CALCULATIONS FOR:

REPORT OF
PRELIMINARY GEOTECHNICAL EXPLORATION
FOR
PROPOSED SOUTH INNERBELT TRENCH RETAINING WALLS
PROJECT FRA-70-8.93 (PID No. 77369)
FRANKLIN COUNTY, OHIO

Calculations: Drilled Shaft Retaining Wall
Calculations: Soldier Pile Retaining Wall Using Ground Anchors
Calculations: Check of Axial Capacity of Drilled Shafts
Calculations: Retaining Walls Supported on Spread Footings

Calculations: Drilled Shaft Retaining Wall

Summary of Analyses for Profile A and Profile B
Example Calculations

Summary of Laterally Loaded Drilled Shaft Analyses

Performed by: SJR Date: 03/25/09

Laterally loaded analyses performed using LPILE 5.0

LPILE Parameters

E_c= 3605000 psi f'_c = 4000 psi

Using distributed lateral loads along the retained wall height

STRENGTH I used for gauging the structural capacity requirements of the drilled shafts

SERVICE I used for estimating the deflection of the proposed retaining wall system

βa= variable p-y modification factor

Type 3 Analysis **Computation of Moment-Curvature Relations and Nominal Moment Capacity and Pile Response Using Nonlinear EI**

Profile A **Based upon boring B-029**

Top-down Construction, Drilled Shaft Retaining Wall

Rotation at Pile Cap (ground level/base of wall)

Deflection of wall due to angular rotation at the base of wall

Total Deflection at top of wall

Retained Height (ft)	Drilled Shaft Dia. (in)	c-c Spacing (in)	p-y reduction factor, βa	Reinforc. Ratio ρ (%)	1% of Retained Height (in)	Loading Case	V_{max} (kips)	M_{max} (k-ft)	δ_{GRD} (in)	Θ_{GRD} (rad)	Θ_{GRD} (deg)	δ_{ANG} (in)	δ_{max} (in)
25	36	72	0.783	2	3.0	STRENGTH I	122.6	1,246	-	-	-	-	-
25	36	72	0.783	2	3.0	SERVICE I	79.8	809	0.42	0.00813	0.4659	2.44	4.24
25	36	60	0.707	2	3.0	STRENGTH I	102.2	1,050	-	-	-	-	-
25	36	60	0.707	2	3.0	SERVICE I	66.5	679	0.37	0.00697	0.3995	2.09	3.59
25	36	48	0.623	2	3.0	STRENGTH I	81.7	845	-	-	-	-	-
25	36	48	0.623	2	3.0	SERVICE I	53.2	547	0.32	0.00578	0.3314	1.74	2.92
25	48	120	0.889	2	3.0	STRENGTH I	204.3	2,202	-	-	-	-	-
25	48	120	0.889	2	3.0	SERVICE I††	133.1	1,428	0.41	0.00573	0.3283	1.72	2.76
25	48	96	0.783	2	3.0	STRENGTH I	163.4	1,777	-	-	-	-	-
25	48	96	0.783	2	3.0	SERVICE I	106.4	1,153	0.35	0.00476	0.2728	1.43	2.24
25	48	72	0.667	2	3.0	STRENGTH I	122.6	1,348	-	-	-	-	-
25	48	72	0.667	2	3.0	SERVICE I	79.8	876	0.29	0.00373	0.2139	1.12	1.70
25	48	48	0.529	2	3.0	STRENGTH I	81.7	915	-	-	-	-	-
25	48	48	0.529	2	3.0	SERVICE I	53.2	604	0.21	0.00253	0.1452	0.76	1.09
35	60	96	0.690	2	4.2	STRENGTH I	312.3	4,568	-	-	-	-	-
35	60	96	0.690	2	4.2	SERVICE I	204.7	2,988	0.60	0.00635	0.3637	2.67	4.39
35	60	84	0.640	2	4.2	STRENGTH I	273.3	4,020	-	-	-	-	-
35	60	84	0.640	2	4.2	SERVICE I	179.1	2,630	0.55	0.00567	0.3248	2.38	3.90
45	72	108	0.667	2	5.4	STRENGTH I	NS†	NS†	-	-	-	-	-
45	72	108	0.667	2	5.4	SERVICE I	449.5	9,893	4.20	0.02187	1.2528	11.81	18.04
45	72	72	0.529	2	5.4	STRENGTH I	449.2	10,433	-	-	-	-	-
45	72	72	0.529	2	5.4	SERVICE I	298.0	6,596	2.48	0.01271	0.7281	6.86	10.67
45	84	120	0.648	2	5.4	STRENGTH I	NS†	NS†	-	-	-	-	-
45	84	120	0.648	2	5.4	SERVICE I	490.7	11,431	2.60	0.01232	0.7061	6.66	10.40
45	84	84	0.529	2	5.4	STRENGTH I	494.7	12,630	-	-	-	-	-
45	84	84	0.529	2	5.4	SERVICE I	337.6	7,966	1.76	0.00842	0.4827	4.55	7.08
45	84	84	0.529	3	5.4	SERVICE I	324.9	8,063	1.41	0.00650	0.3724	3.51	5.48
45	96	120	0.600	2	5.4	STRENGTH I	677.2	18,688	-	-	-	-	-
45	96	120	0.600	2	5.4	SERVICE I	466.2	11,752	1.68	0.00756	0.4332	4.08	6.38
45	96	120	0.600	3	5.4	SERVICE I	449.4	11,900	1.35	0.00584	0.3347	3.15	4.95

† *No Solution was Obtained; Excessive Deflection / Overloading of Pile*

†† *Example Calculations Included*

With Axial Loads and Eccentricity equal to 1% of retained height

														P(kips)	e(in)
25	48	120	0.889	2	3.0	STRENGTH I	204.3	2,436	-	-	-	-	-	402.5	3.0
25	48	120	0.889	2	3.0	SERVICE I	133.1	1,535	0.40	0.00552	0.3163	1.66	2.67	230.0	3.0
35	60	84	0.640	2	4.2	STRENGTH I	273.3	4,249	-	-	-	-	-	281.8	4.2
35	60	84	0.640	2	4.2	SERVICE I	179.2	2,737	0.54	0.00559	0.3201	2.35	3.82	161.0	4.2
45	96	120	0.600	**3**	5.4	STRENGTH I	677.4	19,418	-	-	-	-	-	402.5	5.4
45	96	120	0.600	**3**	5.4	SERVICE I	452.3	12,096	1.34	0.00579	0.3316	2.43	4.91	230.0	5.4

