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

January, 2015

Revised to modify abutment reinforcing
and pile spacing to avoid power lines
1/29/15 - ETC

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Cover photo of Cavendish
Courtesy of Cold River Bridges
BRF-0146(13)

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Appendix B- L-Pile output file for both strong and weak axis ..... 7 pages each axis

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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	52.9
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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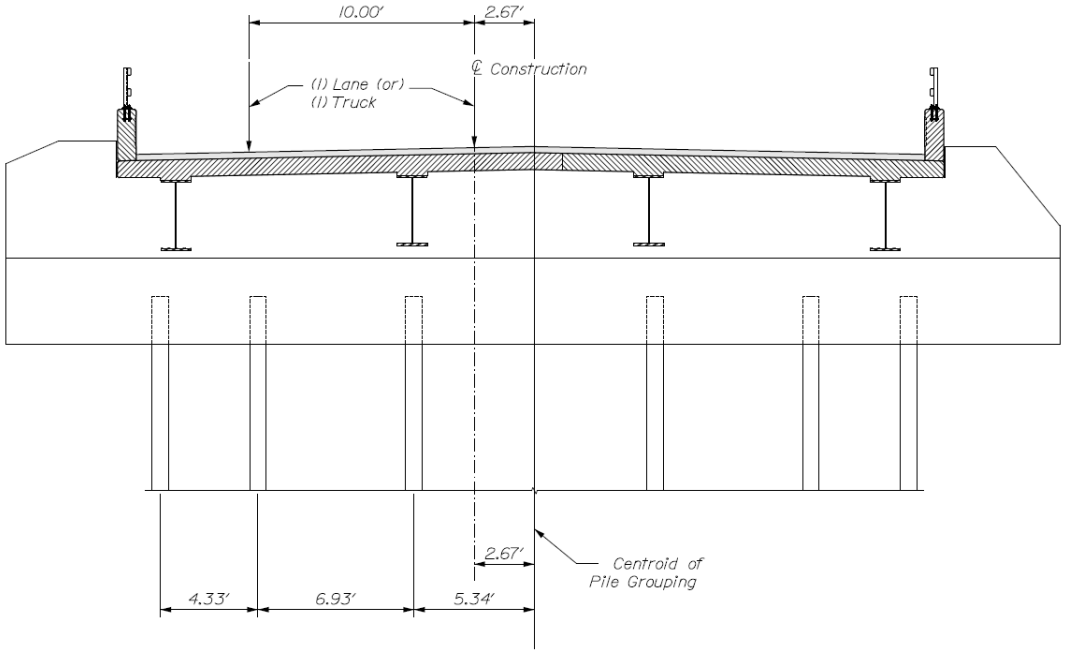

LOCATION OF CENTROID OF DESIGN VEHICLES W.R.T. PILING
ABUTMENT #1 SHOWN - ABUTMENT #2 SIMILAR

$S_1 := 16.60\ ft \qquad S_2 := 12.27\ ft \qquad S_3 := 5.34\ ft$

$$D_{pllunit} := \frac{2}{n_{pile}} + \frac{1 \cdot (12.67\ ft + 2.67\ ft) \cdot 20\ ft}{2 \cdot \left(S_1^{\ 2} + S_2^{\ 2} + S_3^{\ 2}\right)} = 67.0752\%$$

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

$$D_{pllupl} := \frac{2}{n_{pile}} - \frac{1 \cdot (12.67\ ft + 2.67\ ft) \cdot 20\ ft}{2 \cdot \left(S_1^{\ 2} + S_2^{\ 2} + S_3^{\ 2}\right)} = -0.4085\%$$

%age of single lane or truck carried by minimum loaded piling - note percentage is < 1, although once dead load is applied uplift is unlikely to be a real consideration, although it will be checked.

$$R_{pALL} := D_{pllunit} \cdot \left(\max\left(R_{ahs20}, R_{anmtt}\right) + R_{LaneAbut}\right) = 46.6162\ kip$$

$$R_{pALLup} := D_{pllupl} \cdot \left(\max\left(R_{ahs20}, R_{anmtt}\right) + R_{LaneAbut}\right) = -0.28\ kip$$

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.

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

AASHTO LRFD - Tables 3.4.1-1 & 3.4.1-2

$$P_{UAstr1} := \gamma_{LL} \cdot R_{pALL} + \gamma_{DC} \cdot R_{pDC} + \gamma_{DW} \cdot R_{pDW} = 185.89\ kip$$

Factored Maximum Strength I axial load applied to a single piling

$$P_{UsvcI} := R_{pALL} + R_{pDC} + R_{pDW} = 128.46\ kip$$

Unfactored Maximum axial load applied to a single piling

$$P_{UAstr1up} := \gamma_{LL} \cdot R_{pALLup} + 0.90 \cdot R_{pDC} + 0.65 \cdot R_{pDW} = 71.15\ kip$$

Uplift is not a concern as 71.15Kip >> 0

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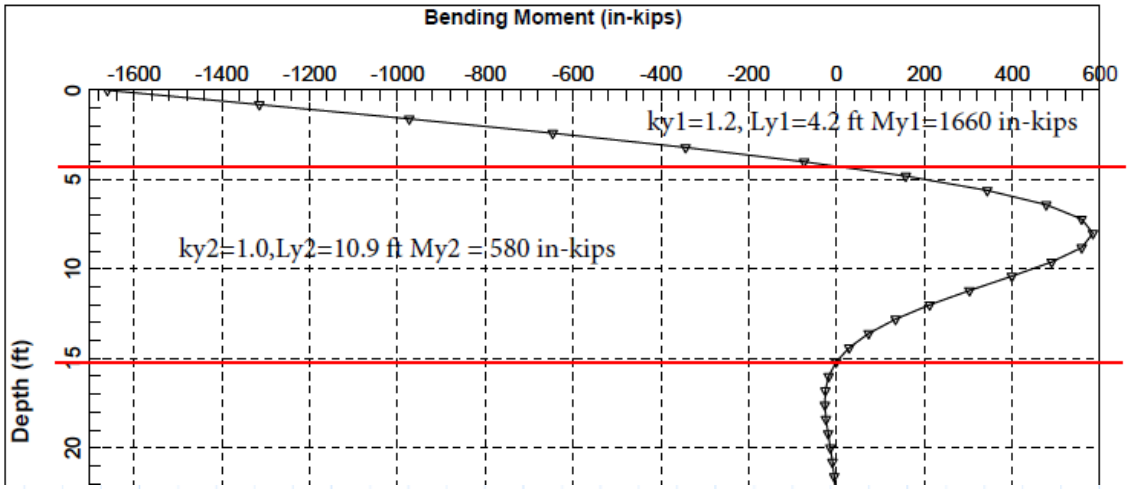
L-Pile Output Y-Y axis

