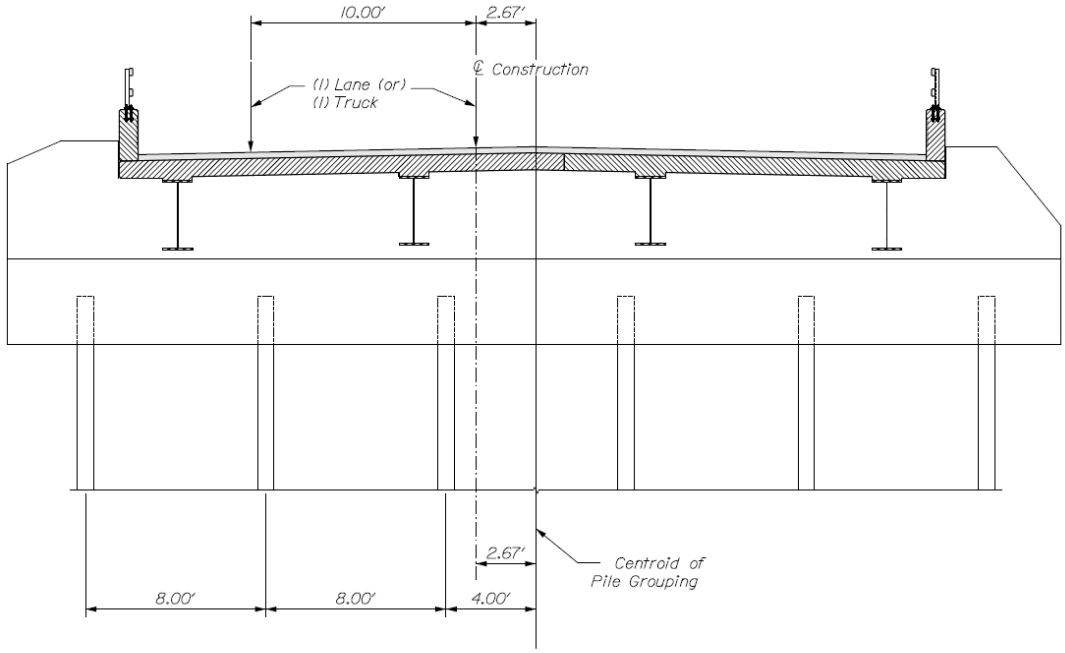

LOCATION OF CENTROID OF DESIGN VEHICLES W.R.T. PILING
ABUTMENT #1 SHOWN - ABUTMENT #2 SIMILAR

$S_1 := 20\ ft$ $S_2 := 12\ ft$ $S_3 := 4\ ft$

$$D_{pllunit} := \frac{2}{n_{pile}} + \frac{1 \cdot (12.67\ ft + 2.67\ ft) \cdot 20\ ft}{2 \cdot \left(S_1^{\ 2} + S_2^{\ 2} + S_3^{\ 2}\right)} = 60.7262\%$$

%age of single lane or truck carried by maximum loaded piling

$$D_{pllupl} := \frac{2}{n_{pile}} - \frac{1 \cdot (12.67\ ft + 2.67\ ft) \cdot 20\ ft}{2 \cdot \left(S_1^{\ 2} + S_2^{\ 2} + S_3^{\ 2}\right)} = 5.9405\%$$

%age of single lane or truck carried by minimum loaded piling - note percentage is > 1, therefor no uplift is anticipated due to live load and certainly no uplift due to total load

$$R_{pALL} := D_{pllunit} \cdot \left(\max\left(R_{ahs20}, R_{anmtt}\right) + R_{LaneAbut}\right) = 42.2038\ kip$$

Abutment Reaction due to Live Load Unfactored Undistributed Dynamic Load Allowance NOT applied per AASHTO LRFD 3.6.2.1-1 for components entirely below ground level.

$\gamma_{LL} := 1.75$ $\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

AASHTO LRFD - Tables 3.4.1-1 & 3.4.1-2

$$P_{UAstr1} := \gamma_{LL} \cdot R_{pALL} + \gamma_{DC} \cdot R_{pDC} + \gamma_{DW} \cdot R_{pDW} = 178.17\ kip$$

Factored Maximum Strength I axial load applied to a single piling

$$P_{UsvcI} := R_{pALL} + R_{pDC} + R_{pDW} = 124.05\ kip$$

Unfactored Maximum axial load applied to a single piling

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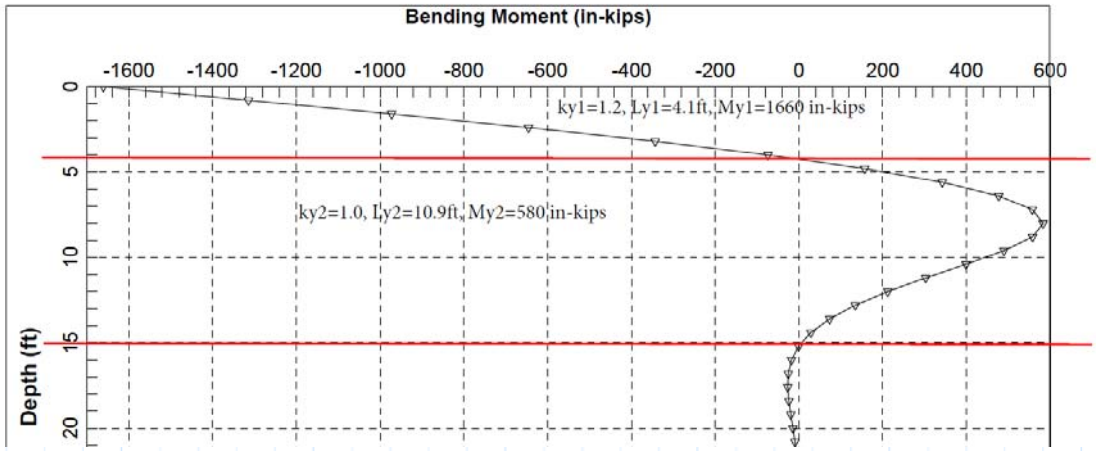
L-Pile Output Y-Y axis

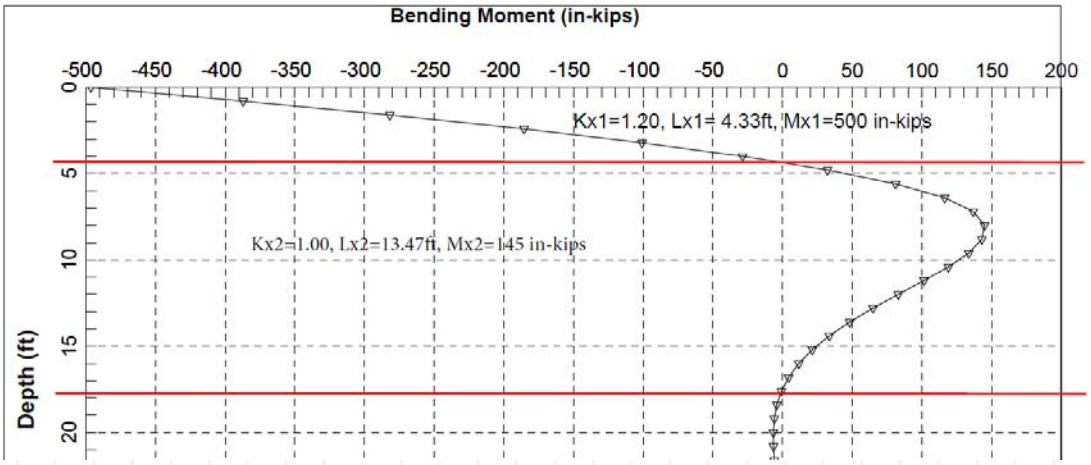


Induced transverse Movement of abutment into fill with HP12x74 strong axis - passive pressure acting at the friction angle from soil to concrete

$\delta_f := 24\ ^\circ$

$\delta_{pilex} := \tan(\text{Skew} - \delta_f) \cdot \delta_{pile} = 0.0656\ \text{in}$

L-Pile Output X-X axis



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Check Strong Axis Flexure:

$L_{x1} := 4.33\ ft$

$M_{ux1} := 500\ in \cdot kip$

$C_b := 1$

$k_{x1} := 1.2$

$k_{x2} := 1.0$

$L_{x2} := 13.47\ ft$

$M_{ux2} := 145\ in \cdot kip$

$Z_{yy} := 46.6\ in^3$ *For HP plastic section modulus weak axis*

$S_{yy} := 30.4\ in^3$ *For HP elastic section modulus weak axis*

$S_{xx} := 93.8\ in^3$ *For HP elastic section modulus strong axis*

$$\lambda_f := \frac{b_{fpile}}{2 \cdot t_{fpile}} = 10.0123$$

$\varphi_{fc} := 1.0$

$\varphi_{cm} := 0.70$

$\varphi_{ct} := 0.50$

Strength Reduction factors (resistance factors) taken from AASHTO 6.5.4.2

Local buckling - flange slenderness ratio is > that the compact slenderness ratio:

$$\lambda_{pflb} := 0.38 \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}} = 9.1516$$ *AASHTO 6.10.8.2.2-4*