Summary of Laterally Loaded Drilled Shaft Analyses

Performed by: SJR Date: 03/25/09

Laterally loaded analyses performed using LPILE 5.0

LPILE Parameters

E_c= 3605000 psi f'_c = 4000 psi

Using distributed lateral loads along the retained wall height

STRENGTH I used for gauging the structural capacity requirements of the drilled shafts

SERVICE I used for estimating the deflection of the proposed retaining wall system

βa= variable p-y modification factor

Type 3 Analysis **Computation of Moment-Curvature Relations and Nominal Moment Capacity and Pile Response Using Nonlinear EI**

Profile B **Based upon boring B-046**

Top-down Construction, Drilled Shaft Retaining Wall

Rotation at Pile Cap (ground level/base of wall)

Deflection of wall due to angular rotation at the base of wall

Total Deflection at top of wall

Retained Height (ft)	Drilled Shaft Dia. (in)	c-c Spacing (in)	p-y reduction factor, βa	Reinforc. Ratio ρ (%)	1% of Retained Height (in)	Loading Case	V_{max} (kips)	M_{max} (k-ft)	δ_{GRD} (in)	Θ_{GRD} (rad)	Θ_{GRD} (deg)	δ_{ANG} (in)	δ_{max} (in)
25	48	120	0.889	2	3.0	STRENGTH I	225.7	2,960	-	-	-	-	-
25	48	120	0.889	2	3.0	SERVICE I	147.0	1,913	0.87	0.00951	0.5448	2.85	4.32
25	48	96	0.783	2	3.0	STRENGTH I	180.6	2,382	-	-	-	-	-
25	48	96	0.783	2	3.0	SERVICE I	117.6	1,535	0.73	0.00777	0.4451	2.33	3.51
25	48	78	0.697	2	3.0	STRENGTH I	146.7	1,941	-	-	-	-	-
25	48	78	0.697	2	3.0	SERVICE I	95.5	1,251	0.61	0.00643	0.3686	1.93	2.87
25	48	72	0.667	2	3.0	STRENGTH I	135.4	1,793	-	-	-	-	-
25	48	72	0.667	2	3.0	SERVICE I	88.1	1,156	0.57	0.00598	0.3427	1.79	2.64
35	60	96	0.690	2	4.2	STRENGTH I	321.1	4,617	-	-	-	-	-
35	60	96	0.690	2	4.2	SERVICE I	210.4	2,995	0.49	0.00589	0.3376	2.48	4.19
35	60	84	0.640	2	4.2	STRENGTH I	280.9	4,050	-	-	-	-	-
35	60	84	0.640	2	4.2	SERVICE I	184.1	2,627	0.45	0.00525	0.3005	2.20	3.71

† *No Solution was Obtained; Excessive Deflection / Overloading of Pile*

†† *Example Calculations Included*

With Axial Loads and Eccentricity equal to 1% of retained height

														P(kips)	e(in)
25	48	78	0.697	2	3.0	STRENGTH I	146.7	2,099	-	-	-	-	-	261.6	3.0
25	48	78	0.697	2	3.0	SERVICE I	95.5	1,322	0.60	0.00625	0.3580	1.87	2.82	149.5	3.0
35	60	84	0.640	2	4.2	STRENGTH I	280.9	4,275	-	-	-	-	-	281.8	4.2
35	60	84	0.640	2	4.2	SERVICE I	184.1	2,731	0.44	0.00515	0.2948	1.54	3.63	161.0	4.2

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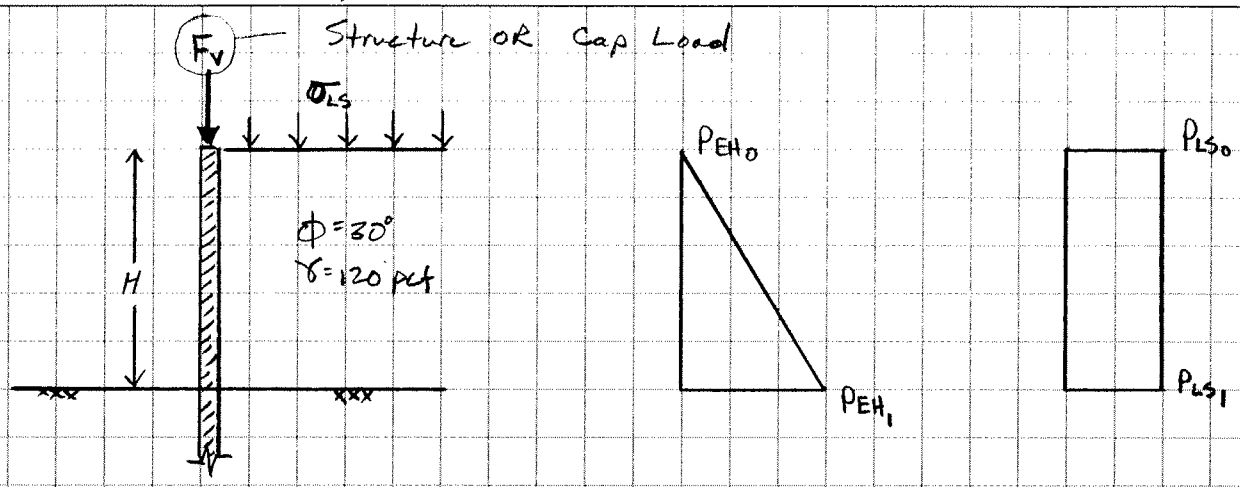
CLIENT MS Consultants
PROJECT FRA-70-8.93
SUBJECT Loads For Drilled Shaft Retaining Wall Design

PROJECT NO. 0221-1004.01
SHEET NO. 1 OF 43
COMP. BY SJR DATE 3/23/09
CHECKED BY ZWT DATE 3-25-09



$P_{EH_0} = 0$

$P_{EH_1} = H \cdot \gamma \cdot K_a \cdot (\gamma_p)$ where; $K_a = \tan^2(45 - \Phi/2) = 0.33$
γ_p = AASHTO LRFD Load Factor

$P_{LS_0} = H_{eq} \cdot \gamma \cdot (\gamma_p)$ where; H_{eq} = Equivalent Height of Soil for Traffic
H_{eq} = 2.0 ft for these cases