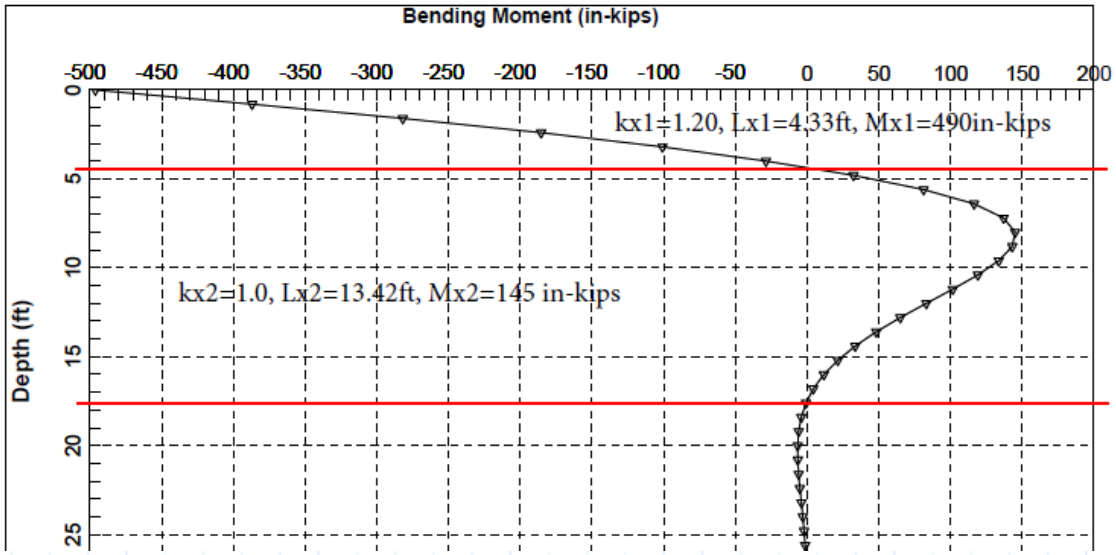


Induced transverse Movement of abutment into fill with HP12x74 weak axis - passive pressure acting at the friction angle from soil to concrete

$\delta_f := 24\ ^\circ$

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

L-Pile Output X-X axis



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Check Strong Axis Flexure:

$L_{x1} := 4.33\ ft$ $\quad L_{x2} := 13.42\ ft$ $\quad \lambda_f := \frac{b_{fpile}}{2 \cdot t_{fpile}} = 10.0123$

$M_{ux1} := 490\ in \cdot kip$ $\quad M_{ux2} := 145\ in \cdot kip$

$C_b := 1$ $\quad Z_{yy} := 46.6\ in^3$ *For HP plastic section modulus weak axis*

$k_{x1} := 1.2$ $\quad S_{yy} := 30.4\ in^3$ *For HP elastic section modulus weak axis*

$k_{x2} := 1.0$ $\quad S_{xx} := 93.8\ in^3$ *For HP elastic section modulus strong axis*

$\varphi_{fc} := 1.0$

$\varphi_{cm} := 0.70$ *Strength Reduction factors (resistance factors) taken from AASHTO 6.5.4.2*

$\varphi_{ct} := 0.50$

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*

$R_b := 1.0$ $\quad R_h := 1.0$

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

$r_{thp} := 3.26\ in$

$$L_p := 1.0 \cdot r_{thp} \cdot \sqrt{\frac{29000\ ksi}{F_{Yhp}}} = 6.54\ ft$$ $\quad L_r := \pi \cdot r_{thp} \cdot \sqrt{\frac{29000\ ksi}{0.7 \cdot F_{Yhp}}} = 24.57\ ft$

For Zone 1 L1<Lp therefore Fnc=50ksi

$F_{ncltb1} := 50\ ksi$

$$F_{ncltb2} := C_b \cdot \left(1 - \left(1 - \frac{0.7 \cdot F_{Yhp}}{R_h \cdot F_{Yhp}}\right) \cdot \left(\frac{L_{x2} - L_p}{L_r - L_p}\right)\right) \cdot F_{Yhp} = 44.28\ ksi$$

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

$F_{cf2} := min(F_{ncltb2}, F_{nclb}) = 44.28\ ksi$

$M_{nx1} := F_{cf1} \cdot S_{xx} = 4516.21\ in \cdot kip$ $\quad M_{nx2} := F_{cf2} \cdot S_{xx} = 4153.14\ in \cdot kip$

$M_{rx1} := \varphi_{fc} \cdot M_{nx1} = 4516.21\ in \cdot kip$ $\quad M_{rx2} := \varphi_{fc} \cdot M_{nx2} = 4153.14\ in \cdot kip$

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Check Weak Axis Flexure:

$L_{y1} := 4.20\ ft$ $\qquad$ $L_{y2} := 10.90\ ft$

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

$k_{y1} := 1.2$

$k_{y2} := 1.0$

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

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

$$P_{ex2} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{y2} \cdot \frac{L_{x2}}{r_{xxpile}}\right)^2} \cdot A_{pile} = 6282.44\ kip$$

$$Q := 1.0$$

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

$$\frac{P_{ex1}}{P_o} = 38.4476 \qquad \frac{P_{ex2}}{P_o} = 5.7637$$

Pe/Po > 0.44 for both zones 1 & 2

$$P_{nx1} := \left(0.658^{\left(\frac{P_o}{P_{ex1}}\right)}\right) \cdot P_o = 1078.2\ kip \qquad P_{nx2} := \left(0.658^{\left(\frac{P_o}{P_{ex2}}\right)}\right) \cdot P_o = 1013.65\ kip$$

$$P_{rx1} := \varphi_{cm} \cdot P_{nx1} = 754.74\ kip \qquad P_{rx2} := \varphi_{cm} \cdot P_{nx2} = 709.56\ kip$$

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Weak Axis Axial Compression

$$P_{ey1} := \frac{\pi^2 \cdot 29000\ ksi}{\left(k_{x1} \cdot \frac{L_{y1}}{r_{yypile}}\right)^2} \cdot A_{pile} = 14544.42\ kip$$

Elastic Flexural Buckling Resistance AASHTO 6.9.4.1.2

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

$$Q := 1.0$$

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

$$\frac{P_{ey1}}{P_o} = 13.3435 \qquad \frac{P_{ey2}}{P_o} = 2.8528$$

Pe/Po > 0.44 for both zones 1 & 2

$$P_{ny1} := \left(0.658^{\left(\frac{P_o}{P_{ey1}}\right)}\right) \cdot P_o = 1056.34\ kip \qquad P_{ny2} := \left(0.658^{\left(\frac{P_o}{P_{ey2}}\right)}\right) \cdot P_o = 941.26\ kip$$

$$P_{ry1} := \varphi_{cm} \cdot P_{ny1} = 739.44\ kip \qquad P_{ry2} := \varphi_{cm} \cdot P_{ny2} = 658.88\ kip$$

$$P_{r1} := min\left(P_{rx1}, P_{ry1}\right) = 739.44\ kip$$

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

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$$\frac{P_{UAstr1}}{P_{r1}} = 0.2514 \qquad \frac{P_{UAstr1}}{P_{r2}} = 0.2821$$ *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

$$\frac{P_{UAstr1}}{P_{r1}} + \frac{8}{9} \cdot \left(\frac{M_{ux1}}{M_{rx1}} + \frac{M_{uy1}}{M_{ry}} \right) = 0.9991$$ *In the Upper Zone < 1 therefor okay*

$$\frac{P_{UAstr1}}{P_{r2}} + \frac{8}{9} \left(\frac{M_{ux2}}{M_{rx2}} + \frac{M_{uy2}}{M_{ry}} \right) = 0.5407$$ *In the middle Zone < 1 therefor okay*

$$\frac{P_{UAstr1}}{P_{r3}} = 0.3411$$ *In the lower Zone < 1 therefor okay*

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Check Passive Pressure behind abutment wall to design Reinforcing