$R_b := 1.0$ $R_h := 1.0$

$$\lambda_{rflb} := 0.56 \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp} \cdot 0.7}} = 16.1196$$ *AASHTO 6.10.8.2.2-5*

$$F_{nclb} := min\left(\left(\left(1 - \left(1 - \frac{0.7 \cdot F_{Yhp}}{R_h \cdot F_{Yhp}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pflb}}{\lambda_{rflb} - \lambda_{pflb}}\right)\right) \cdot R_b \cdot R_h \cdot F_{Yhp}\right), F_{Yhp}\right) = 48.15\ ksi$$

Lateral Torsional buckling: *AASHTO 6.10.8.2.3*

$r_{thp} := 3.26\ in$

$$L_p := 1.0 \cdot r_{thp} \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}} = 6.54\ ft$$

$$L_r := \pi \cdot r_{thp} \cdot \sqrt{\frac{29000\ ksi}{0.7 \cdot F_{Yhp}}} = 24.57\ ft$$

For Zone 1 L1<Lp therefore Fnc=50ksi

$F_{ncltb1} := 50\ ksi$

$$F_{ncltb2} := C_b \cdot \left(1 - \left(1 - \frac{0.7 \cdot F_{Yhp}}{R_h \cdot F_{Yhp}}\right) \cdot \left(\frac{L_{x2} - L_p}{L_r - L_p}\right)\right) \cdot F_{Yhp} = 44.23\ ksi$$

$F_{cf1} := min\left(F_{ncltb1}, F_{nclb}\right) = 48.15\ ksi$

$F_{cf2} := min\left(F_{ncltb2}, F_{nclb}\right) = 44.23\ ksi$

$M_{nx1} := F_{cf1} \cdot S_{xx} = 4516.21\ in \cdot kip$

$M_{nx2} := F_{cf2} \cdot S_{xx} = 4149.24\ in \cdot kip$

$M_{rx1} := \varphi_{fc} \cdot M_{nx1} = 4516.21\ in \cdot kip$

$M_{rx2} := \varphi_{fc} \cdot M_{nx2} = 4149.24\ in \cdot kip$

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Check Weak Axis Flexure:

$L_{y1} := 4.10\ ft$ $\qquad$ $L_{y2} := 10.90\ ft$

$M_{uy1} := 1660\ in \cdot kip$ $\qquad$ $M_{uy2} := 580\ in \cdot kip$

$k_{y1} := 1.2$

$k_{y2} := 1.0$

$$\lambda_f := \frac{b_{fpile}}{2 \cdot t_{fpile}} = 10.0123$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}} = 9.1516$$ *AASHTO 6.12.2.2.1-4*

$$\lambda_{rf} := 0.83 \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}} = 19.989$$ *AASHTO 6.12.2.2.1-5*

$$M_{py} := Z_{yy} \cdot F_{Yhp} = 2330\ in \cdot kip$$

$$M_{ny} := min\left(\left(1 - \left(1 - \frac{S_{yy}}{Z_{yy}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}}}\right)\right) \cdot F_{Yhp} \cdot Z_{yy}, M_{py}\right) = 2265.67\ in \cdot kip$$ *AASHTO 6.12.2.2.1-2*

$$M_{ry} := \varphi_{fc} \cdot M_{ny} = 2265.67\ in \cdot kip$$

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Strong Axis Axial Compression

$$P_{ex1} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{y1} \cdot \frac{L_{x1}}{r_{xxpile}}\right)^2} \cdot A_{pile} = 41907.84\ kip$$

Elastic Flexural Buckling Resistance AASHTO 6.9.4.1.2

$$P_{ex2} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{y2} \cdot \frac{L_{x2}}{r_{xxpile}}\right)^2} \cdot A_{pile} = 6235.89\ kip$$

$$Q := 1.0$$

$$P_o := Q \cdot A_{pile} \cdot F_{Yhp} = 1090\ kip$$

$$\frac{P_{ex1}}{P_o} = 38.4476 \qquad \frac{P_{ex2}}{P_o} = 5.721$$

Pe/Po > 0.44 for both zones 1 & 2

$$P_{nx1} := \left(0.658^{\left(\frac{P_o}{P_{ex1}}\right)}\right) \cdot P_o = 1078.2\ kip \qquad P_{nx2} := \left(0.658^{\left(\frac{P_o}{P_{ex2}}\right)}\right) \cdot P_o = 1013.1\ kip$$

$$P_{rx1} := \varphi_{cm} \cdot P_{nx1} = 754.74\ kip \qquad P_{rx2} := \varphi_{cm} \cdot P_{nx2} = 709.17\ kip$$

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Weak Axis Axial Compression

$$P_{ey1} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{x1} \cdot \frac{L_{y1}}{r_{yypile}}\right)^2} \cdot A_{pile} = 15262.56\ kip$$

Elastic Flexural Buckling Resistance AASHTO 6.9.4.1.2

$$P_{ey2} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{x2} \cdot \frac{L_{y2}}{r_{yypile}}\right)^2} \cdot A_{pile} = 3109.6\ kip$$

$$Q := 1.0$$

$$P_o := Q \cdot A_{pile} \cdot F_{Yhp} = 1090\ kip$$

$$\frac{P_{ey1}}{P_o} = 14.0023 \qquad \frac{P_{ey2}}{P_o} = 2.8528$$

Pe/Po > 0.44 for both zones 1 & 2

$$P_{ny1} := \left(0.658^{\left(\frac{P_o}{P_{ey1}}\right)}\right) \cdot P_o = 1057.9\ kip \qquad P_{ny2} := \left(0.658^{\left(\frac{P_o}{P_{ey2}}\right)}\right) \cdot P_o = 941.26\ kip$$

$$P_{ry1} := \varphi_{cm} \cdot P_{ny1} = 740.53\ kip \qquad P_{ry2} := \varphi_{cm} \cdot P_{ny2} = 658.88\ kip$$

$$P_{r1} := min\left(P_{rx1}, P_{ry1}\right) = 740.53\ kip$$

$$P_{r2} := min\left(P_{rx2}, P_{ry2}\right) = 658.88\ kip$$

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$$\frac{P_{UAstr1}}{P_{r1}} = 0.2406 \qquad \frac{P_{UAstr1}}{P_{r2}} = 0.2704$$ *Pu/Pr >0.2*

Axial Compression in the fully braced lower zone: when Pe tends toward infinity as L tends toward zero, say L=0 for lower zone

$$P_{n3} := P_o = 1090 \text{ kip}$$
$$P_{r3} := \varphi_{ct} \cdot P_o = 545 \text{ kip}$$

Check Combined Axial Compression and Flexure per AASHTO 6.9.2.2-2

$$\frac{P_{UAstr1}}{P_{r1}} + \frac{8}{9} \cdot \left(\frac{M_{ux1}}{M_{rx1}} + \frac{M_{uy1}}{M_{ry}} \right) = 0.9903$$ *In the Upper Zone < 1 therefor okay*

$$\frac{P_{UAstr1}}{P_{r2}} + \frac{8}{9} \left(\frac{M_{ux2}}{M_{rx2}} + \frac{M_{uy2}}{M_{ry}} \right) = 0.5295$$ *In the middle Zone < 1 therefor okay*

$$\frac{P_{UAstr1}}{P_{r3}} = 0.3269$$ *In the lower Zone < 1 therefor okay*

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Check Passive Pressure behind abutment wall to design Reinforcing

REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

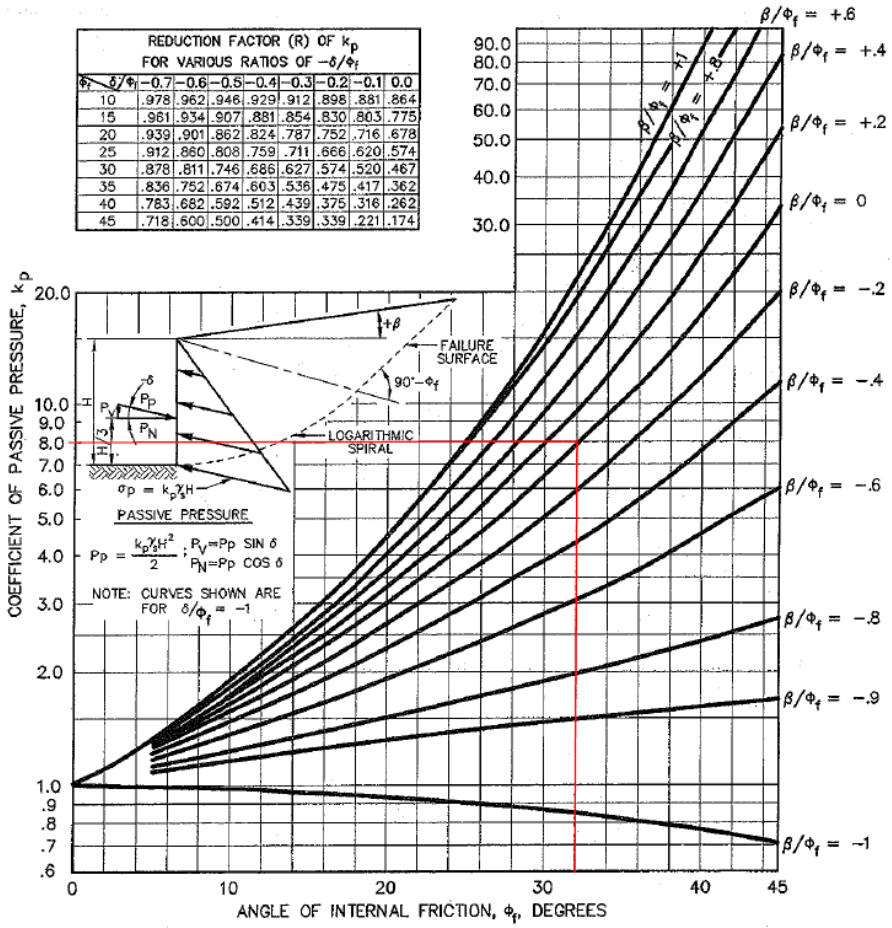

Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

$\delta_f := 24\ °$ $\qquad \varphi_{bf} := 32\ °$ $\qquad \frac{\delta_f}{\varphi_{bf}} = 0.75$ $\qquad$ *Use 0.7 for the reduction factor*

$$R := 0.878 - \frac{\varphi_{bf} - 30\ °}{35\ ° - 30\ °} \cdot (0.878 - 0.836) = 0.8612$$