$P_{LS_0} = P_{LS_1}$

$P_0 = P_{EH_0} + P_{LS_0}$ and $P_1 = P_{EH_1} + P_{LS_1}$

* Example Calculations *

- For H = 25' - SERVICE I Case 48" Drilled Shafts on 120" Centers

$P_{EH_0} = 0$ $P_{EH_1} = 25'(120 pcf)(0.33)(1.0) = 990$ psf

$P_{LS_0} = P_{LS_1} = (2.0')(120 pcf)(0.33)(1.0) = 79.2$ psf

$P_0 = 0 + 79.2$ psf $= 79.2$ psf $P_1 = 990 + 79.2 = 1069$ psf

LRFD LOAD FACTORS - STRENGTH I

Active Horizontal Earth Pressure: EH - $\gamma_p = 1.5$
Live Load Surcharge: LS - $\gamma_p = 1.75$

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CLIENT MS Consultants
PROJECT FRA-70-8.93
SUBJECT Loads : Axial Load from Proposed Structure (Bridge / Cap)

PROJECT NO. 0221-1004.01
SHEET NO. 2 OF 43
COMP. BY SJK DATE 3/23/09
CHECKED BY SWT DATE 3-25-09

F_v - Vertical Structure Load

From M. Lawler; the larger of the vertical load between the bridge and cap loads should be used for design.

From the available information, it has been determined that an unfactored vertical load of 23 kips per foot of wall length should be used.

Because of the preliminary nature of this computation, meaning that the value of the force components (i.e. wind load, live load) are estimates only, it was decided to conservatively factor the 23^{k} (unfactored) load by a load factor of 1.75 for the STRENGTH case.

$\therefore$ $F_v = 23$ k/ft - SERVICE I

$F_v = 23(1.75) = 40.25$ $^{k}/_{ft}$ - STRENGTH I

Drilled Shaft Retaining Wall Loads
LRFD

Load Factors	γ_p	**Limit State**	*Refer to Table 3.4.1-1*
EH	**1.5**	STRENGTH I	
LS	**1.75**	STRENGTH I	

Wall Height =	**25**	feet	Retained height
Unit Weight of Backfill =	120	pcf	Unit wieght
Traffic Surcharge =	240	psf	
Ka =	0.33		

From Live Load Surcharge

Δp =	79.2	psf	Uniform increase in horizontal earth pressure due to live load surchange
Δp =	138.6	psf	FACTORED Uniform increase in horizontal earth pressure due to live load surchange
P_{LSH} =	**1,980**	**lbs/ft width**	**Total horizontal force from live load surcharge**
P_{LSH} =	**3,465**	**lbs/ft width**	**FACTORED Total horizontal force from live load surcharge**

From Earth Pressure

p_{EH0} =	0	psf	Pressure from horizontal earth pressure at the top of the wall
p_{EH1} =	990	psf	Pressure from horizontal earth pressure at the bottom of the wall
p_{EH0} =	0	psf	FACTORRED Pressure from horizontal earth pressure at the top of the wall
p_{EH1} =	1,485	psf	FACTORED Pressure from horizontal earth pressure at the bottom of the wall
P_{EH} =	**12,375**	**lbs/ft width**	**Total horizontal force from earth pressure**
P_{EH} =	**18,563**	**lbs/ft width**	**FACTORED Total horizontal force from earth pressure**

Combined pressure distribution along retained height

Unfactored

0		0		0	
0	p_0 =	79.2	psf	6.6	lbs/in per foot width
25	p_1 =	1,069	psf	89.1	lbs/in per foot width

Factored

0		0		0	
0	p_0 =	138.6	psf	11.55	lbs/in per foot width
25	p_1 =	1,624	psf	135.3	lbs/in per foot width