$\delta_f := 24\ °$ $\quad \varphi_{bf} := 32\ °$ $\quad H1 := 8.5\ ft$ $\quad H2 := 4.7\ ft$

$k_p := \tan\left(45\ ° + \frac{\varphi_{bf}}{2}\right)^2 = 3.2546$ *use rankine theory for Hz reinforcement*

$\gamma_{bf} := 130\ pcf$

$p_1 := H1 \cdot \gamma_{bf} \cdot k_p = 3596.32\ psf$ $\quad p_2 := H2 \cdot \gamma_{bf} \cdot k_p = 1988.55\ psf$

$\omega_{lower} := \frac{p_1 + p_2}{2} \cdot (H1 - H2) = 10.61\ klf$

$W_{ing} := 8.75\ ft$

$M_{uAlower} := \frac{\omega_{lower} \cdot \left(\frac{\langle S_3 \cdot 2 + \langle S_2 - S_3 \rangle \rangle \cdot 0.5}{\cos(Skew)}\right)^2}{10} = 1316.27\ in \cdot kip$ *moment in concrete abutment due to passive pressure average spacing between interior piling*

$M_{uAlowing} := \frac{\omega_{lower} \cdot W_{ing}^2}{2} = 4874.55\ in \cdot kip$ *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*

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

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

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$b_{low} := H1 - H2 = 45.6\ in$

$\gamma_{EHP} := 1.1$

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

$n_5 := 6$ *Number of #5 bars*

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

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

$A_{s1} := n_5 \cdot A_5 + n_6 \cdot A_6 + n_7 \cdot A_7 = 1.86\ in^2$ *Area of Horizontal Steel in Lower section of Abutment*

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

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

$\varphi_{vertbar} := 0.75\ in$

$$d_{s1} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} - \varphi_{vertbar} = 31.9375\ in$$

$f_s := f_y = 60\ ksi$ *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*

$$c_{btm} := \frac{A_{ps} \cdot f_{pu} + A_{s1} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{low} + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}\right)} = 0.847\ in$$

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

$$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{btm}}\right) \cdot (c_{btm} - d_{s1}) = -0.1101\ \frac{in}{in}$$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$$M_{n1} := A_{s1} \cdot f_s \cdot \left(d_{s1} - \frac{\beta_1 \cdot c_{btm}}{2}\right) = 3524.06\ in \cdot kip$$

Abutment wall lower section moment nominal capacity

$M_{r1} := \varphi_{tc} \cdot M_{n1} = 3171.6534\ in \cdot kip$

Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay for internal reinforcement add additional reinforcing at wings

$M_{str11} := \gamma_{EHP} \cdot M_{uAlower} = 1447.9\ in \cdot kip$

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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)*

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

$$M_{min1} := min\left(M_{cr1}, \frac{4}{3} \cdot M_{str11}\right) = 1930.53\ in{\cdot}kip$$

$M_{r1} = 3171.65\ in{\cdot}kip$

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

Using (6) number 5 bars in the lower 3.8 feet
of abutment gives us a spacing of 8" centers

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc11} := \gamma_{svc} \cdot M_{uAlower} = 1316.272 \text{ in·kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{sb1t} := A_{s1} \cdot n = 14.8187 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr1} := 4.2426 \text{ in}$$ *distance to cracked neutral axis (assumed)*

$$y_{sb} := C_{lear} + \frac{\varphi_{rebar}}{2} + \varphi_{vertbar} = 4.0625 \text{ in}$$

$$y_{cnc1} := t_{abut} - \frac{c_{cr1}}{2} = 33.8787 \text{ in}$$

$$A_{cnc1} := c_{cr1} \cdot b_{low} = 193.4626 \text{ in}^2$$

$$y_{bar1} := \frac{A_{cnc1} \cdot y_{cnc1} + A_{sb1t} \cdot y_{sb}}{A_{cnc1} + A_{sb1t}} = 31.7574 \text{ in}$$

$$c_{cr1} := t_{abut} - y_{bar1} = 4.2426 \text{ in}$$

$$I_{cr1} := \frac{\left(c_{cr1}^{\ 3} \cdot b_{low}\right)}{12} + A_{cnc1} \cdot \left(y_{cnc1} - y_{bar1}\right)^2 + A_{sb1t} \cdot \left(y_{sb} - y_{bar1}\right)^2 = 12526.8 \text{ in}^4$$

$$S_{cr1s} := \frac{I_{cr1}}{\left(y_{bar1} - y_{sb}\right) \cdot n} = 56.77 \text{ in}^3 \qquad S_{cr1c} := \frac{I_{cr1}}{t_{abut} - y_{bar1}} = 2952.59 \text{ in}^3$$

$$f_{ss1} := \frac{M_{svc11}}{S_{cr1s}} = 23.18 \text{ ksi}$$

$$f_{cnc1} := \frac{M_{svc11}}{S_{cr1c}} = 445.8 \text{ psi}$$

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Using (6) number 5 bars in the lower 3.8 feet of abutment gives us a spacing of 8" centers

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

$d_{c1} := y_{sb} = 4.0625\ \text{in}$

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

$\beta_{sb} := 1 + \frac{d_{c1}}{0.7 \cdot (h - d_{c1})} = 1.1817$ *5.7.3.4*

$s_{max1} := \frac{700 \cdot \gamma_e \cdot \frac{\text{kip}}{\text{in}}}{\beta_{sb} \cdot f_{ss1}} - 2 \cdot d_{c1} = 17.42\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 10 inches per bar << 13.04 in therefor okay*

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Design reinfocement for lower abutment section cantilevered wing walls

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

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

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

$b_{low} := H1 - H2 = 45.6\ in$ $\gamma_{EHP} := 1.1$ *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*

$n_5 := 11$ *Number of #5 bars*

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

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

$A_{s1} := n_5 \cdot A_5 + n_6 \cdot A_6 + n_7 \cdot A_7 = 3.41\ in^2$ *Area of Horizontal Steel in Lower section of Cantilevered Wings in Abutment*

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

$\varphi_{rebar} := 0.625\ in$ $\varphi_{vertbar} := 0.75\ in$

$$d_{s1} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} - \varphi_{vertbar} = 31.9375\ in$$

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

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Flexural Capacity of Lower section of abutment at cantilevered wings:

$k := 0.28$

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

$d_p := 1\ \text{in}$

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

$$c_{btm} := \frac{A_{ps} \cdot f_{pu} + A_{s1} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{low} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 1.553\ \text{in}$$

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

$$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{btm}}\right) \cdot \left(c_{btm} - d_{s1}\right) = -0.0587\ \frac{\text{in}}{\text{in}}$$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$$M_{n1} := A_{s1} \cdot f_s \cdot \left(d_{s1} - \frac{\beta_1 \cdot c_{btm}}{2}\right) = 6399.41\ \text{in·kip}$$