$k_p := R \cdot 8.0 = 6.8896$ $\qquad \gamma_{bf} := 130$ pcf $\qquad H1 := 8.5$ ft

$H2 := 4.7$ ft

$p_1 := H1 \cdot \gamma_{bf} \cdot k_p = 7613.01$ psf $\qquad p_2 := H2 \cdot \gamma_{bf} \cdot k_p = 4209.55$ psf

$$\omega_{lower} := \frac{p_1 + p_2}{2} \cdot (H1 - H2) = 22.46 \text{ klf}$$

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$W_{lng} := 3.5\ ft$

$$M_{uAlower} := \frac{\omega_{lower} \cdot \left(\frac{S_1 - S_2}{\cos(Skew)}\right)^2}{10} = 2300.2\ in \cdot kip$$

moment in concrete abutment due to passive pressure

$$M_{uAlowing} := \frac{\omega_{lower} \cdot W_{lng}^2}{2} = 1651.02\ in \cdot kip$$

moment in wings due to passive pressure

$f'_c := 4\ ksi$ $\quad t_{abut} := 36\ in$

$f_y := 60\ ksi$ $\quad A_5 := 0.31\ in^2$ *Area of a single #5 bar*

$\varepsilon_{cmax} := 0.003\ \frac{in}{in}$ $\quad A_6 := 0.44\ in^2$ *Area of a single #6 bar*

$\varepsilon_{smintc} := -0.005\ \frac{in}{in}$ $\quad A_7 := 0.60\ in^2$ *Area of a single #7 bar*

$C_{lear} := 3\ in$ $\quad \varphi_{tc} := 0.90$ *Tension controlled resistance factor for reinforced concrete sections 5.5.4.2.1*

$E_c := 1820 \cdot \sqrt{f'_c \cdot ksi} = 3640\ ksi$ *C5.4.2.4*

$E_s := 29000\ ksi$ *5.4.3.2*

$n := \frac{E_s}{E_c} = 7.967$ $\quad A_{ps} := 0\ in^2$ $\quad f_{pu} := 0\ ksi$

$\gamma_{svc} := 1.00$ $\quad A'_s := 0\ in^2$ $\quad f'_s := 0\ ksi$

$b_{low} := H1 - H2 = 45.6\ in$ $\quad \gamma_{EHP} := 1.1$ *AASHTO does not provide a load factor for passive pressure as it is usually taken to be a resistance and not a load, for that reason based on engineering judgement and the fact that we have used the log spiral method we will assign a load factor of 1.1 to passive earth pressure*

$n_5 := 7$ *Number of #5 bars*

$n_6 := 0$ *Number of #6 bars*

$n_7 := 0$ *Number of #7 bars*

$A_{s1} := n_5 \cdot A_5 + n_6 \cdot A_6 + n_7 \cdot A_7 = 2.17\ in^2$ *Area of Horizontal Steel in Lower section of Abutment*

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c < 4\ ksi \\ 0.85 - (f'_c - 4\ ksi) \cdot \frac{0.05}{ksi} & \text{else} \end{cases} = 0.85$$

$\varphi_{rebar} := 0.625\ in$ $\quad \varphi_{vertbar} := 1\ in$

$d_{s1} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} - \varphi_{vertbar} = 31.6875\ in$

$f_s := f_y = 60\ ksi$ *assuming tensile controlled setion*

Flexural Capacity of Lower section of abutment:

$k := 0.28$

Assume fs=fy, rectangular distribution, and neglect compression reinforcing

$d_p := 1\ in$ *dp set to 1in although Aps=0 in^2 as there is no prestressing steel*

$$c_{btm} := \frac{A_{ps} \cdot f_{pu} + A_{s1} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{low} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 0.988\ in$$

$\varepsilon_{cmax} = 0.003\ \frac{in}{in}$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{btm}}\right) \cdot \langle c_{btm} - d_{s1} \rangle = -0.0932\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_{n1} := A_{s1} \cdot f_s \cdot \left(d_{s1} - \frac{\beta_1 \cdot c_{btm}}{2}\right) = 4071.04\ in \cdot kip$ *Abutment wall lower section moment nominal capacity*

$M_{r1} := \varphi_{tc} \cdot M_{n1} = 3663.9383\ in \cdot kip$ *Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay*

$M_{str11} := \gamma_{EHP} \cdot \max\langle M_{uAlower}, M_{uAlowing} \rangle = 2530.22\ in \cdot kip$

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Check Minimum Reinforcing
requirements: per section 5.7.3.3.2

$S_c := \frac{\left(t_{abut}^{\ 2} \cdot b_{low}\right)}{6} = 9849.6\ in^3$ *Section modulus of Lower strip of abutment wall (to construction joint)*

$\gamma_1 := 1.6$ *for other than precast segmental structures*

$\gamma_2 := 1.0$ *for unbonded tendons (no prestressing force applied gamma 2x fcpe = 0*

$\gamma_3 := 0.67$ *for A615 grade 60 reinforcing per 5.7.3.3.2*

$f_{cpe} := 0\ ksi$ *there is no prestressing force*

$S_{nc} := S_c$ *non composite section is the same as the composite section*

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.48\ ksi$ *modulus of rupture per 5.4.2.6*

$M_{dnc} := 0\ ft \cdot kip$ *all load is composite (ie there is no non composite section, or rather there is no composite section)*

$$M_{cr1} := \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}\right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right) = 5068.21\ in \cdot kip$$

$$M_{min1} := min\left(M_{cr1}, \frac{4}{3} \cdot M_{str11}\right) = 3373.62\ in \cdot kip$$

$M_{r1} = 3663.94\ in \cdot kip$

Resisting moment is > Mmin1 therefor minimum reinforcement is adequate per section 5.7.3.3.2

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Using (7) number 5 bars in the lower 3.8 feet
of abutment gives us a spacing of 6" centers

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc11} := \gamma_{svc} \cdot \max\left(M_{uAlower}, M_{uAlowing}\right) = 2300.196 \text{ in}\cdot\text{kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{sb1t} := A_{s1} \cdot n = 17.2885 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr1} := 4.5473 \text{ in}$$ *distance to cracked neutral axis (assumed)*

$$y_{sb} := C_{lear} + \frac{\varphi_{rebar}}{2} + \varphi_{vertbar} = 4.3125 \text{ in}$$

$$y_{cnc1} := t_{abut} - \frac{c_{cr1}}{2} = 33.7264 \text{ in}$$

$$A_{cnc1} := c_{cr1} \cdot b_{low} = 207.3569 \text{ in}^2$$

$$y_{bar1} := \frac{A_{cnc1} \cdot y_{cnc1} + A_{sb1t} \cdot y_{sb}}{A_{cnc1} + A_{sb1t}} = 31.4627 \text{ in}$$

$$c_{cr1} := t_{abut} - y_{bar1} = 4.5373 \text{ in}$$

$$I_{cr1} := \frac{\left(c_{cr1}^{\ 3} \cdot b_{low}\right)}{12} + A_{cnc1} \cdot \left(y_{cnc1} - y_{bar1}\right)^2 + A_{sb1t} \cdot \left(y_{sb} - y_{bar1}\right)^2 = 14161.38 \text{ in}^4$$

$$S_{cr1s} := \frac{I_{cr1}}{\left(y_{bar1} - y_{sb}\right) \cdot n} = 65.47 \text{ in}^3 \qquad S_{cr1c} := \frac{I_{cr1}}{t_{abut} - y_{bar1}} = 3121.1 \text{ in}^3$$

$$f_{ss1} := \frac{M_{svc11}}{S_{cr1s}} = 35.13 \text{ ksi}$$

$$f_{cnc1} := \frac{M_{svc11}}{S_{cr1c}} = 736.98 \text{ psi}$$

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$\gamma_e := 1.0$ *Exposure factor for Class I Exposure condition*

$d_{c1} := y_{sb} = 4.3125$ in

$h := t_{abut} = 36$ in

$\beta_{sb} := 1 + \frac{d_{c1}}{0.7 \cdot (h - d_{c1})} = 1.1944$ *5.7.3.4*

$s_{max1} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_{sb} \cdot f_{ss1}} - 2 \cdot d_{c1} = 8.06$ in *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 6 inches per bar << 8.06 in therefor okay*

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$p_3 := 0\ psf$ $\quad p_2 = 4209.55\ psf$