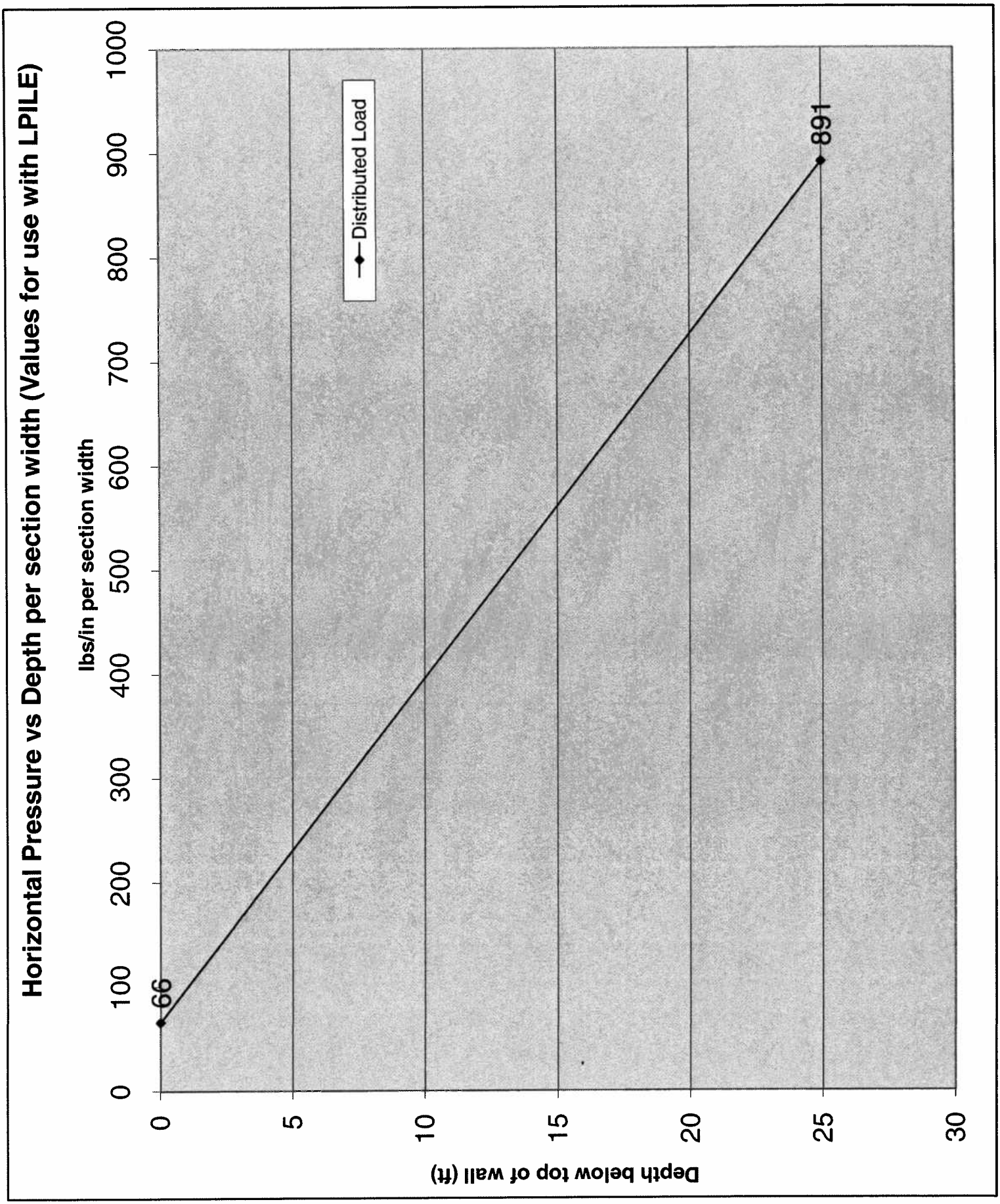
LPILE input

Section Width	**10**	ft
Factored (Y/N)	**n**	

Distributed Loads

p_0 =	**66** lbs/in per section width	0 inches	0 ft
p_1 =	**891** lbs/in per section width	300 inches	25 ft

Sheet 3 of 43
S/K 3.25-09
Sht 3-25-09


Horizontal Pressure vs Depth per section width (Values for use with LPILE)
lbs/in per section width
0
100
200
300
400
500
600
700
800
900
1000
Distributed Load
66
891
Depth below top of wall (ft)
0
5
10
15
20
25
30

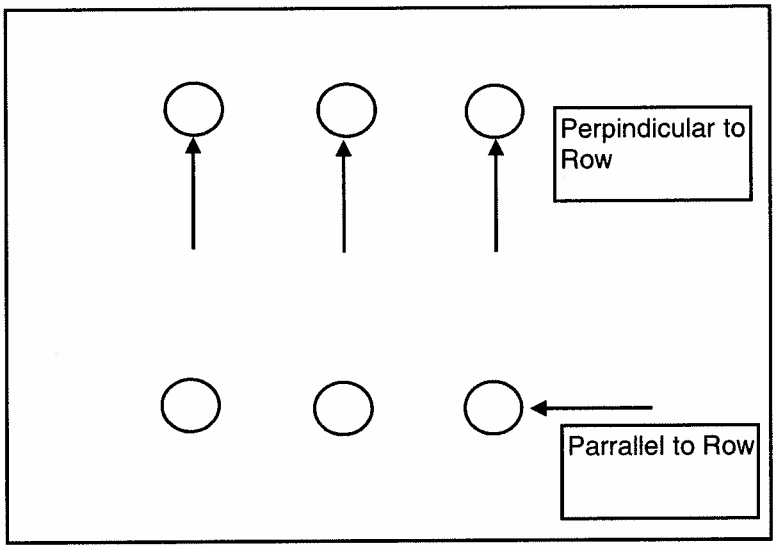
SJK 3-25-09
EWT 3-25-09

p-y reduction factor
Reference: Reese, GROUP 6.0 Technical Manual

Pile Diameter =	4.0	feet	S/B =	2.5
B	48	inches		
Pile Spacing =	10.0	feet		
S	120	inches		
Loading Perpr to Row [1]	Y	Y/N	β_a =	**0.889**

$$\beta_a \approx 0.5292\left(\frac{s}{b}\right)^{0.5659} \quad \text{for} \cdot 1.0 \le \frac{s}{b} \le 3.28$$

$$\beta_a \approx 1.0 \rightarrow \text{for} \cdot \frac{s}{b} \ge 3.28$$



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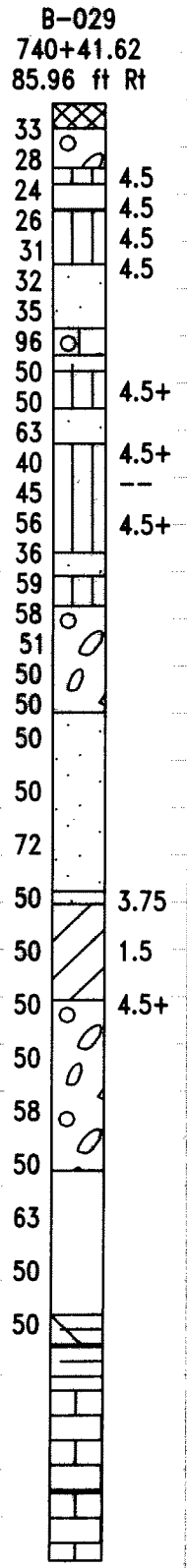
PROJECT FRA-70-8.93
SUBJECT Retaining Wall Evaluations
Profile-A

PROJECT NO. 0221-1004.01
SHEET NO. 6 OF 43
COMP. BY SJR DATE 3-19-09
CHECKED BY GWT DATE 3-25-09

Profile-A Based upon boring B-029 - West Trench



Depth

0 — Ex. Ground Surface

Sandy Silt & Sand and Gravel
$\gamma = 125$ pcf

25' — Base of 25' Wall — $\gamma_z^{*35'}$

Sandy Silt (Till)
$\bar{N}_{60} = 45$ bpf $\gamma = 125$ pcf
32'

Use p-y curve No. 1

Sandy Silt (Till)
$\bar{N}_{60} = 58$ bpf $\gamma = 125$ pcf
42'

Use p-y curve No. 2

Sand and Gravel

$\bar{N}_{60} \simeq 72$ bpf $\phi' = 38°$
$\gamma = 130$ pcf

74'

Silt and Clay
$S_u = 1500$ psf $\gamma = 125$
84'

Sand and Gravel

$\bar{N}_{60} \simeq 61+$ bpf $\phi' = 38°$
$\gamma = 130$ pcf

114'

Shale & Limestone

Wall Heights Evaluated:
25, 35, and 45'

* Assume Groundwater level at the base of the proposed retaining walls.