Abutment wall lower section moment nominal capacity

$$M_{r1} := \varphi_{tc} \cdot M_{n1} = 5759.4702\ \text{in·kip}$$

Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay for internal reinforcement add additional reinforcing at wings

$$M_{str11} := \gamma_{EHP} \cdot M_{uAlowing} = 5362\ \text{in·kip}$$

Check Minimum Reinforcing requirements: per section 5.7.3.3.2

$$S_c := \frac{\left(t_{abut}^{\ 2} \cdot b_{low}\right)}{6} = 9849.6\ \text{in}^3$$

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*

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

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

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

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

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

$$M_{min1} := min\left(M_{cr1}, \frac{4}{3} \cdot M_{str11}\right) = 5068.21\ \text{in·kip}$$

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

$M_{r1} = 5759.47\ \text{in·kip}$

Using (11) number 5 bars in the lower 3.8 feet gives us a spacing of 4" centers

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc11} := \gamma_{svc} \cdot M_{uAlowing} = 4874.5474 \text{ in}\cdot\text{kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{sb1t} := A_{s1} \cdot n = 27.1676 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr1} := 5.6018 \text{ in}$$ *distance to cracked neutral axis (assumed)*

$$y_{sb} := C_{lear} + \frac{\varphi_{rebar}}{2} + \varphi_{vertbar} = 4.0625 \text{ in}$$

$$y_{cnc1} := t_{abut} - \frac{c_{cr1}}{2} = 33.1991 \text{ in}$$

$$A_{cnc1} := c_{cr1} \cdot b_{low} = 255.4421 \text{ in}^2$$

$$y_{bar1} := \frac{A_{cnc1} \cdot y_{cnc1} + A_{sb1t} \cdot y_{sb}}{A_{cnc1} + A_{sb1t}} = 30.3982 \text{ in}$$

$$c_{cr1} := t_{abut} - y_{bar1} = 5.6018 \text{ in}$$

$$I_{cr1} := \frac{\left(c_{cr1}{}^3 \cdot b_{low}\right)}{12} + A_{cnc1} \cdot \left(y_{cnc1} - y_{bar1}\right)^2 + A_{sb1t} \cdot \left(y_{sb} - y_{bar1}\right)^2 = 21514.55 \text{ in}^4$$

$$S_{cr1s} := \frac{I_{cr1}}{\left(y_{bar1} - y_{sb}\right) \cdot n} = 102.54 \text{ in}^3 \qquad S_{cr1c} := \frac{I_{cr1}}{t_{abut} - y_{bar1}} = 3840.63 \text{ in}^3$$

$$f_{ss1} := \frac{M_{svc11}}{S_{cr1s}} = 47.54 \text{ ksi}$$

$$f_{cnc1} := \frac{M_{svc11}}{S_{cr1c}} = 1269.21 \text{ psi}$$

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

$d_{c1} := y_{sb} = 4.0625\ \text{in}$

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

$\beta_{sb} := 1 + \dfrac{d_{c1}}{0.7 \cdot \langle h - d_{c1} \rangle} = 1.1817$ *5.7.3.4*

$s_{max1} := \dfrac{700 \cdot \gamma_e \cdot \dfrac{\text{kip}}{\text{in}}}{\beta_{sb} \cdot f_{ss1}} - 2 \cdot d_{c1} = 4.34\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 4 inches per bar << 4.34 in therefor okay*

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$p_3 := 0\ psf$ $\quad p_2 = 1988.55\ psf$

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

$$\omega_{upper} := \frac{p_2 + p_3}{2} \cdot (H2) = 4.67\ klf$$

$$M_{uAupper} := \frac{\omega_{upper} \cdot \left(\frac{S}{\cos(Skew)}\right)^2}{10} = 824.33\ in \cdot kip$$

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

$$M_{uAupwing} := \frac{\omega_{upper} \cdot W_{ing}^2}{2} = 2146.71\ in \cdot kip$$

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

$n_{5U} := 7$ *Number of #5 bars*

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

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

$A_{s2} := n_{5U} \cdot A_5 + n_{6U} \cdot A_6 + n_{7U} \cdot A_7 = 2.17\ in^2$ *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*

$$c_{top} := \frac{A_{ps} \cdot f_{pu} + A_{s2} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{up} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 0.799\ in$$

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

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{top}}\right) \cdot \left(c_{top} - d_{s2}\right) = -0.116\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_{n2} := A_{s2} \cdot f_s \cdot \left(d_{s2} - \frac{\beta_1 \cdot c_{top}}{2}\right) = 4081.51\ in \cdot kip$ *Abutment wall upper section moment nominal capacity*

$M_{r2} := \varphi_{tc} \cdot M_{n2} = 3673.3602\ in \cdot kip$ *Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay*

$M_{str12} := \gamma_{EHP} \cdot \max\left(M_{uAupper}, M_{uAupwing}\right) = 2361.38\ in \cdot kip$

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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)*

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

$$M_{min2} := min\left(M_{cr2}, \frac{4}{3} \cdot M_{str12} \right) = 3148.5\ in \cdot kip$$

$M_{r2} = 3673.36\ in \cdot kip$ *Resisting moment is > Mmin2 therefor minimum reinforcement is adequate per section 5.7.3.3.2*

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Using (7) number 5 bars in the Upper 4.7 feet
of abutment gives us a spacing of 8" centers

Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svc12} := \gamma_{svc} \cdot \max\left(M_{uAupper}, M_{uAupwing}\right) = 2146.7056 \text{ in}\cdot\text{kip}$$

Cracked Neutral Axis Positive Flexure:

$$A_{su1t} := A_{s2} \cdot n = 17.2885 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr2} := 4.1117 \text{ in}$$ *distance to cracked neutral axis (assumed)*

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

$$y_{cnc2} := t_{abut} - \frac{c_{cr2}}{2} = 33.9442 \text{ in}$$

$$A_{cnc2} := c_{cr2} \cdot b_{up} = 231.8999 \text{ in}^2$$

$$y_{bar2} := \frac{A_{cnc2} \cdot y_{cnc2} + A_{su1t} \cdot y_{su}}{A_{cnc2} + A_{su1t}} = 31.8883 \text{ in}$$

$$c_{cr2} := t_{abut} - y_{bar2} = 4.1117 \text{ in}$$

$$I_{cr2} := \frac{\left(c_{cr2}^{\ 3} \cdot b_{up}\right)}{12} + A_{cnc2} \cdot \left(y_{cnc2} - y_{bar2}\right)^2 + A_{su1t} \cdot \left(y_{su} - y_{bar2}\right)^2 = 14453.41 \text{ in}^4$$

$$S_{cr2s} := \frac{I_{cr2}}{\left(y_{bar2} - y_{su}\right) \cdot n} = 65.79 \text{ in}^3 \qquad S_{cr2c} := \frac{I_{cr2}}{t_{abut} - y_{bar2}} = 3515.22 \text{ in}^3$$

$$f_{ss2} := \frac{M_{svc12}}{S_{cr2s}} = 32.63 \text{ ksi}$$

$$f_{cnc2} := \frac{M_{svc12}}{S_{cr2c}} = 610.69 \text{ psi}$$

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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*

$s_{max2} := \frac{700 \cdot \gamma_e \cdot \frac{\text{kip}}{\text{in}}}{\beta_{su} \cdot f_{ss2}} - 2 \cdot d_{c2} = 9.34\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 8 inches per bar <9.34 in therefor okay*