P3 is at top of abutment - P2 is at the construction joint and is from previous computations

$$\omega_{upper} := \frac{p_2 + p_3}{2} \cdot (H2) = 9.89\ klf$$

$$M_{uAupper} := \frac{\omega_{upper} \cdot \left(\frac{S}{\cos(Skew)}\right)^2}{10} = 1745.03\ in \cdot kip$$

moment in concrete abutment due to passive pressure Note Girder spacing used in lieu of pile spacing at the upper section

$$M_{uAupwing} := \frac{\omega_{upper} \cdot W_{lng}^2}{2} = 727.09\ in \cdot kip$$

moment in wings due to passive pressure

$f'_c := 4\ ksi$ $\quad t_{abut} := 36\ in$

$f_y := 60\ ksi$ $\quad A_5 := 0.31\ in^2$ *Area of a single #5 bar*

$\varepsilon_{cmax} := 0.003\ \frac{in}{in}$ $\quad A_6 := 0.44\ in^2$ *Area of a single #6 bar*

$\varepsilon_{smintc} := -0.005\ \frac{in}{in}$ $\quad A_7 := 0.60\ in^2$ *Area of a single #7 bar*

$C_{lear} := 3\ in$ $\quad \varphi_{tc} := 0.90$

Tension controlled resistance factor for reinforced concrete sections 5.5.4.2.1

$E_c := 1820 \cdot \sqrt{f'_c \cdot ksi} = 3640\ ksi$ *C5.4.2.4*

$E_s := 29000\ ksi$ *5.4.3.2*

$n := \frac{E_s}{E_c} = 7.967$ $\quad A_{ps} := 0\ in^2$ $\quad f_{pu} := 0\ ksi$

$\gamma_{svc} := 1.00$ $\quad A'_s := 0\ in^2$ $\quad f'_s := 0\ ksi$

$b_{up} := H2 = 56.4\ in$ $\quad \gamma_{EHP} := 1.1$

AASHTO does not provide a load factor for passive pressure as it is usually taken to be a resistance and not a load, for that reason based on engineering judgement and the fact that we have used the log spiral method we will assign a load factor of 1.1 to passive earth pressure

$n_{5U} := 6$ *Number of #5 bars*

$n_{6U} := 0$ *Number of #6 bars*

$n_{7U} := 0$ *Number of #7 bars*

$A_{s2} := n_{5U} \cdot A_5 + n_{6U} \cdot A_6 + n_{7U} \cdot A_7 = 1.86\ in^2$ *Area of Horizontal Steel in Upper section of Abutment*

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c < 4\ ksi \\ 0.85 - (f'_c - 4\ ksi) \cdot \frac{0.05}{ksi} & \text{else} \end{cases} = 0.85$$

$\varphi_{rebar} := 0.625\ in$

$\varphi_{vertbar} := 1\ in$

$d_{s2} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} - \varphi_{vertbar} = 31.6875\ in$

$f_s := f_y = 60\ ksi$ *assuming tensile controlled setion*

Flexural Capacity of Upper section of abutment:

$k := 0.28$

Assume fs=fy, rectangular distribution, and neglect compression reinforcing

$d_p := 1\ in$ *dp set to 1in although Aps=0 in^2 as there is no prestressing steel*

$$c_{top} := \frac{A_{ps} \cdot f_{pu} + A_{s2} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{up} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 0.685\ in$$

$\varepsilon_{cmax} = 0.003\ \frac{in}{in}$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{top}}\right) \cdot \left(c_{top} - d_{s2}\right) = -0.1358\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_{n2} := A_{s2} \cdot f_s \cdot \left(d_{s2} - \frac{\beta_1 \cdot c_{top}}{2}\right) = 3503.85\ in \cdot kip$ *Abutment wall upper section moment nominal capacity*

$M_{r2} := \varphi_{tc} \cdot M_{n2} = 3153.4656\ in \cdot kip$ *Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay*

$M_{str12} := \gamma_{EHP} \cdot \max\left(M_{uAupper}, M_{uAupwing}\right) = 1919.53\ in \cdot kip$

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Check Minimum Reinforcing requirements: per section 5.7.3.3.2

$S_c := \frac{\left(t_{abut}^{2} \cdot b_{up}\right)}{6} = 12182.4\ in^3$ *Section modulus of upper strip of abutment wall (from construction joint to the top of the abutment)*

$\gamma_1 := 1.6$ *for other than precast segmental structures*

$\gamma_2 := 1.0$ *for unbonded tendons (no prestressing force applied gamma 2x fcpe = 0*

$\gamma_3 := 0.67$ *for A615 grade 60 reinforcing per 5.7.3.3.2*

$f_{cpe} := 0\ ksi$ *there is no prestressing force*

$S_{nc} := S_c$ *non composite section is the same as the composite section*

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.48\ ksi$ *modulus of rupture per 5.4.2.6*

$M_{dnc} := 0\ ft \cdot kip$ *all load is composite (ie there is no non composite section, or rather no composite section)*

$$M_{cr2} := \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right) = 6268.58\ in \cdot kip$$

$$M_{min2} := min\left(M_{cr2}, \frac{4}{3} \cdot M_{str12} \right) = 2559.37\ in \cdot kip$$

$M_{r2} = 3153.47\ in \cdot kip$ *Resisting moment is > Mmin2 therefor minimum reinforcement is adequate per section 5.7.3.3.2*

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Using (6) number 5 bars in the Upper 4.7 feet
of abutment gives us a spacing of 10" centers

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc12} := \gamma_{svc} \cdot \max\left(M_{uAupper}, M_{uAupwing}\right) = 1745.025 \text{ in}\cdot\text{kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{su1t} := A_{s2} \cdot n = 14.8187 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr2} := 3.8263 \text{ in}$$ *distance to cracked neutral axis (assumed)*

$$y_{su} := C_{lear} + \frac{\varphi_{rebar}}{2} + \varphi_{vertbar} = 4.3125 \text{ in}$$

$$y_{cnc2} := t_{abut} - \frac{c_{cr2}}{2} = 34.0869 \text{ in}$$

$$A_{cnc2} := c_{cr2} \cdot b_{up} = 215.8033 \text{ in}^2$$

$$y_{bar2} := \frac{A_{cnc2} \cdot y_{cnc2} + A_{su1t} \cdot y_{su}}{A_{cnc2} + A_{su1t}} = 32.1737 \text{ in}$$

$$c_{cr2} := t_{abut} - y_{bar2} = 3.8263 \text{ in}$$

$$I_{cr2} := \frac{\left(c_{cr2}^{\ 3} \cdot b_{up}\right)}{12} + A_{cnc2} \cdot \left(y_{cnc2} - y_{bar2}\right)^2 + A_{su1t} \cdot \left(y_{su} - y_{bar2}\right)^2 = 12556.11 \text{ in}^4$$

$$S_{cr2s} := \frac{I_{cr2}}{\left(y_{bar2} - y_{su}\right) \cdot n} = 56.57 \text{ in}^3 \qquad S_{cr2c} := \frac{I_{cr2}}{t_{abut} - y_{bar2}} = 3281.52 \text{ in}^3$$

$$f_{ss2} := \frac{M_{svc12}}{S_{cr2s}} = 30.85 \text{ ksi}$$

$$f_{cnc2} := \frac{M_{svc12}}{S_{cr2c}} = 531.77 \text{ psi}$$

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$\gamma_e := 1.0$ *Exposure factor for Class I Exposure condition*

$d_{c2} := y_{su} = 4.3125\ \text{in}$

$h := t_{abut} = 36\ \text{in}$

$\beta_{su} := 1 + \frac{d_{c2}}{0.7 \cdot \langle h - d_{c2} \rangle} = 1.1944$ *5.7.3.4*

$s_{max2} := \frac{700 \cdot \gamma_e \cdot \frac{\text{kip}}{\text{in}}}{\beta_{su} \cdot f_{ss2}} - 2 \cdot d_{c2} = 10.37\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 10 inches per bar <10.37 in therefor okay*

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Vertical Reinforcement design:

$b_w := 126\ in$

$M_{LLI} := 961.329\ ft \cdot kip$ *From RISA output*

$M_{SIDL} := 173.539\ ft \cdot kip$ *From RISA output 77% of load from DW 23% of load from DC*

$M_{str13} := \gamma_{LL} \cdot M_{LLI} + \langle 0.23 \cdot \gamma_{DC} + 0.77 \cdot \gamma_{DW} \rangle \cdot M_{SIDL} = 23191.8691\ in \cdot kip$

$f'_c = 4\ ksi$ $t_{abut} = 36\ in$

$f_y = 60\ ksi$ $A_5 := 0.31\ in^2$ *Area of a single #5 bar*

$\varepsilon_{cmax} := 0.003\ \frac{in}{in}$ $A_6 := 0.44\ in^2$ *Area of a single #6 bar*

$\varepsilon_{smintc} := -0.005\ \frac{in}{in}$ $A_7 := 0.60\ in^2$ *Area of a single #7 bar*

$A_8 := 0.79\ in^2$ *Area of a single #8 bar*

$C_{lear} := 3\ in$ $\varphi_{tc} := 0.90$ *Tension controlled resistance factor for reinforced concrete sections 5.5.4.2.1*