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

LPILE Plus for Windows, Version 5.0 (5.0.5)

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

This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations: M:\proj\0221\1004\01\geotechnical\Retaining Walls\LPILE\Final DS Wall\
Name of input data file: Profile A, 25 ft 48 in on 120 in centers SERVICE.lpd
Name of output file: Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo
Name of plot output file: Profile A, 25 ft 48 in on 120 in centers SERVICE.lpp
Name of runtime file: Profile A, 25 ft 48 in on 120 in centers SERVICE.lpr

Time and Date of Analysis

Date: March 23, 2009 Time: 10:57:14

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 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- User-specified p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- 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

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

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 = 1200.00 in
Depth of ground surface below top of pile = 300.00 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	48.00000000	260576.0000	1810.0000	3605000.
2	1200.0000	48.00000000	260576.0000	1810.0000	3605000.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 384.000 in

Layer 2 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 384.000 in
Distance from top of pile to bottom of layer = 504.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 504.000 in
Distance from top of pile to bottom of layer = 888.000 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

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 888.000 in
Distance from top of pile to bottom of layer = 1008.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

Distance from top of pile to top of layer = 1008.000 in
Distance from top of pile to bottom of layer = 1368.000 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 168.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	300.00	.03600
2	384.00	.03600
3	384.00	.03600
4	504.00	.03600
5	504.00	.03900
6	888.00	.03900
7	888.00	.03600
8	1008.00	.03600
9	1008.00	.03900
10	1368.00	.03900

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	504.000	.00000	38.00	------	------
2	888.000	.00000	38.00	------	------
3	888.000	10.40000	.00	------	------
4	1008.000	10.40000	.00	------	------
5	1008.000	.00000	38.00	------	------
6	1368.000	.00000	38.00	------	------

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.

p-y Modification Factors

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	300.000	.8890	1.0000
2	1368.000	.8890	1.0000

User-specified p-y Curves

User-specified p-y curves defined using 4 curves.

User-specified curve number 1 at depth = 300.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	7820.000
3	1.2400	13840.000
4	1.8700	17690.000
5	2.4900	20700.000
6	3.1100	23470.000
7	3.7300	25630.000
8	4.3600	27380.000
9	4.9800	28880.000
10	5.6000	30330.000
11	100.0000	31590.000

Curve No. 1

User-specified curve number 2 at depth = 384.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	7820.000
3	1.2400	13840.000
4	1.8700	17690.000
5	2.4900	20700.000
6	3.1100	23470.000
7	3.7300	25630.000
8	4.3600	27380.000
9	4.9800	28880.000
10	5.6000	30330.000
11	100.0000	31590.000

User-specified curve number 3 at depth = 384.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	7820.000
3	1.2400	15640.000
4	1.8700	22290.000
5	2.4900	27220.000
6	3.1100	30890.000

Curve No. 2

7	3.7300	33790.000
8	4.3600	36130.000
9	4.9800	38480.000
10	5.6000	40830.000
11	100.0000	43020.000

User-specified curve number 4 at depth = 504.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	7820.000
3	1.2400	15640.000
4	1.8700	22290.000
5	2.4900	27220.000
6	3.1100	30890.000
7	3.7300	33790.000
8	4.3600	36130.000
9	4.9800	38480.000
10	5.6000	40830.000
11	100.0000	43020.000

Curve No. 2

Loading Type

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

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Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	66.00000
2	300.000	891.00000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = .000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = .000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

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Number of pile sections = 1

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 48.0000 In

Material Properties:

Compressive Strength of Concrete	=	4.000 Kip/In**2
Yield Stress of Reinforcement	=	60. Kip/In**2
Modulus of Elasticity of Reinforcement	=	29000. Kip/In**2
Number of Reinforcing Bars	=	16
Area of Single Bar	=	2.25000 In**2
Number of Rows of Reinforcing Bars	=	9
Cover Thickness (edge to bar center)	=	3.000 In

Unfactored Axial Squash Load Capacity = 8190.10 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	2.250000	21.0000
2	4.500000	19.4015
3	4.500000	14.8492
4	4.500000	8.0364
5	4.500000	.0000
6	4.500000	-8.0364
7	4.500000	-14.8492
8	4.500000	-19.4015
9	2.250000	-21.0000

Axial Thrust Force = .00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
1176132.	1.176132E+12	.00000100	.00002407	24.07269287
5797752.	1.159550E+12	.00000500	.00012036	24.07196045
5797752.	6.441946E+11	.00000900	.00013188	14.65338135
5797752.	4.459809E+11	.00001300	.00019094	14.68743896
6947221.	4.086601E+11	.00001700	.00025027	14.72186279
8560777.	4.076561E+11	.00002100	.00030990	14.75701904
10166016.	4.066407E+11	.00002500	.00036982	14.79290771
11762525.	4.056043E+11	.00002900	.00043005	14.82916260
13350302.	4.045546E+11	.00003300	.00049058	14.86614990
14929143.	4.034904E+11	.00003700	.00055144	14.90386963
16498503.	4.024025E+11	.00004100	.00061262	14.94195557

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

18058764.	4.013059E+11	.00004500	.00067415	14.98114014
19609365.	4.001911E+11	.00004900	.00073603	15.02105713
21150024.	3.990571E+11	.00005300	.00079827	15.06170654
22680438.	3.979024E+11	.00005700	.00086088	15.10308838
24200715.	3.967330E+11	.00006100	.00092388	15.14556885
25710112.	3.955402E+11	.00006500	.00098727	15.18878174
27212079.	3.943780E+11	.00006900	.00104947	15.20965576
28500170.	3.904133E+11	.00007300	.00111052	15.21258545
29361528.	3.813185E+11	.00007700	.00116705	15.15655518
30167822.	3.724423E+11	.00008100	.00122127	15.07745361
30968478.	3.643350E+11	.00008500	.00127576	15.00897217
31558584.	3.545908E+11	.00008900	.00132644	14.90386963
31994986.	3.440321E+11	.00009300	.00137434	14.77789307
32428182.	3.343112E+11	.00009700	.00142244	14.66436768
32858055.	3.253273E+11	.00010100	.00147074	14.56182861
35162475.	2.684158E+11	.00013100	.00181477	13.85321045
36460150.	2.264606E+11	.00016100	.00213916	13.28668213
37541457.	1.965521E+11	.00019100	.00246012	12.88018799
37931651.	1.716364E+11	.00022100	.00275021	12.44439697
38191855.	1.521588E+11	.00025100	.00304652	12.13751221
38456311.	1.368552E+11	.00028100	.00336114	11.96136475
38543155.	1.239330E+11	.00031100	.00369516	11.88153076
38543155.	1.130298E+11	.00034100	.00403649	11.83721924