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

$n_{6V} := 32$ *Number of #6 bars* $n_{8V} := 0$ *Number of #8 bars*

$A_{s3} := n_{5V} \cdot A_5 + n_{6V} \cdot A_6 + n_{7V} \cdot A_7 + n_{8V} \cdot A_8 = 14.08\ in^2$ *Area of Vetrical 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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$$d_{s3} := t_{abut} - C_{lear} - \frac{\varphi_{rebar}}{2} = 32.5 \text{ in}$$

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

Flexural Capacity of abutment due to vertical reinforcing:

$$k = 0.28$$

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

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

$$c_{vert} := \frac{A_{ps} \cdot f_{pu} + A_{s3} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 2.32 \text{ in}$$

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

$$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{vert}}\right) \cdot \left(c_{vert} - d_{s3}\right) = -0.039 \frac{\text{in}}{\text{in}}$$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$$M_{n3} := A_{s3} \cdot f_s \cdot \left(d_{s3} - \frac{\beta_1 \cdot c_{vert}}{2}\right) = 26623.03 \text{ in·kip}$$

Abutment wall lower section moment nominal capacity

$$M_{r3} := \varphi_{tc} \cdot M_{n3} = 23960.7287 \text{ in·kip}$$

Moment Capacity is > Str I moment therefor okay for str. I Moment therefor okay

$$M_{str13} = 23191.87 \text{ in·kip}$$

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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)*

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

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

$M_{r3} = 23960.73\ in \cdot kip$ *Resisting moment is > Mmin3 therefor minimum reinforcement is adequate per section 5.7.3.3.2*

Using number 6 bars at 4"

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:

$$A_{svt} := A_{s3} \cdot n = 112.1758 \text{ in}^2$$ *area of steel transformed to concrete*

$$c_{cr3} := 6.7688 \text{ in}$$ *distance to cracked neutral axis (assumed)*

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

$$y_{cnc3} := t_{abut} - \frac{c_{cr3}}{2} = 32.6156 \text{ in}$$

$$A_{cnc3} := c_{cr3} \cdot b_w = 852.8688 \text{ in}^2$$

$$y_{bar3} := \frac{A_{cnc3} \cdot y_{cnc3} + A_{svt} \cdot y_{sv}}{A_{cnc3} + A_{svt}} = 29.2312 \text{ in}$$

$$c_{cr3} := t_{abut} - y_{bar3} = 6.7688 \text{ in}$$

$$I_{cr3} := \frac{\left(c_{cr3}^{\ 3} \cdot b_w\right)}{12} + A_{cnc3} \cdot \left(y_{cnc3} - y_{bar3}\right)^2 + A_{svt} \cdot \left(y_{sv} - y_{bar3}\right)^2 = 87296.17 \text{ in}^4$$

$$S_{cr3s} := \frac{I_{cr3}}{\left(y_{bar3} - y_{sv}\right) \cdot n} = 425.83 \text{ in}^3 \qquad S_{cr3c} := \frac{I_{cr3}}{t_{abut} - y_{bar3}} = 12896.91 \text{ in}^3$$

$$f_{ss3} := \frac{M_{svc13}}{S_{cr3s}} = 31.98 \text{ ksi}$$

$$f_{cnc3} := \frac{M_{svc13}}{S_{cr3c}} = 1055.94 \text{ psi}$$

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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*

$s_{max3} := \frac{700 \cdot \gamma_e \cdot \frac{\text{kip}}{\text{in}}}{\beta_{sv} \cdot f_{ss3}} - 2 \cdot d_{c3} = 11.97\ \text{in}$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 4 inches per bar << 11.97 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

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

612x74 Strong way Abutment Piling Weak Axis.lpo

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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 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

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Time and Date of Analysis

Date: January 29, 2015 Time: 7:19:19

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 = 185890.000 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 = 185890.000 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	-496330.	11599.4178	0.0000	13854.5438	0.0000	0.0000
9.600	.064214	-387012.	11121.5916	-.0002570	12681.1469	-49.7732	7441.1043
19.200	.060666	-281878.	10412.4549	-.0004515	11552.6740	-97.9643	15502.1021
28.800	.055545	-185481.	9319.5832	-.0005875	10517.9682	-129.7173	22419.5821
38.400	.049387	-100846.	8043.3160	-.0006708	9609.5147	-136.1717	26469.5935
48.000	.042666	-28655.1546	6804.1804	-.0007084	8834.6413	-121.9816	27446.4373
57.600	.035785	32323.1876	5587.4554	-.0007074	8874.0130	-131.5028	35278.4047
67.200	.029084	81148.6727	4252.6156	-.0006744	9398.0933	-146.5888	48385.5391
76.800	.022837	116380.	2791.3177	-.0006169	9776.2603	-157.8483	66355.2000
86.400	.017240	136944.	1390.1841	-.0005432	9996.9835	-134.0546	74649.6000
96.000	.012407	145011.	232.1767	-.0004612	10083.5709	-107.1970	82944.0000
105.600	.008384	143048.	-664.8625	-.0003774	10062.5007	-79.6862	91238.4000
115.200	.005161	133592.	-1304.1936	-.0002969	9961.0091	-53.5078	99532.8000
124.800	.002683	119067.	-1705.7004	-.0002234	9805.0977	-30.1395	107827.
134.400	.000871	101640.	-1900.9325	-.0001592	9618.0448	-10.5339	116122.
144.000	-.000374	83137.3162	-1928.2312	-.0001055	9419.4388	4.8467	124416.
153.600	-.001154	64994.7169	-1828.3624	-6.2395E-05	9224.7008	15.9593	132710.
163.200	-.001572	48255.4513	-1640.9308	-2.9452E-05	9045.0259	23.0889	141005.
172.800	-.001720	33593.9613	-1401.7112	-5.6424E-06	8887.6532	26.7485	149299.
182.400	-.001680	21362.7340	-1140.9168	1.0344E-05	8756.3663	27.5837	157594.
192.000	-.001521	11651.4400	-882.3296	1.9948E-05	8652.1278	26.2886	165888.
201.600	-.001297	4350.8106	-643.1608	2.4602E-05	8573.7647	23.5382	174182.
211.200	-.001049	-785.0549	-434.4714	2.5640E-05	8535.4908	19.9387	182477.
220.800	-.000805	-4082.5505	-261.9784	2.4224E-05	8570.8853	15.9973	190771.
230.400	-.000584	-5901.4968	-127.0772	2.1320E-05	8590.4094	12.1071	199066.
240.000	-.000396	-6598.5241	-27.9387	1.7683E-05	8597.8911	8.5467	207360.
249.600	-.000244	-6501.0331	39.4331	1.3873E-05	8596.8446	5.4891	215654.
259.200	-.000129	-5890.9213	80.2616	1.0268E-05	8590.2959	3.0169	223949.
268.800	-4.72E-05	-4996.6574	100.2238	7.1010E-06	8580.6971	1.1419	