$E_c = 3640\ ksi$ *C5.4.2.4*

$E_s := 29000\ ksi$ *5.4.3.2*

$n := \frac{E_s}{E_c} = 7.967$ $A_{ps} = 0\ in^2$ $f_{pu} = 0\ ksi$

$\gamma_{svc} := 1.00$ $A'_s = 0\ in^2$ $f'_s = 0\ ksi$

$b_{up} := H2 = 56.4\ in$

$n_{5V} := 0$ *Number of #5 bars* $n_{7V} := 11$ *Number of #7 bars*

$n_{6V} := 0$ *Number of #6 bars* $n_{8V} := 11$ *Number of #8 bars*

$A_{s3} := n_{5V} \cdot A_5 + n_{6V} \cdot A_6 + n_{7V} \cdot A_7 + n_{8V} \cdot A_8 = 15.29\ in^2$ *Area of Vertical Steel in Abutment*

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c < 4\ ksi \\ 0.85 - \langle f'_c - 4\ ksi \rangle \cdot \frac{0.05}{ksi} & \text{else} \end{cases} = 0.85$$

$\varphi_{rebar} := 1\ in$

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$d_{s3} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} = 32.5\ in$

$f_s := f_y = 60\ ksi$ *assuming tensile controlled setion*

Flexural Capacity of abutment due to vertical reinforcing:

$k = 0.28$

Assume fs=fy, rectangular distribution, and neglect compression reinforcing

$d_p = 1\ in$ *dp set to 1in although Aps=0 in^2 as there is no prestressing steel*

$$c_{vert} := \frac{A_{ps} \cdot f_{pu} + A_{s3} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 2.519\ in$$

$\varepsilon_{cmax} = 0.003\ \frac{in}{in}$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{vert}}\right) \cdot \left(c_{vert} - d_{s3}\right) = -0.0357\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_{n3} := A_{s3} \cdot f_s \cdot \left(d_{s3} - \frac{\beta_1 \cdot c_{vert}}{2}\right) = 28833.21\ in \cdot kip$ *Abutment wall moment nominal capacity (Vertical Bars)*

$M_{r3} := \varphi_{tc} \cdot M_{n3} = 25949.8925\ in \cdot kip$ *Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay*

$M_{str13} = 23191.87\ in \cdot kip$

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Check Minimum Reinforcing
requirements: per section 5.7.3.3.2

$S_{c3} := \frac{\left(t_{abut}^{2} \cdot b_w\right)}{6} = 27216\ in^3$ *Section modulus of Lower strip of abutment wall (to construction joint)*

$\gamma_1 := 1.6$ *for other than precast segmental structures*

$\gamma_2 := 1.0$ *for unbonded tendons (no prestressing force applied gamma 2x fcpe = 0*

$\gamma_3 := 0.67$ *for A615 grade 60 reinforcing per 5.7.3.3.2*

$f_{cpe} := 0\ ksi$ *there is no prestressing force*

$S_{nc} := S_c$ *non composite section is the same as the composite section*

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.48\ ksi$ *modulus of rupture per 5.4.2.6*

$M_{dnc} := 0\ ft \cdot kip$ *all load is composite (ie there is no non composite section, or rather no composite section)*

$$M_{cr3} := \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}\right) \cdot S_{c3} - M_{dnc} \cdot \left(\frac{S_{c3}}{S_{nc}} - 1 \right) \right) = 14004.26\ in \cdot kip$$

$$M_{min3} := min\left(M_{cr3}, \frac{4}{3} \cdot M_{str13}\right) = 14004.26\ in \cdot kip$$

$M_{r3} = 25949.89\ in \cdot kip$ *Resisting moment is > Mmin3 therefor minimum reinforcement is adequate per section 5.7.3.3.2*

Using number 8 bars at 12" with number 7 bars at 12" between
#8's vertical in far face of the abutment spacing = 6" max

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc13} := \gamma_{svc} \cdot M_{LLI} + \gamma_{svc} \cdot M_{SIDL} = 13618.42 \text{ in}\cdot\text{kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{svt} := A_{s3} \cdot n = 121.8159 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr3} := 7.0192 \text{ in}$$ *distance to cracked neutral axis (assumed)*

$$y_{sv} := C_{lear} + \frac{\varphi_{rebar}}{2} = 3.5 \text{ in}$$

$$y_{cnc3} := t_{abut} - \frac{c_{cr3}}{2} = 32.4904 \text{ in}$$

$$A_{cnc3} := c_{cr3} \cdot b_w = 884.4192 \text{ in}^2$$

$$y_{bar3} := \frac{A_{cnc3} \cdot y_{cnc3} + A_{svt} \cdot y_{sv}}{A_{cnc3} + A_{svt}} = 28.9808 \text{ in}$$

$$c_{cr3} := t_{abut} - y_{bar3} = 7.0192 \text{ in}$$

$$I_{cr3} := \frac{\left(c_{cr3}^{\ 3} \cdot b_w\right)}{12} + A_{cnc3} \cdot \left(y_{cnc3} - y_{bar3}\right)^2 + A_{svt} \cdot \left(y_{sv} - y_{bar3}\right)^2 = 93616.46 \text{ in}^4$$

$$S_{cr3s} := \frac{I_{cr3}}{\left(y_{bar3} - y_{sv}\right) \cdot n} = 461.15 \text{ in}^3 \qquad S_{cr3c} := \frac{I_{cr3}}{t_{abut} - y_{bar3}} = 13337.18 \text{ in}^3$$

$$f_{ss3} := \frac{M_{svc13}}{S_{cr3s}} = 29.53 \text{ ksi}$$

$$f_{cnc3} := \frac{M_{svc13}}{S_{cr3c}} = 1021.09 \text{ psi}$$

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$\gamma_e := 1.0$ *Exposure factor for Class I Exposure condition*

$d_{c3} := y_{sv} = 3.5\ \text{in}$

$h := t_{abut} = 36\ \text{in}$

$\beta_{sv} := 1 + \frac{d_{c3}}{0.7 \cdot (h - d_{c3})} = 1.1538$ *5.7.3.4*

$s_{max3} := \frac{700 \cdot \gamma_e \cdot \frac{\text{kip}}{\text{in}}}{\beta_{sv} \cdot f_{ss3}} - 2 \cdot d_{c3} = 13.54\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 6 inches per bar << 13.54 in therefor okay*

Envelope Member Section Forces

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[...	LC	y-y Mom...	LC	z-z Moment[k-ft]	LC
341	M35	1	max	93.275	1	-6.727	1	0	1	0	1	0	1	0	1
342			min	12.579	1	-113.098	1	0	1	0	1	0	1	0	1
343		2	max	93.275	1	-6.727	1	0	1	0	1	0	1	240.332	1
344			min	12.579	1	-113.098	1	0	1	0	1	0	1	14.295	1
345		3	max	93.275	1	-6.727	1	0	1	0	1	0	1	480.664	1
346			min	12.579	1	-113.098	1	0	1	0	1	0	1	28.591	1
347		4	max	93.275	1	-6.727	1	0	1	0	1	0	1	720.997	1
348			min	12.579	1	-113.098	1	0	1	0	1	0	1	42.886	1
349		5	max	93.275	1	-6.727	1	0	1	0	1	0	1	961.329	1
350			min	12.579	1	-113.098	1	0	1	0	1	0	1	57.182	1
351	M36	1	max	93.275	1	113.098	1	0	1	0	1	0	1	0	1
352			min	12.579	1	6.727	1	0	1	0	1	0	1	0	1
353		2	max	93.275	1	113.098	1	0	1	0	1	0	1	-14.295	1
354			min	12.579	1	6.727	1	0	1	0	1	0	1	-240.332	1
355		3	max	93.275	1	113.098	1	0	1	0	1	0	1	-28.591	1
356			min	12.579	1	6.727	1	0	1	0	1	0	1	-480.664	1
357		4	max	93.275	1	113.098	1	0	1	0	1	0	1	-42.886	1
358			min	12.579	1	6.727	1	0	1	0	1	0	1	-720.997	1
359		5	max	93.275	1	113.098	1	0	1	0	1	0	1	-57.182	1
360			min	12.579	1	6.727	1	0	1	0	1	0	1	-961.329	1

Members 35 & 36 are the abutments modeled with a pinned support at the piling

Member Section Forces (By Item)

	LC	Member La...	Sec	Axial[k]	y She...	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
171	1	M35	1	17.551	-20.416	0	0	0	0
172			2	17.551	-20.416	0	0	0	43.385
173			3	17.551	-20.416	0	0	0	86.77
174			4	17.551	-20.416	0	0	0	130.155
175			5	17.551	-20.416	0	0	0	173.539
176	1	M36	1	17.551	20.416	0	0	0	0
177			2	17.551	20.416	0	0	0	-43.385
178			3	17.551	20.416	0	0	0	-86.77
179			4	17.551	20.416	0	0	0	-130.155
180			5	17.551	20.416	0	0	0	-173.539

Members 35 & 36 are the abutments modeled with a pinned support at the piling

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LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Eric Calderwood
Calderwood Engineering

Path to file locations: Y:\z_2014 Pin\106-br-14 Rockingham\
Name of input data file: 612x74 Weak way Abutment Piling Weak Axis.lpd
Name of output file: 612x74 Weak way Abutment Piling Weak Axis.lpo
Name of plot output file: 612x74 Weak way Abutment Piling Weak Axis.lpp
Name of runtime file: 612x74 Weak way Abutment Piling Weak Axis.lpr

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Time and Date of Analysis

Date: January 12, 2015 Time: 14:30:59

Problem Title

New LPILE Plus 5.0 Data File

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100

- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = .00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	12.13000000	186.0000	21.8000	29000000.
2	960.0000	12.13000000	186.0000	21.8000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 300.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 1140.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends 180.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.07523
2	300.00	.07523
3	300.00	.04201