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 38151.00628 In-Kip

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = .000 lbs
Specified moment at pile head = .000 in-lbs
Specified axial load at pile head = .000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	2.755	-3.99E-05	1.51E-07	-.008216	3.67E-09	1.18E+12	0.000
12.000	2.657	4752.000	990.000	-.008216	.437677	1.18E+12	0.000
24.000	2.558	23760.	2376.000	-.008216	2.188	1.18E+12	0.000
36.000	2.459	61776.	4158.000	-.008215	5.690	1.18E+12	0.000
48.000	2.361	1.24E+05	6336.000	-.008215	11.380	1.18E+12	0.000
60.000	2.262	2.14E+05	8910.000	-.008213	19.695	1.18E+12	0.000
72.000	2.164	3.37E+05	11880.	-.008210	31.075	1.18E+12	0.000
84.000	2.065	4.99E+05	15246.	-.008206	45.956	1.18E+12	0.000
96.000	1.967	7.03E+05	19008.	-.008200	64.776	1.18E+12	0.000
108.000	1.868	9.55E+05	23166.	-.008191	87.973	1.18E+12	0.000
120.000	1.770	1.26E+06	27720.	-.008180	115.984	1.17E+12	0.000
132.000	1.672	1.62E+06	32670.	-.008165	149.248	1.17E+12	0.000
144.000	1.574	2.04E+06	38016.	-.008146	188.201	1.17E+12	0.000
156.000	1.477	2.53E+06	43758.	-.008123	233.282	1.16E+12	0.000
168.000	1.379	3.09E+06	49896.	-.008094	284.927	1.16E+12	0.000
180.000	1.282	3.73E+06	56430.	-.008059	343.576	1.16E+12	0.000

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

192.000	1.186	4.45E+06	63360.	-.008016	409.665	1.16E+12	0.000
204.000	1.090	5.25E+06	70686.	-.007966	483.633	1.16E+12	0.000
216.000	.994687	6.14E+06	78408.	-.007854	565.916	4.33E+11	0.000
228.000	.901465	7.13E+06	86526.	-.007664	656.952	4.09E+11	0.000
240.000	.810757	8.22E+06	95040.	-.007438	757.180	4.08E+11	0.000
252.000	.722952	9.41E+06	1.04E+05	-.007178	867.037	4.07E+11	0.000
264.000	.638477	1.07E+07	1.13E+05	-.006881	986.961	4.06E+11	0.000
276.000	.557800	1.21E+07	1.23E+05	-.006543	1117.388	4.05E+11	0.000
288.000	.481433	1.37E+07	1.33E+05	-.006161	1258.758	4.04E+11	0.000
300.000	.409934	1.53E+07	1.16E+05	-.005730	1411.507	4.03E+11	-4596.537
312.000	.343908	1.65E+07	70600.	-.005257	1515.110	4.02E+11	-3856.195
324.000	.283768	1.70E+07	28372.	-.004758	1567.568	4.02E+11	-3181.855
336.000	.229725	1.71E+07	-6174.289	-.004248	1577.826	4.02E+11	-2575.873
348.000	.181819	1.69E+07	-33862.	-.003740	1553.920	4.02E+11	-2038.710
360.000	.139954	1.63E+07	-55510.	-.003245	1502.975	4.03E+11	-1569.291
372.000	.103928	1.55E+07	-71918.	-.002771	1431.216	4.03E+11	-1165.332
384.000	.073454	1.46E+07	-83851.	-.002323	1344.002	4.04E+11	-823.625
396.000	.048184	1.35E+07	-92035.	-.001905	1245.864	4.04E+11	-540.283
408.000	.027731	1.24E+07	-97142.	-.001521	1140.560	4.05E+11	-310.945
420.000	.011679	1.12E+07	-99794.	-.001172	1031.133	4.06E+11	-130.959
432.000	-.000401	9.99E+06	-1.01E+05	-.000859	919.968	4.07E+11	4.497
444.000	-.008945	8.78E+06	-99924.	-.000583	808.863	4.07E+11	100.301
456.000	-.014386	7.59E+06	-98354.	-.000342	699.088	4.08E+11	161.308
468.000	-.017149	6.42E+06	-96232.	-.000139	591.453	4.23E+11	192.291
480.000	-.017728	5.28E+06	-93886.	-2.09E-05	486.368	1.16E+12	198.778
492.000	-.017651	4.17E+06	-91506.	2.80E-05	383.919	1.16E+12	197.915
504.000	-.017057	3.08E+06	-84115.	6.54E-05	284.096	1.16E+12	1033.881
516.000	-.016081	2.15E+06	-71935.	9.24E-05	197.985	1.17E+12	996.181
528.000	-.014840	1.36E+06	-60323.	.000110	125.085	1.17E+12	939.089
540.000	-.013432	7.02E+05	-49481.	.000121	64.641	1.18E+12	867.917
552.000	-.011939	1.71E+05	-39549.	.000125	15.709	1.18E+12	787.323
564.000	-.010424	-2.47E+05	-30617.	.000125	22.782	1.18E+12	701.341
576.000	-.008940	-5.64E+05	-22729.	.000121	51.971	1.18E+12	613.398
588.000	-.007525	-7.93E+05	-15891.	.000114	73.024	1.18E+12	526.329
600.000	-.006206	-9.46E+05	-10078.	.000105	87.097	1.18E+12	442.405
612.000	-.005004	-1.03E+06	-5243.600	9.49E-05	95.302	1.18E+12	363.372
624.000	-.003928	-1.07E+06	-1320.354	8.42E-05	98.688	1.18E+12	290.502
636.000	-.002984	-1.07E+06	1770.492	7.33E-05	98.221	1.18E+12	224.639
648.000	-.002170	-1.03E+06	4115.889	6.26E-05	94.774	1.18E+12	166.261
660.000	-.001482	-9.68E+05	5806.644	5.24E-05	89.123	1.18E+12	115.532
672.000	-.000913	-8.90E+05	6934.005	4.29E-05	81.939	1.18E+12	72.362
684.000	-.000452	-8.01E+05	7586.896	3.43E-05	73.795	1.18E+12	36.454
696.000	-8.98E-05	-7.08E+05	7849.755	2.66E-05	65.168	1.18E+12	7.356
708.000	.000186	-6.13E+05	7800.937	1.99E-05	56.443	1.18E+12	-15.493
720.000	.000387	-5.20E+05	7511.611	1.41E-05	47.924	1.18E+12	-32.728
732.000	.000524	-4.33E+05	7045.099	9.21E-06	39.839	1.18E+12	-45.024
744.000	.000608	-3.51E+05	6456.584	5.22E-06	32.351	1.18E+12	-53.062
756.000	.000649	-2.78E+05	5793.128	2.01E-06	25.567	1.18E+12	-57.514
768.000	.000656	-2.12E+05	5093.936	-4.91E-07	19.545	1.18E+12	-59.018
780.000	.000637	-1.55E+05	4390.811	-2.37E-06	14.307	1.18E+12	-58.169
792.000	.000599	-1.07E+05	3708.740	-3.70E-06	9.840	1.18E+12	-55.509
804.000	.000548	-66324.	3066.575	-4.59E-06	6.109	1.18E+12	-51.519
816.000	.000489	-33234.	2477.752	-5.09E-06	3.061	1.18E+12	-46.619
828.000	.000426	-6857.805	1951.035	-5.30E-06	.631629	1.18E+12	-41.168
840.000	.000362	13591.	1491.232	-5.26E-06	1.252	1.18E+12	-35.466
852.000	.000300	28932.	1099.882	-5.05E-06	2.665	1.18E+12	-29.759
864.000	.000241	39988.	775.876	-4.70E-06	3.683	1.18E+12	-24.242
876.000	.000187	47553.	516.019	-4.25E-06	4.380	1.18E+12	-19.067
888.000	.000139	52372.	401.615	-3.74E-06	4.824	1.18E+12	0.000
900.000	9.73E-05	57192.	150.526	-3.18E-06	5.268	1.18E+12	-41.848
912.000	6.26E-05	55985.	-262.153	-2.60E-06	5.156	1.18E+12	-26.932
924.000	3.48E-05	50900.	-513.528	-2.06E-06	4.688	1.18E+12	-14.964