232243.							
278.400	7.01E-06	-3991.9683	104.8613	4.4863E-06	8569.9130	-.1757535	
240538.							
288.000	3.89E-05	-2999.3321	99.1736	2.4526E-06	8559.2583	-1.0092	
248832.							
297.600	5.41E-05	-2096.5890	87.3737	9.7021E-07	8549.5685	-1.4491	
257126.							
307.200	5.76E-05	-1325.2195	71.5693	-2.5170E-08	8541.2888	-1.8435	
307443.							
316.800	5.36E-05	-722.3680	54.1692	-6.2080E-07	8534.8179	-1.7816	
318963.							
326.400	4.56E-05	-282.9547	38.0755	-9.1324E-07	8530.1014	-1.5713	
330483.							
336.000	3.61E-05	11.9420	24.3625	-9.9207E-07	8527.1924	-1.2856	
342003.							
345.600	2.66E-05	188.3466	13.4906	-9.3381E-07	8529.0859	-.9793979	
353523.							
355.200	1.82E-05	274.2944	5.4754	-7.9923E-07	8530.0084	-.6904327	
365043.							
364.800	1.13E-05	296.3272	.0430683	-6.3324E-07	8530.2449	-.4413068	
376563.							
374.400	6.00E-06	277.3814	-3.2393	-4.6635E-07	8530.0416	-.2425098	
388083.							
384.000	2.30E-06	235.7980	-4.8622	-3.1707E-07	8529.5952	-.0955951	
399603.							
393.600	-8.89E-08	185.1597	-5.3027	-1.9462E-07	8529.0517	.0038061	
411123.							
403.200	-1.44E-06	134.6800	-4.9801	-1.0158E-07	8528.5098	.0634040	
422643.							
412.800	-2.04E-06	89.9037	-4.2331	-3.6253E-08	8528.0292	.0922264	
434163.							
422.400	-2.14E-06	53.5337	-3.3144	5.4713E-09	8527.6388	.0991754	
445683.							
432.000	-1.93E-06	26.2481	-2.3962	2.8679E-08	8527.3460	.0921176	
457203.							
441.600	-1.59E-06	7.4248	-1.5824	3.8474E-08	8527.1439	.0774172	
468723.							
451.200	-1.20E-06	-4.2714	-.9237352	3.9392E-08	8527.1101	.0598056	
480243.							
460.800	-8.29E-07	-10.4515	-.4327648	3.5109E-08	8527.1764	.0424799	
491763.							
470.400	-5.21E-07	-12.7058	-.0976517	2.8373E-08	8527.2006	.0273354	
503283.							
480.000	-2.85E-07	-12.4276	.1067936	2.1062E-08	8527.1976	.0152574	
514803.							
489.600	-1.17E-07	-10.7305	.2108275	1.4325E-08	8527.1794	.0064163	
526323.							
499.200	-9.48E-09	-8.4309	.2441741	8.7513E-09	8527.1547	.0005309	
537843.							
508.800	5.10E-08	-6.0736	.2327159	4.5320E-09	8527.1294	-.0029180	
549363.							
518.400	7.75E-08	-3.9789	.1969642	1.6078E-09	8527.1069	-.0045302	
560883.							
528.000	8.19E-08	-2.2976	.1517899	-2.1794E-10	8527.0889	-.0048811	
572403.							
537.600	7.34E-08	-1.0638	.1069441	-1.1957E-09	8527.0756	-.0044618	
583923.							
547.200	5.89E-08	-.2400121	.0679902	-1.5750E-09	8527.0668	-.0036536	
595443.							
556.800	4.31E-08	.2472722	.0373686	-1.5729E-09	8527.0669	-.0027259	
606963.							
566.400	2.87E-08	.4830779	.0154072	-1.3604E-09	8527.0694	-.0018494	
618483.							