4 1140.00 .04201

Shear Strength of Soils

Shear strength parameters with depth defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	32.00	------	------
2	300.000	.00000	32.00	------	------
3	300.000	.00000	35.00	------	------
4	1140.000	.00000	35.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Displacement and Slope (BC Type 5)
Deflection at pile head = .624 in
Slope at pile head = .000 in/in
Axial load at pile head = 178170.000 lbs

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Displacement and Slope (BC Type 5)
Specified deflection at pile head = .624000 in
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 178170.000 lbs

Depth Deflect. Moment Shear Slope Total Soil Res.
Es*h

X	y	M	V	S	Stress	p	F/L
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in	lbs/in
0.000	.624000	-1660761.	36290.5829	0.0000	62326.2425	0.0000	0.0000
9.600	.609812	-1314188.	35385.4472	-.0026473	51025.3547	-94.2799	1484.2059
19.200	.573171	-972303.	33987.8042	-.0046820	39877.3392	-196.9058	3297.9609
28.800	.519917	-645605.	31678.5595	-.0061218	29224.5343	-284.1868	5247.3610
38.400	.455633	-343133.	28546.6953	-.0070016	19361.6578	-368.2849	7759.6124
48.000	.385486	-73557.3047	24750.9102	-.0073724	10571.4576	-422.5037	10521.8777
57.600	.314082	157304.	20377.0392	-.0072979	13302.2458	-488.7194	14937.8350
67.200	.245366	342647.	15439.3158	-.0068530	19345.8077	-539.9730	21126.5680
76.800	.182504	477183.	10098.6390	-.0061235	23732.6797	-572.6681	30123.2243
86.400	.127795	557488.	4601.2957	-.0052027	26351.2553	-572.6118	43014.6681
96.000	.082612	583325.	-709.9411	-.0041876	27193.7320	-533.8959	62042.1795
105.600	.047394	558183.	-5434.7299	-.0031718	26373.8910	-450.4351	91238.4000
115.200	.021714	489829.	-8677.4411	-.0022392	24145.0365	-225.1297	99532.8000
124.800	.004403	399236.	-9995.4225	-.0014480	21191.0211	-49.4497	107827.
134.400	-.006088	302870.	-9879.3333	-.0008232	18048.7748	73.6350	116122.
144.000	-.011403	212368.	-8816.5309	-.0003647	15097.7462	147.7822	124416.
153.600	-.013090	134840.	-7238.5945	-5.5732E-05	12569.7390	180.9546	132710.
163.200	-.012473	73578.1014	-5490.6361	.0001297	10572.1357	183.2034	141005.
172.800	-.010599	28976.1126	-3820.0504	.0002210	9117.7752	164.8353	149299.
182.400	-.008230	-522.8613	-2380.3524	.0002463	8189.9850	135.1018	157594.
192.000	-.005870	-17569.2641	-1245.0056	.0002302	8745.8260	101.4288	165888.
201.600	-.003810	-25214.5049	-426.3522	.0001921	8995.1184	69.1240	174182.
211.200	-.002181	-26412.5224	104.3947	.0001462	9034.1828	41.4483	182477.
220.800	-.001003	-23710.2622	398.9871	.0001016	8946.0688	19.9251	190771.
230.400	-.000230	-19099.5254	517.5083	6.3503E-05	8795.7241	4.7668	199066.
240.000	.000217	-13991.3382	517.9335	3.4056E-05	8629.1587	-4.6782	207360.
249.600	.000424	-9271.7035	449.7596	1.3355E-05	8475.2631	-9.5247	215654.
259.200	.000473	-5401.6386	351.0775	2.9741E-07	8349.0699	-11.0341	223949.
268.800	.000430	-2532.0324	248.2155	-6.7626E-06	8255.4991	-10.3955	

232243.							
278.400	.000343	-612.7666	157.0462	-9.5611E-06	8192.9166	-8.5981	
240538.							
288.000	.000246	515.9619	85.1521	-9.6472E-06	8189.7600	-6.3798	
248832.							
297.600	.000158	1055.1555	34.2250	-8.2491E-06	8207.3418	-4.2300	
257126.							
307.200	8.78E-05	1201.3016	.4646701	-6.2411E-06	8212.1073	-2.8034	
306692.							
316.800	3.81E-05	1085.4273	-19.0536	-4.2062E-06	8208.3289	-1.2629	
318212.							
326.400	6.99E-06	849.8608	-26.2683	-2.4841E-06	8200.6476	-.2401644	
329732.							
336.000	-9.59E-06	589.5737	-25.7840	-1.2031E-06	8192.1603	.3410590	
341252.							
345.600	-1.61E-05	358.9238	-21.3057	-3.5910E-07	8184.6394	.5919278	
352772.							
355.200	-1.65E-05	181.7334	-15.4610	1.2202E-07	8178.8617	.6257206	
364292.							
364.800	-1.38E-05	61.6561	-9.8709	3.3861E-07	8174.9462	.5388751	
375812.							
374.400	-9.99E-06	-8.9461	-5.3499	3.8551E-07	8173.2275	.4029891	
387332.							
384.000	-6.36E-06	-42.3816	-2.1465	3.3984E-07	8174.3177	.2643880	
398852.							
393.600	-3.46E-06	-51.3220	-.1668691	2.5645E-07	8174.6093	.1480420	
410372.							
403.200	-1.44E-06	-46.4627	.8474342	1.6943E-07	8174.4508	.0632711	
421892.							
412.800	-2.10E-07	-35.6309	1.1967	9.6381E-08	8174.0976	.0094837	
433412.							
422.400	4.11E-07	-23.8166	1.1508	4.3480E-08	8173.7124	-.0190398	
444932.							
432.000	6.25E-07	-13.6845	.9168114	1.0109E-08	8173.3820	-.0297054	
456452.							
441.600	6.05E-07	-6.2484	.6326883	-7.6292E-09	8173.1395	-.0294869	
467972.							
451.200	4.78E-07	-1.5108	.3764859	-1.4534E-08	8172.9850	-.0238885	
479492.							
460.800	3.26E-07	1.0298	.1818241	-1.4962E-08	8172.9694	-.0166660	
491012.							
470.400	1.91E-07	2.0314	.0538337	-1.2238E-08	8173.0020	-.0099987	
502532.							
480.000	9.09E-08	2.1053	-.0175177	-8.5567E-09	8173.0044	-.0048662	
514052.							
489.600	2.67E-08	1.7243	-.0478966	-5.1488E-09	8172.9920	-.0014627	
525572.							
499.200	-7.98E-09	1.2033	-.0527745	-2.5436E-09	8172.9750	.0004465	
537092.							
508.800	-2.21E-08	.7197516	-.0445638	-8.3235E-10	8172.9592	.0012641	
548612.							
518.400	-2.40E-08	.3505163	-.0317854	1.2006E-10	8172.9472	.0013981	
560132.							
528.000	-1.98E-08	.1090619	-.0194110	5.2903E-10	8172.9393	.0011799	
571652.							
537.600	-1.38E-08	-.0239849	-.0097224	6.0474E-10	8172.9366	.0008386	
583172.							
547.200	-8.20E-09	-.0796764	-.0032579	5.1249E-10	8172.9384	.0005082	
594692.							
556.800	-3.96E-09	-.0882904	.0003830	3.6302E-10	8172.9387	.0002503	
606212.							
566.400	-1.23E-09	-.0735643	.0019657	2.1899E-10	8172.9382	7.9380E-05	
617732.							