936.000	1.32E-05	43660.	-637.383	-1.58E-06	4.021	1.18E+12	-5.678
948.000	-3.04E-06	35603.	-663.603	-1.17E-06	3.279	1.18E+12	1.308
960.000	-1.49E-05	27734.	-617.242	-8.48E-07	2.554	1.18E+12	6.419
972.000	-2.34E-05	20789.	-518.316	-6.01E-07	1.915	1.18E+12	10.069
984.000	-2.93E-05	15294.	-382.163	-4.17E-07	1.409	1.18E+12	12.624
996.000	-3.34E-05	11617.	-220.186	-2.80E-07	1.070	1.18E+12	14.373
1008.	-3.60E-05	10010.	-129.554	-1.69E-07	.921920	1.18E+12	.732579
1020.	-3.75E-05	8507.684	-120.291	-7.47E-08	.783589	1.18E+12	.811349
1032.	-3.78E-05	7122.611	-110.203	5.01E-09	.656018	1.18E+12	.869954
1044.	-3.73E-05	5862.812	-99.533	7.13E-08	.539986	1.18E+12	.908351
1056.	-3.61E-05	4733.815	-88.521	1.25E-07	.436002	1.18E+12	.927009
1068.	-3.43E-05	3738.308	-77.398	1.69E-07	.344312	1.18E+12	.926786
1080.	-3.21E-05	2876.257	-66.385	2.02E-07	.264914	1.18E+12	.908817
1092.	-2.95E-05	2145.077	-55.685	2.28E-07	.197569	1.18E+12	.874401
1104.	-2.66E-05	1539.810	-45.489	2.47E-07	.141822	1.18E+12	.824902
1116.	-2.36E-05	1053.329	-35.970	2.60E-07	.097015	1.18E+12	.761666
1128.	-2.04E-05	676.528	-27.284	2.69E-07	.062311	1.18E+12	.685943
1140.	-1.71E-05	398.503	-19.576	2.74E-07	.036704	1.18E+12	.598832
1152.	-1.38E-05	206.709	-12.975	2.77E-07	.019039	1.18E+12	.501237
1164.	-1.05E-05	87.094	-7.605	2.79E-07	.008022	1.18E+12	.393847
1176.	-7.11E-06	24.192	-3.579	2.79E-07	.002228	1.18E+12	.277133
1188.	-3.75E-06	1.198	-1.008	2.80E-07	.000110	1.18E+12	.151362
1200.	-3.99E-07	0.000	0.000	2.80E-07	0.000	1.18E+12	.016640

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	2.75517932 in
Computed slope at pile head	=	-.00821610
Maximum bending moment	=	17130981. lbs-in
Maximum shear force	=	133056.00000 lbs
Depth of maximum bending moment	=	336.00000 in
Depth of maximum shear force	=	288.00000 in
Number of iterations	=	19
Number of zero deflection points	=	4

Summary of Pile-Head 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 = rotational stiffness of pile-headin-lbs/rad

Profile A, 25 ft 48 in on 120 in centers SERVICE.lpo

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 0.000	M= 0.000	0.0000	2.7552	1.7131E+07	133056.

Pile-head Deflection vs. Pile Length

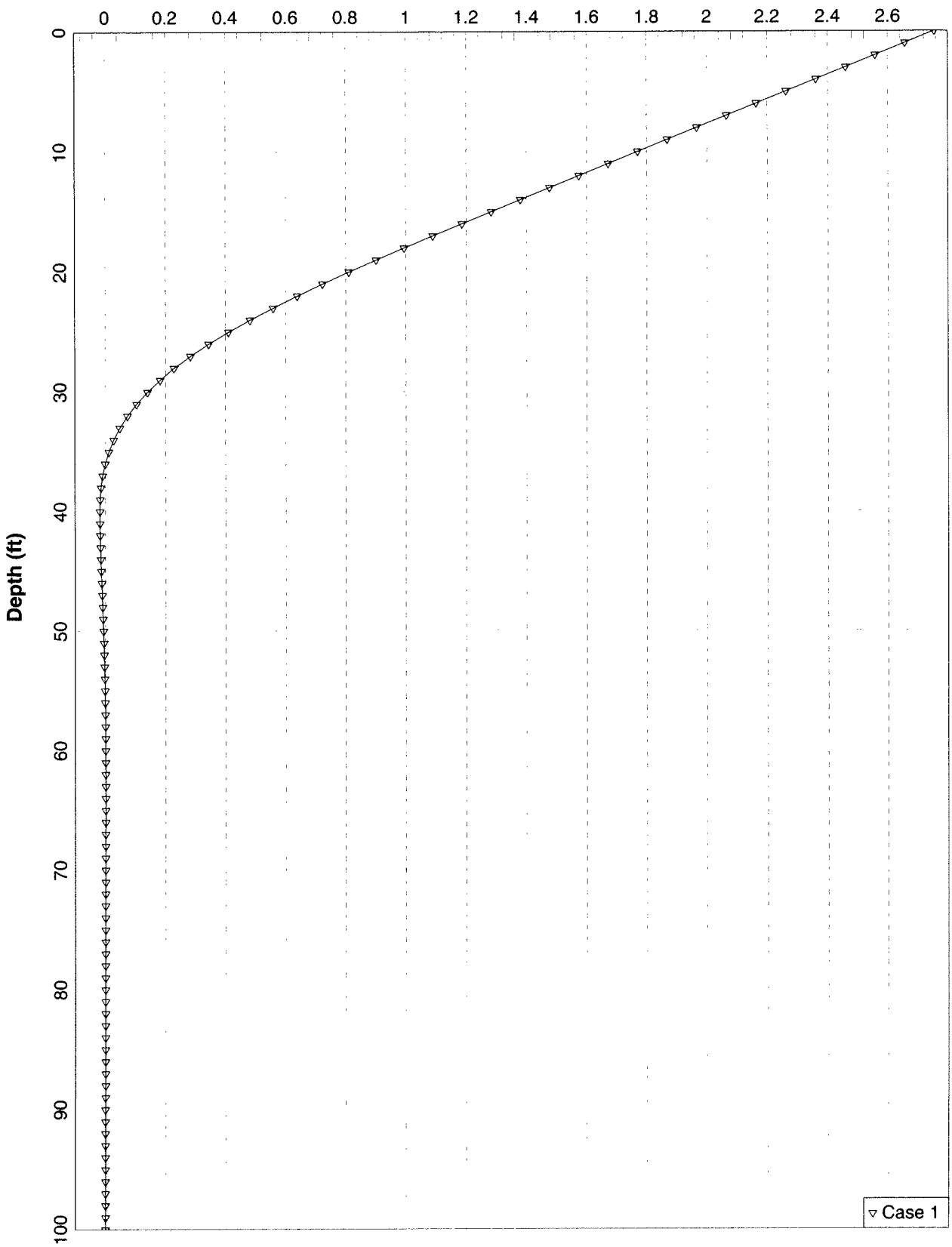
Boundary Condition Type 1, Shear and Moment