576.000	1.70E-08	.5479452	.0011770	-1.0605E-09	8527.0701	-.0011153
630003.						
585.600	8.34E-09	.5094607	-.0068527	-7.5292E-10	8527.0697	-.0005576
641523.						
595.200	2.54E-09	.4190614	-.0103579	-4.8282E-10	8527.0687	-.0001727
653043.						
604.800	-9.26E-10	.3123130	-.0108789	-2.7007E-10	8527.0676	6.4128E-05
664563.						
614.400	-2.65E-09	.2111504	-.0096763	-1.1780E-10	8527.0665	.0001864
676083.						
624.000	-3.19E-09	.1269478	-.0076855	-1.9446E-11	8527.0656	.0002283
687603.						
633.600	-3.02E-09	.0636576	-.0055337	3.5999E-11	8527.0649	.0002199
699123.						
643.200	-2.50E-09	.0205718	-.0035908	6.0501E-11	8527.0644	.0001848
710643.						
652.800	-1.86E-09	-.0055013	-.0020325	6.4885E-11	8527.0643	.0001398
722163.						
662.400	-1.25E-09	-.0186834	-.0009024	5.7850E-11	8527.0644	9.5614E-05
733683.						
672.000	-7.48E-10	-.0230343	-.0001648	4.5714E-11	8527.0645	5.8055E-05
745203.						
681.600	-3.73E-10	-.0220109	.0002551	3.2611E-11	8527.0645	2.9430E-05
756723.						
691.200	-1.22E-10	-.0182524	.0004432	2.0899E-11	8527.0644	9.7439E-06
768243.						
700.800	2.79E-11	-.0135769	.0004791	1.1640E-11	8527.0644	-2.2659E-06
779763.						
710.400	1.02E-10	-.0090962	.0004279	5.0445E-12	8527.0643	-8.3848E-06
791283.						
720.000	1.25E-10	-.0053787	.0003376	8.3390E-13	8527.0643	-1.0432E-05
802803.						
729.600	1.18E-10	-.0026172	.0002396	-1.4920E-12	8527.0642	-9.9871E-06
814323.						
739.200	9.61E-11	-.0007733	.0001520	-2.4783E-12	8527.0642	-8.2675E-06
825843.						
748.800	7.02E-11	.0003095	8.2912E-05	-2.6132E-12	8527.0642	-6.1192E-06
837363.						
758.400	4.59E-11	.0008280	3.4045E-05	-2.2823E-12	8527.0642	-4.0615E-06
848883.						
768.000	2.63E-11	.0009713	3.2216E-06	-1.7590E-12	8527.0642	-2.3601E-06
860403.						
777.600	1.22E-11	.0008961	-1.3408E-05	-1.2158E-12	8527.0642	-1.1043E-06
871923.						
787.200	2.99E-12	.0007182	-2.0029E-05	-7.4619E-13	8527.0642	-2.7516E-07
883443.						
796.800	-2.17E-12	.0005142	-2.0380E-05	-3.8770E-13	8527.0642	2.0209E-07
894963.						
806.400	-4.45E-12	.0003283	-1.7392E-05	-1.4263E-13	8527.0642	4.2054E-07
906483.						
816.000	-4.91E-12	.0001808	-1.3121E-05	5.4465E-15	8527.0642	4.6916E-07
918003.						
825.600	-4.35E-12	7.6333E-05	-8.8477E-06	8.0238E-14	8527.0642	4.2111E-07
929523.						
835.200	-3.37E-12	1.0617E-05	-5.2427E-06	1.0553E-13	8527.0642	3.2992E-07
941043.						
844.800	-2.32E-12	-2.4704E-05	-2.5527E-06	1.0143E-13	8527.0642	2.3049E-07
952563.						
854.400	-1.42E-12	-3.8757E-05	-7.6272E-07	8.2973E-14	8527.0642	1.4242E-07
964083.						
864.000	-7.30E-13	-3.9644E-05	2.7692E-07	6.0167E-14	8527.0642	7.4172E-08
975603.						
873.600	-2.63E-13	-3.3655E-05	7.6273E-07	3.8845E-14	8527.0642	2.7039E-08

```
987123.
 883.200  1.60E-14 -2.5138E-05  8.8454E-07  2.1743E-14   8527.0642 -1.6618E-09
998643.
 892.800  1.55E-13 -1.6749E-05  7.9853E-07  9.5583E-15   8527.0642 -1.6258E-08
1010163.
 902.400  1.99E-13 -9.8407E-06  6.1858E-07  1.8236E-15   8527.0642 -2.1231E-08
1021683.
 912.000  1.90E-13 -4.8788E-06  4.1876E-07 -2.4582E-15   8527.0642 -2.0397E-08
1033203.
 921.600  1.52E-13 -1.7917E-06  2.4130E-07 -4.3986E-15   8527.0642 -1.6574E-08
1044723.
 931.200  1.05E-13 -2.3024E-07  1.0625E-07 -4.9867E-15   8527.0642 -1.1560E-08
1056243.
 940.800  5.66E-14  2.6611E-07  2.0571E-08 -4.9763E-15   8527.0642 -6.2901E-09
1067763.
 950.400  9.52E-15  1.8249E-07 -1.4761E-08 -4.8458E-15   8527.0642 -1.0707E-09
1079283.
 960.000 -3.65E-14      0.0000      0.0000 -4.7927E-15   8527.0642  4.1458E-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      =     -.00003180
Maximum bending moment           =  -496330.06456 lbs-in
Maximum shear force              =    11599.41783 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	185890.	.0656000	-496330.	11599.4178

The analysis ended normally.

==========

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

---

Time and Date of Analysis

Date: January 29, 2015 Time: 7:17:46

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 = 185890.000 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 = 185890.000 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	-1660602.	36247.2243	0.0000	62675.1959	0.0000	0.0000
9.600	.609814	-1314336.	35342.1254	-.0026473	51384.3175	-94.2799	1484.2026
19.200	.573171	-972585.	33944.4456	-.0046824	40240.6438	-196.9058	3297.9599
28.800	.519911	-645891.	31635.2009	-.0061227	29587.9697	-284.1868	5247.4196
38.400	.455616	-343336.	28503.3367	-.0070029	19722.4177	-368.2849	7759.8964
48.000	.385455	-73632.7182	24707.6656	-.0073740	10928.0451	-422.4799	10522.1325
57.600	.314035	157369.	20334.1508	-.0072995	13658.4831	-488.6690	14938.5108
67.200	.245305	342835.	15397.0914	-.0068544	19706.0813	-539.8851	21128.3954
76.800	.182432	477457.	10057.4999	-.0061244	24095.7605	-572.5298	30127.9138
86.400	.127716	557798.	4561.8540	-.0052032	26715.4778	-572.3964	43025.1052
96.000	.082531	583615.	-746.8326	-.0041874	27557.3138	-533.5800	62065.8619
105.600	.047318	558404.	-5466.6039	-.0031712	26735.2424	-449.7057	91238.4000
115.200	.021645	489975.	-8702.3636	-.0022382	24503.9259	-224.4109	99532.8000
124.800	.004343	399307.	-10013.6911	-.0014469	21547.4867	-48.7823	107827.
134.400	-.006136	302876.	-9891.5951	-.0008220	18403.0977	74.2190	116122.
144.000	-.011440	212323.	-8823.6850	-.0003636	15450.3782	148.2622	124416.
153.600	-.013116	134759.	-7241.6799	-5.4715E-05	12921.2142	181.3222	132710.
163.200	-.012491	73477.6011	-5490.7218	.0001306	10922.9871	183.4608	141005.
172.800	-.010609	28870.8177	-3818.1408	.0002217	9468.4702	164.9936	149299.
182.400	-.008235	-621.8519	-2377.3184	.0002468	8547.3413	135.1778	157594.
192.000	-.005870	-17654.5644	-1241.5428	.0002305	9102.7359	101.4421	165888.
201.600	-.003808	-25282.2975	-422.9672	.0001923	9351.4574	69.0945	174182.
211.200	-.002178	-26461.9880	107.3762	.0001463	9389.9242	41.3938	182477.
220.800	-.000999	-23742.7857	401.3953	.0001016	9301.2577	19.8602	190771.
230.400	-.000227	-19117.8559	519.2960	6.3470E-05	9150.4502	4.7024	199066.
240.000	.000219	-13998.8329	519.1392	3.4000E-05	8983.5315	-4.7351	207360.
249.600	.000426	-9271.7340	450.4732	1.3292E-05	8829.3925	-9.5703	215654.
259.200	.000474	-5397.1885	351.4118	2.3873E-07	8703.0532	-11.0675	223949.
268.800	.000431	-2525.4789	248.2845	-6.8115E-06	8609.4138	-10.4174	

232243.							
278.400	.000344	-605.8148	156.9509	-9.5979E-06	8546.8183	-8.6104	
240538.							
288.000	.000246	522.2349	84.9731	-9.6723E-06	8544.0930	-6.3849	
248832.							
297.600	.000158	1060.1903	34.0202	-8.2642E-06	8561.6344	-4.2303	
257126.							
307.200	8.77E-05	1204.9185	.2726000	-6.2485E-06	8566.3536	-2.8005	
306692.							
316.800	3.80E-05	1087.7256	-19.2108	-4.2083E-06	8562.5323	-1.2585	
318212.							
326.400	6.86E-06	851.0903	-26.3830	-2.4830E-06	8554.8162	-.2356539	
329732.							
336.000	-9.71E-06	590.0340	-25.8582	-1.2006E-06	8546.3038	.3449912	
341252.							
345.600	-1.62E-05	358.8981	-21.3465	-3.5615E-07	8538.7670	.5949432	
352772.							
355.200	-1.65E-05	181.4524	-15.4775	1.2470E-07	8532.9809	.6277671	
364292.							
364.800	-1.38E-05	61.2853	-9.8719	3.4071E-07	8529.0626	.5400732	
375812.							
374.400	-1.00E-05	-9.3033	-5.3425	3.8696E-07	8527.3676	.4035384	
387332.							
384.000	-6.37E-06	-42.6722	-2.1359	3.4071E-07	8528.4557	.2645014	
398852.							
393.600	-3.46E-06	-51.5291	-.1563749	2.5688E-07	8528.7445	.1479055	
410372.							
403.200	-1.43E-06	-46.5914	.8560961	1.6957E-07	8528.5835	.0630260	
421892.							
412.800	-2.04E-07	-35.6973	1.2029	9.6342E-08	8528.2282	.0092234	
433412.							
422.400	4.16E-07	-23.8397	1.1547	4.3361E-08	8527.8416	-.0192631	
444932.							
432.000	6.28E-07	-13.6817	.9188601	9.9715E-09	8527.5103	-.0298707	
456452.							
441.600	6.07E-07	-6.2332	.6334322	-7.7503E-09	8527.2675	-.0295934	
467972.							
451.200	4.79E-07	-1.4922	.3764424	-1.4625E-08	8527.1129	-.0239461	
479492.							
460.800	3.26E-07	1.0467	.1813967	-1.5021E-08	8527.0984	-.0166884	
491012.							
470.400	1.91E-07	2.0443	.0532953	-1.2271E-08	8527.1309	-.0099994	
502532.							
480.000	9.07E-08	2.1138	-.0180102	-8.5705E-09	8527.1331	-.0048560	
514052.							
489.600	2.65E-08	1.7291	-.0482738	-5.1509E-09	8527.1206	-.0014490	
525572.							
499.200	-8.21E-09	1.2053	-.0530238	-2.5396E-09	8527.1035	.0004594	
537092.							
508.800	-2.23E-08	.7200668	-.0447032	-8.2629E-10	8527.0877	.0012741	
548612.							
518.400	-2.41E-08	.3499570	-.0318448	1.2590E-10	8527.0756	.0014047	
560132.							
528.000	-1.99E-08	.1081975	-.0194205	5.3360E-10	8527.0677	.0011836	
571652.							
537.600	-1.38E-08	-.0248208	-.0097062	6.0780E-10	8527.0650	.0008402	
583172.							
547.200	-8.21E-09	-.0803303	-.0032328	5.1422E-10	8527.0668	.0005084	
594692.							
556.800	-3.96E-09	-.0887260	.0004073	3.6379E-10	8527.0671	.0002499	
606212.							
566.400	-1.22E-09	-.0738088	.0019846	2.1915E-10	8527.0666	7.8704E-05	
617732.							

576.000	2.50E-10	-.0514038	.0022837	1.0773E-10	8527.0659	-1.6399E-05	629252.
585.600	8.45E-10	-.0303469	.0019342	3.4977E-11	8527.0652	-5.6416E-05	640772.
595.200	9.22E-10	-.0143929	.0013627	-4.8357E-12	8527.0647	-6.2630E-05	652292.
604.800	7.52E-10	-.0041652	.0008124	-2.1350E-11	8527.0644	-5.2024E-05	663812.
614.400	5.12E-10	.0012812	.0003898	-2.3917E-11	8527.0643	-3.6006E-05	675332.
624.000	2.93E-10	.0034052	.0001163	-1.9746E-11	8527.0643	-2.0976E-05	686852.
633.600	1.33E-10	.0035853	-3.0682E-05	-1.3526E-11	8527.0643	-9.6534E-06	698372.
643.200	3.35E-11	.0028644	-8.8902E-05	-7.7862E-12	8527.0643	-2.4758E-06	709892.
652.800	-1.68E-11	.0019062	-9.4727E-05	-3.5410E-12	8527.0643	1.2623E-06	721412.
662.400	-3.45E-11	.0010583	-7.6022E-05	-9.0303E-13	8527.0643	2.6345E-06	732932.
672.000	-3.41E-11	.0004498	-5.0670E-05	4.3894E-13	8527.0642	2.6471E-06	744452.
681.600	-2.61E-11	8.3827E-05	-2.8106E-05	9.1378E-13	8527.0642	2.0537E-06	755972.
691.200	-1.66E-11	-9.3119E-05	-1.1881E-05	9.0551E-13	8527.0642	1.3264E-06	767492.
700.800	-8.69E-12	-.0001475	-2.1282E-06	6.9137E-13	8527.0642	7.0548E-07	779012.
710.400	-3.32E-12	-.0001364	2.5693E-06	4.3866E-13	8527.0642	2.7315E-07	790532.
720.000	-2.72E-13	-9.9762E-05	3.9893E-06	2.2846E-13	8527.0642	2.2687E-08	802052.
729.600	1.07E-12	-6.0669E-05	3.6631E-06	8.5700E-14	8527.0642	-9.0634E-08	813572.
739.200	1.37E-12	-2.9736E-05	2.6613E-06	5.2504E-15	8527.0642	-1.1808E-07	825092.
748.800	1.17E-12	-9.5912E-06	1.6050E-06	-2.9746E-14	8527.0642	-1.0199E-07	836612.
758.400	8.03E-13	1.1853E-06	7.7500E-07	-3.7226E-14	8527.0642	-7.0923E-08	848132.
768.000	4.56E-13	5.4217E-06	2.3878E-07	-3.1347E-14	8527.0642	-4.0791E-08	859652.
777.600	2.01E-13	5.8817E-06	-4.4530E-08	-2.1288E-14	8527.0642	-1.8232E-08	871172.
787.200	4.68E-14	4.6427E-06	-1.5269E-07	-1.1923E-14	8527.0642	-4.3021E-09	882692.
796.800	-2.80E-14	2.9925E-06	-1.6082E-07	-5.1283E-15	8527.0642	2.6088E-09	894212.
806.400	-5.17E-14	1.5733E-06	-1.2490E-07	-1.0653E-15	8527.0642	4.8754E-09	905732.
816.000	-4.85E-14	5.9830E-07	-7.9270E-08	8.6710E-16	8527.0642	4.6304E-09	917252.
825.600	-3.50E-14	4.8173E-08	-4.0779E-08	1.4424E-15	8527.0642	3.3887E-09	928772.
835.200	-2.08E-14	-1.8981E-07	-1.4749E-08	1.3163E-15	8527.0642	2.0341E-09	940292.
844.800	-9.75E-15	-2.3971E-07	-3.4432E-10	9.3413E-16	8527.0642	9.6692E-10	951812.
854.400	-2.83E-15	-1.9975E-07	5.6612E-09	5.4306E-16	8527.0642	2.8424E-10	963332.
864.000	6.74E-16	-1.3296E-07	6.6968E-09	2.4699E-16	8527.0642	-6.8485E-11	974852.
873.600	1.91E-15	-7.2053E-08	5.4263E-09	6.4557E-17	8527.0642	-1.9622E-10	