576.000	2.40E-10	-.0512987	.0022711	1.0788E-10	8172.9375	-1.5747E-05	629252.
585.600	8.38E-10	-.0303280	.0019272	3.5240E-11	8172.9368	-5.5909E-05	640772.
595.200	9.17E-10	-.0144177	.0013598	-4.5783E-12	8172.9362	-6.2297E-05	652292.
604.800	7.50E-10	-.0042047	.0008119	-2.1150E-11	8172.9359	-5.1841E-05	663812.
614.400	5.11E-10	.0012434	.0003906	-2.3785E-11	8172.9358	-3.5931E-05	675332.
624.000	2.93E-10	.0033763	.0001175	-1.9674E-11	8172.9359	-2.0967E-05	686852.
633.600	1.33E-10	.0035667	-2.9584E-05	-1.3496E-11	8172.9359	-9.6764E-06	698372.
643.200	3.39E-11	.0028545	-8.8072E-05	-7.7819E-12	8172.9359	-2.5086E-06	709892.
652.800	-1.64E-11	.0019023	-9.4199E-05	-3.5489E-12	8172.9358	1.2322E-06	721412.
662.400	-3.42E-11	.0010580	-7.5746E-05	-9.1458E-13	8172.9358	2.6122E-06	732932.
672.000	-3.40E-11	.0004511	-5.0567E-05	4.2835E-13	8172.9358	2.6333E-06	744452.
681.600	-2.60E-11	8.5617E-05	-2.8104E-05	9.0600E-13	8172.9358	2.0467E-06	755972.
691.200	-1.66E-11	-9.1546E-05	-1.1924E-05	9.0073E-13	8172.9358	1.3241E-06	767492.
700.800	-8.70E-12	-.0001464	-2.1812E-06	6.8898E-13	8172.9358	7.0571E-07	779012.
710.400	-3.33E-12	-.0001358	2.5238E-06	4.3786E-13	8172.9358	2.7448E-07	790532.
720.000	-2.90E-13	-9.9448E-05	3.9575E-06	2.2854E-13	8172.9358	2.4205E-08	802052.
729.600	1.05E-12	-6.0580E-05	3.6446E-06	8.6134E-14	8172.9358	-8.9382E-08	813572.
739.200	1.36E-12	-2.9766E-05	2.6529E-06	5.7371E-15	8172.9358	-1.1724E-07	825092.
748.800	1.16E-12	-9.6648E-06	1.6029E-06	-2.9351E-14	8172.9358	-1.0151E-07	836612.
758.400	8.01E-13	1.1100E-06	7.7614E-07	-3.6964E-14	8172.9358	-7.0723E-08	848132.
768.000	4.55E-13	5.3635E-06	2.4104E-07	-3.1203E-14	8172.9358	-4.0756E-08	859652.
777.600	2.01E-13	5.8447E-06	-4.2322E-08	-2.1229E-14	8172.9358	-1.8277E-08	871172.
787.200	4.75E-14	4.6236E-06	-1.5103E-07	-1.1914E-14	8172.9358	-4.3710E-09	882692.
796.800	-2.73E-14	2.9856E-06	-1.5979E-07	-5.1426E-15	8172.9358	2.5462E-09	894212.
806.400	-5.12E-14	1.5731E-06	-1.2439E-07	-1.0859E-15	8172.9358	4.8305E-09	905732.
816.000	-4.82E-14	6.0109E-07	-7.9100E-08	8.4893E-16	8172.9358	4.6039E-09	917252.
825.600	-3.49E-14	5.1506E-08	-4.0795E-08	1.4297E-15	8172.9358	3.3764E-09	928772.
835.200	-2.07E-14	-1.8706E-07	-1.4840E-08	1.3090E-15	8172.9358	2.0309E-09	940292.
844.800	-9.77E-15	-2.3790E-07	-4.4377E-10	9.3087E-16	8172.9358	9.6828E-10	951812.
854.400	-2.86E-15	-1.9877E-07	5.5826E-09	5.4229E-16	8172.9358	2.8720E-10	963332.
864.000	6.46E-16	-1.3257E-07	6.6464E-09	2.4744E-16	8172.9358	-6.5578E-11	974852.
873.600	1.89E-15	-7.2005E-08	5.4001E-09	6.5393E-17	8172.9358	-1.9406E-10	

```
986372.
 883.200  1.90E-15 -2.9109E-08  3.5200E-09 -2.4586E-17   8172.9358 -1.9764E-10
997892.
 892.800  1.42E-15 -4.3380E-09  1.8563E-09 -5.4349E-17   8172.9358 -1.4896E-10
1009412.
 902.400  8.58E-16  6.7183E-09  7.0343E-10 -5.2231E-17   8172.9358 -9.1228E-11
1020932.
 912.000  4.14E-16  9.3465E-09  5.1917E-11 -3.7936E-17   8172.9358 -4.4503E-11
1032452.
 921.600  1.29E-16  7.8448E-09 -2.2928E-10 -2.2637E-17   8172.9358 -1.4079E-11
1043972.
 931.200 -2.08E-17  5.0218E-09 -2.8586E-10 -1.1188E-17   8172.9358  2.2912E-12
1055492.
 940.800 -8.53E-17  2.3947E-09 -2.2933E-10 -4.5880E-18   8172.9358  9.4854E-12
1067012.
 950.400 -1.09E-16  6.3438E-10 -1.2506E-10 -1.8925E-18   8172.9358  1.2238E-11
1078532.
 960.000 -1.22E-16      0.0000      0.0000 -1.3280E-18   8172.9358  1.3816E-11
545026.
```

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      .62400000 in
Computed slope at pile head      =     -.00030841
Maximum bending moment           =      -1660761. lbs-in
Maximum shear force              =    36290.58288 lbs
Depth of maximum bending moment  =        0.00000 in
Depth of maximum shear force     =        0.00000 in
Number of iterations             =              8
Number of zero deflection points =             11
```

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