```
986372.
 883.200  1.91E-15 -2.9003E-08  3.5295E-09 -2.5371E-17   8527.0642 -1.9895E-10
997892.
 892.800  1.42E-15 -4.1964E-09  1.8566E-09 -5.4915E-17   8527.0642 -1.4958E-10
1009412.
 902.400  8.60E-16  6.8387E-09  6.9981E-10 -5.2563E-17   8527.0642 -9.1410E-11
1020932.
 912.000  4.13E-16  9.4276E-09  4.7654E-11 -3.8088E-17   8527.0642 -4.4456E-11
1032452.
 921.600  1.28E-16  7.8896E-09 -2.3268E-10 -2.2678E-17   8527.0642 -1.3947E-11
1043972.
 931.200 -2.21E-17  5.0411E-09 -2.8799E-10 -1.1171E-17   8527.0642  2.4247E-12
1055492.
 940.800 -8.62E-17  2.4000E-09 -2.3035E-10 -4.5495E-18   8527.0642  9.5844E-12
1067012.
 950.400 -1.09E-16  6.3464E-10 -1.2535E-10 -1.8490E-18   8527.0642  1.2291E-11
1078532.
 960.000 -1.22E-16      0.0000      0.0000 -1.2843E-18   8527.0642  1.3822E-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      =     -.00030813
Maximum bending moment           =      -1660602. lbs-in
Maximum shear force              =    36247.22432 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	185890.	.6240000	-1660602.	36247.2243

The analysis ended normally.