```
Type 1 = Shear and Moment,          y = pile-head displacment in
Type 2 = Shear and Slope,           M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness,  V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment,     S = Pile-head Slope, radians
Type 5 = Deflection and Slope,      R = Rot. Stiffness of Pile-head in-lbs/rad
```

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
5	y= .624000	S= 0.000	178170.	.6240000	-1660761.	36290.5829

The analysis ended normally.

612x74 Strong way Abutment Piling Weak Axis.lpo

==========

LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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==========

This program is licensed to:

Eric Calderwood
Calderwood Engineering

Path to file locations: Y:\z_2014 Pin\106-br-14 Rockingham\
Name of input data file: 612x74 Strong way Abutment Piling Weak Axis.lpd
Name of output file: 612x74 Strong way Abutment Piling Weak Axis.lpo
Name of plot output file: 612x74 Strong way Abutment Piling Weak Axis.lpp
Name of runtime file: 612x74 Strong way Abutment Piling Weak Axis.lpr

---

Time and Date of Analysis

Date: January 12, 2015 Time: 14:28:03

Problem Title

New LPILE Plus 5.0 Data File

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100

- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = .00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	12.21500000	569.0000	21.8000	29000000.
2	960.0000	12.21500000	569.0000	21.8000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 300.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 1140.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends 180.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.07523
2	300.00	.07523
3	300.00	.04201

4 1140.00 .04201

Shear Strength of Soils

Shear strength parameters with depth defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	32.00	------	------
2	300.000	.00000	32.00	------	------
3	300.000	.00000	35.00	------	------
4	1140.000	.00000	35.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Displacement and Slope (BC Type 5)
Deflection at pile head = .066 in
Slope at pile head = .000 in/in
Axial load at pile head = 178170.000 lbs

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Displacement and Slope (BC Type 5)
Specified deflection at pile head = .065600 in
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 178170.000 lbs

Depth Deflect. Moment Shear Slope Total Soil Res.
Es*h

X	y	M	V	S	Stress	p	F/L
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in	lbs/in
0.000	.065600	-496341.	11603.6296	0.0000	13500.5359	0.0000	0.0000
9.600	.064214	-386993.	11125.7916	-.0002570	12326.8198	-49.7732	7441.1069
19.200	.060666	-281847.	10416.6667	-.0004515	11198.2070	-97.9643	15502.0949
28.800	.055545	-185448.	9323.7938	-.0005874	10163.4921	-129.7175	22419.5048
38.400	.049388	-100820.	8047.5216	-.0006707	9255.1159	-136.1725	26469.3672
48.000	.042667	-28641.6180	6808.3725	-.0007084	8480.3676	-121.9836	27446.0587
57.600	.035787	32323.6392	5591.6176	-.0007073	8519.8894	-131.5070	35277.6292
67.200	.029087	81137.0444	4256.7197	-.0006743	9043.8400	-146.5967	48383.8550
76.800	.022840	116359.	2795.2788	-.0006169	9421.9073	-157.8701	66355.2000
86.400	.017243	136917.	1393.9048	-.0005432	9642.5631	-134.0828	74649.6000
96.000	.012411	144980.	235.5983	-.0004612	9729.1189	-107.2311	82944.0000
105.600	.008389	143018.	-661.7910	-.0003774	9708.0508	-79.7251	91238.4000
115.200	.005165	133565.	-1301.5112	-.0002969	9606.5894	-53.5500	99532.8000
124.800	.002687	119044.	-1703.4305	-.0002235	9450.7285	-30.1832	107827.
134.400	.000874	101624.	-1899.0815	-.0001593	9263.7383	-10.5774	116122.
144.000	-.000371	83126.9490	-1926.7886	-.0001055	9065.1991	4.8051	124416.
153.600	-.001152	64990.3386	-1827.3023	-6.2443E-05	8870.5254	15.9212	132710.
163.200	-.001570	48256.3555	-1640.2146	-2.9501E-05	8690.9071	23.0554	141005.
172.800	-.001718	33599.1357	-1401.2917	-5.6897E-06	8533.5803	26.7202	149299.
182.400	-.001679	21371.0189	-1140.7413	1.0301E-05	8402.3268	27.5611	157594.
192.000	-.001520	11661.6655	-882.3442	1.9910E-05	8298.1091	26.2716	165888.
201.600	-.001297	4361.9026	-643.3133	2.4571E-05	8219.7553	23.5265	174182.
211.200	-.001049	-774.0028	-434.7144	2.5614E-05	8181.2437	19.9316	182477.
220.800	-.000805	-4072.2378	-262.2709	2.4205E-05	8216.6461	15.9941	190771.
230.400	-.000584	-5892.4050	-127.3858	2.1306E-05	8236.1833	12.1070	199066.
240.000	-.000396	-6590.9295	-28.2378	1.7675E-05	8243.6811	8.5488	207360.
249.600	-.000245	-6495.0341	39.1612	1.3868E-05	8242.6518	5.4926	215654.
259.200	-.000130	-5886.4755	80.0277	1.0266E-05	8236.1197	3.0212	223949.
268.800	-4.74E-05	-4993.6216	100.0329	7.1015E-06	8226.5360	1.1465	

232243.							
278.400	6.84E-06	-3990.1375	104.7136	4.4882E-06	8215.7649	-.1713401	
240538.							
288.000	3.88E-05	-2998.4736	99.0660	2.4553E-06	8205.1206	-1.0052	
248832.							
297.600	5.40E-05	-2096.4695	87.3009	9.7324E-07	8195.4387	-1.4458	
257126.							
307.200	5.75E-05	-1325.6252	71.5268	-2.2217E-08	8187.1647	-1.8405	
307443.							
316.800	5.36E-05	-723.0787	54.1518	-6.1817E-07	8180.6971	-1.7793	
318963.							
326.400	4.56E-05	-283.7968	38.0759	-9.1106E-07	8175.9820	-1.5698	
330483.							
336.000	3.61E-05	11.0947	24.3743	-9.9039E-07	8173.0549	-1.2847	
342003.							
345.600	2.66E-05	187.5770	13.5085	-9.3259E-07	8174.9492	-.9789921	
353523.							
355.200	1.82E-05	273.6488	5.4956	-7.9843E-07	8175.8731	-.6903754	
365043.							
364.800	1.13E-05	295.8233	.0626671	-6.3277E-07	8176.1111	-.4414793	
376563.							
374.400	6.01E-06	277.0166	-3.2219	-4.6614E-07	8175.9092	-.2428124	
388083.							
384.000	2.31E-06	235.5567	-4.8480	-3.1704E-07	8175.4642	-.0959506	
399603.							
393.600	-8.06E-08	185.0196	-5.2920	-1.9469E-07	8174.9217	.0034529	
411123.							
403.200	-1.43E-06	134.6166	-4.9726	-1.0171E-07	8174.3807	.0630884	
422643.							
412.800	-2.03E-06	89.8939	-4.2283	-3.6405E-08	8173.9007	.0919669	
434163.							
422.400	-2.13E-06	53.5574	-3.3118	5.3235E-09	8173.5107	.0989780	
445683.							
432.000	-1.93E-06	26.2894	-2.3952	2.8550E-08	8173.2180	.0919795	
457203.							
441.600	-1.58E-06	7.4721	-1.5825	3.8371E-08	8173.0160	.0773304	
468723.							
451.200	-1.19E-06	-4.2259	-.9244714	3.9315E-08	8172.9811	.0597596	
480243.							
460.800	-8.29E-07	-10.4123	-.4337994	3.5057E-08	8173.0475	.0424637	
491763.							
470.400	-5.21E-07	-12.6748	-.0987459	2.8342E-08	8173.0718	.0273391	
503283.							
480.000	-2.85E-07	-12.4051	.1057901	2.1046E-08	8173.0689	.0152726	
514803.							
489.600	-1.17E-07	-10.7156	.2099946	1.4320E-08	8173.0508	.0064366	
526323.							
499.200	-9.85E-09	-8.4222	.2435396	8.7533E-09	8173.0262	.0005519	
537843.							
508.800	5.07E-08	-6.0696	.2322731	4.5378E-09	8173.0009	-.0028991	
549363.							
518.400	7.73E-08	-3.9781	.1966864	1.6150E-09	8172.9785	-.0045148	
560883.							
528.000	8.17E-08	-2.2987	.1516417	-2.1089E-10	8172.9605	-.0048695	
572403.							
537.600	7.32E-08	-1.0659	.1068889	-1.1896E-09	8172.9472	-.0044539	
583923.							
547.200	5.88E-08	-.2424083	.0679955	-1.5702E-09	8172.9384	-.0036488	
595443.							
556.800	4.31E-08	.2450158	.0374078	-1.5694E-09	8172.9384	-.0027236	
606963.							
566.400	2.87E-08	.4811901	.0154608	-1.3582E-09	8172.9409	-.0018487	
618483.							

576.000	1.70E-08	.5465100	.0012320	-1.0592E-09	8172.9416	-.0011156	630003.
585.600	8.36E-09	.5084679	-.0068040	-7.5236E-10	8172.9412	-.0005585	641523.
595.200	2.56E-09	.4184477	-.0103191	-4.8272E-10	8172.9403	-.0001738	653043.
604.800	-9.11E-10	.3119930	-.0108508	-2.7024E-10	8172.9391	6.3034E-05	664563.
614.400	-2.63E-09	.2110370	-.0096579	-1.1810E-10	8172.9380	.0001855	676083.
624.000	-3.18E-09	.1269645	-.0076750	-1.9778E-11	8172.9371	.0002276	687603.
633.600	-3.01E-09	.0637442	-.0055290	3.5697E-11	8172.9365	.0002195	699123.
643.200	-2.49E-09	.0206852	-.0035899	6.0257E-11	8172.9360	.0001845	710643.
652.800	-1.86E-09	-.0053888	-.0020339	6.4707E-11	8172.9358	.0001397	722163.
662.400	-1.25E-09	-.0185871	-.0009049	5.7732E-11	8172.9360	9.5557E-05	733683.
672.000	-7.48E-10	-.0229604	-.0001675	4.5647E-11	8172.9360	5.8064E-05	745203.
681.600	-3.74E-10	-.0219596	.0002527	3.2580E-11	8172.9360	2.9474E-05	756723.
691.200	-1.22E-10	-.0182207	.0004412	2.0892E-11	8172.9360	9.8009E-06	768243.
700.800	2.72E-11	-.0135604	.0004776	1.1647E-11	8172.9359	-2.2097E-06	779763.
710.400	1.01E-10	-.0090902	.0004270	5.0579E-12	8172.9359	-8.3369E-06	791283.
720.000	1.24E-10	-.0053794	.0003371	8.4880E-13	8172.9358	-1.0396E-05	802803.
729.600	1.17E-10	-.0026212	.0002394	-1.4785E-12	8172.9358	-9.9621E-06	814323.
739.200	9.59E-11	-.0007786	.0001519	-2.4675E-12	8172.9358	-8.2523E-06	825843.
748.800	7.01E-11	.0003043	8.2983E-05	-2.6055E-12	8172.9358	-6.1116E-06	837363.
758.400	4.59E-11	.0008236	3.4164E-05	-2.2774E-12	8172.9358	-4.0591E-06	848883.
768.000	2.63E-11	.0009680	3.3488E-06	-1.7562E-12	8172.9358	-2.3608E-06	860403.
777.600	1.22E-11	.0008939	-1.3295E-05	-1.2146E-12	8172.9358	-1.1067E-06	871923.
787.200	3.02E-12	.0007169	-1.9942E-05	-7.4603E-13	8172.9358	-2.7799E-07	883443.
796.800	-2.14E-12	.0005136	-2.0319E-05	-3.8810E-13	8172.9358	1.9941E-07	894963.
806.400	-4.43E-12	.0003281	-1.7354E-05	-1.4326E-13	8172.9358	4.1836E-07	906483.
816.000	-4.89E-12	.0001809	-1.3101E-05	4.7980E-15	8172.9358	4.6757E-07	918003.
825.600	-4.34E-12	7.6548E-05	-8.8404E-06	7.9679E-14	8172.9358	4.2007E-07	929523.
835.200	-3.36E-12	1.0863E-05	-5.2432E-06	1.0511E-13	8172.9358	3.2934E-07	941043.
844.800	-2.32E-12	-2.4481E-05	-2.5572E-06	1.0115E-13	8172.9358	2.3024E-07	952563.
854.400	-1.42E-12	-3.8581E-05	-7.6856E-07	8.2801E-14	8172.9358	1.4238E-07	964083.
864.000	-7.31E-13	-3.9520E-05	2.7128E-07	6.0082E-14	8172.9358	7.4252E-08	975603.
873.600	-2.64E-13	-3.3578E-05	7.5810E-07	3.8819E-14	8172.9358	2.7168E-08	

```
987123.
 883.200  1.47E-14 -2.5098E-05  8.8118E-07  2.1750E-14   8172.9358 -1.5266E-09
998643.
 892.800  1.53E-13 -1.6733E-05  7.9637E-07  9.5822E-15   8172.9358 -1.6141E-08
1010163.
 902.400  1.99E-13 -9.8401E-06  6.1742E-07  1.8521E-15   8172.9358 -2.1142E-08
1021683.
 912.000  1.89E-13 -4.8854E-06  4.1832E-07 -2.4314E-15   8172.9358 -2.0336E-08
1033203.
 921.600  1.52E-13 -1.8000E-06  2.4133E-07 -4.3761E-15   8172.9358 -1.6538E-08
1044723.
 931.200  1.05E-13 -2.3694E-07  1.0652E-07 -4.9686E-15   8172.9358 -1.1545E-08
1056243.
 940.800  5.66E-14  2.6231E-07  2.0904E-08 -4.9612E-15   8172.9358 -6.2924E-09
1067763.
 950.400  9.68E-15  1.8138E-07 -1.4523E-08 -4.8321E-15   8172.9358 -1.0881E-09
1079283.
 960.000 -3.62E-14      0.0000      0.0000 -4.7794E-15   8172.9358  4.1137E-09
545402.
```

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      .06560000 in
Computed slope at pile head      =     -.00003181
Maximum bending moment           =  -496341.29333 lbs-in
Maximum shear force              =    11603.62962 lbs
Depth of maximum bending moment  =        0.00000 in
Depth of maximum shear force     =        0.00000 in
Number of iterations             =              5
Number of zero deflection points =              9
```

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

```
Type 1 = Shear and Moment,          y = pile-head displacment in
Type 2 = Shear and Slope,           M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness,  V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment,     S = Pile-head Slope, radians
Type 5 = Deflection and Slope,      R = Rot. Stiffness of Pile-head in-lbs/rad
```

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
5	y= .065600	S= 0.000	178170.	.0656000	-496341.	11603.6296

The analysis ended normally.