Shear = 0. lbs
Moment = 0. in-lbs
Axial Load = 0. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1200.000	2.75517932	17130981.	133056.00000
1140.000	2.98728971	18124358.	140360.21980
1080.000	2.80208807	17344946.	136162.61958
1020.000	2.94452673	17935917.	139832.05432
960.000	2.98612486	18152404.	141419.51902
900.000	2.96091523	18007632.	140889.37370
840.000	2.83563969	17513585.	138253.49836
780.000	2.90890709	17824644.	140360.21790
720.000	2.86126667	17587145.	139304.87744
660.000	2.91507480	17820587.	140889.37183

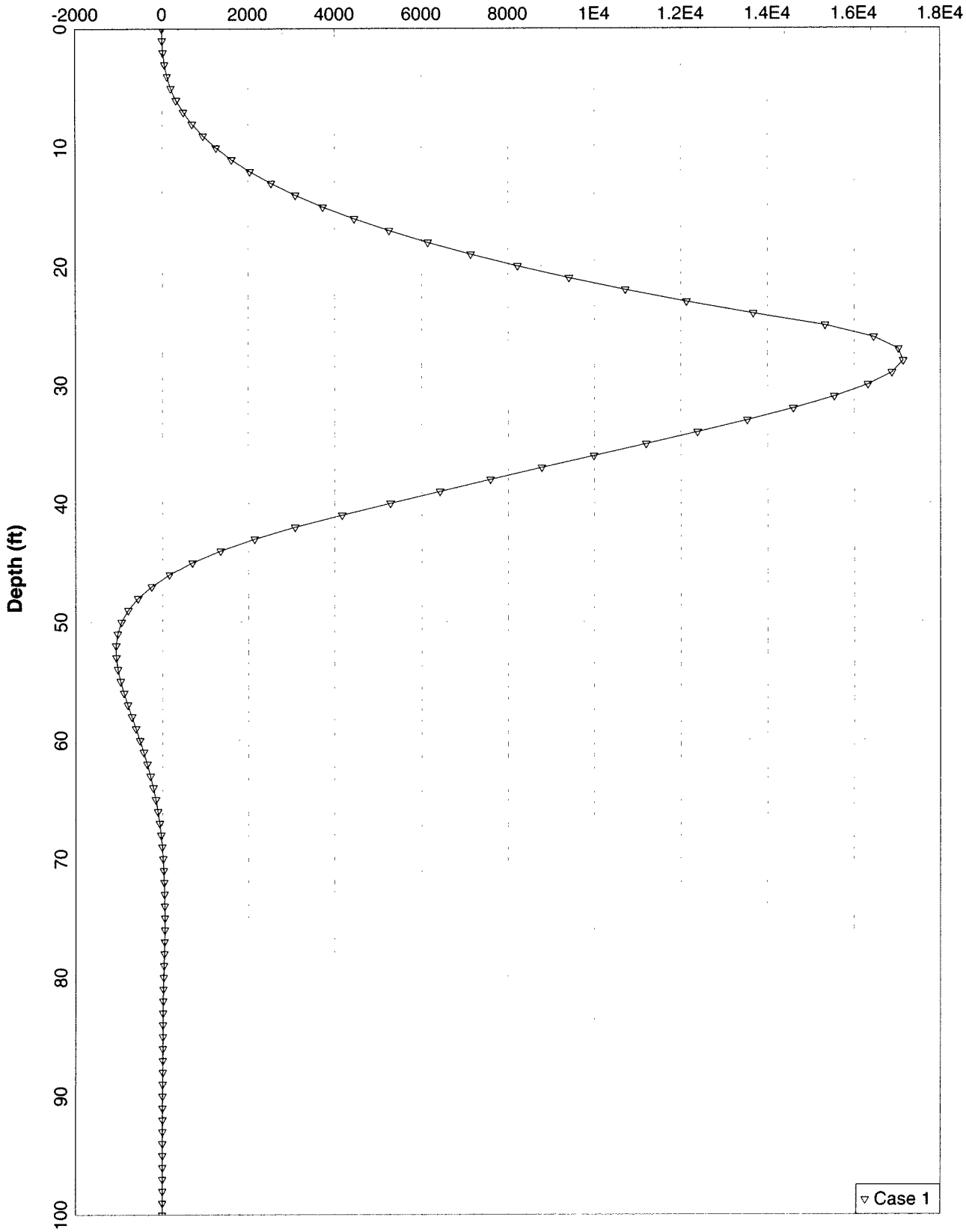
The analysis ended normally.

Profile A, 25 ft 48-inch Diameter Drilled Shafts on 120-inch Centers

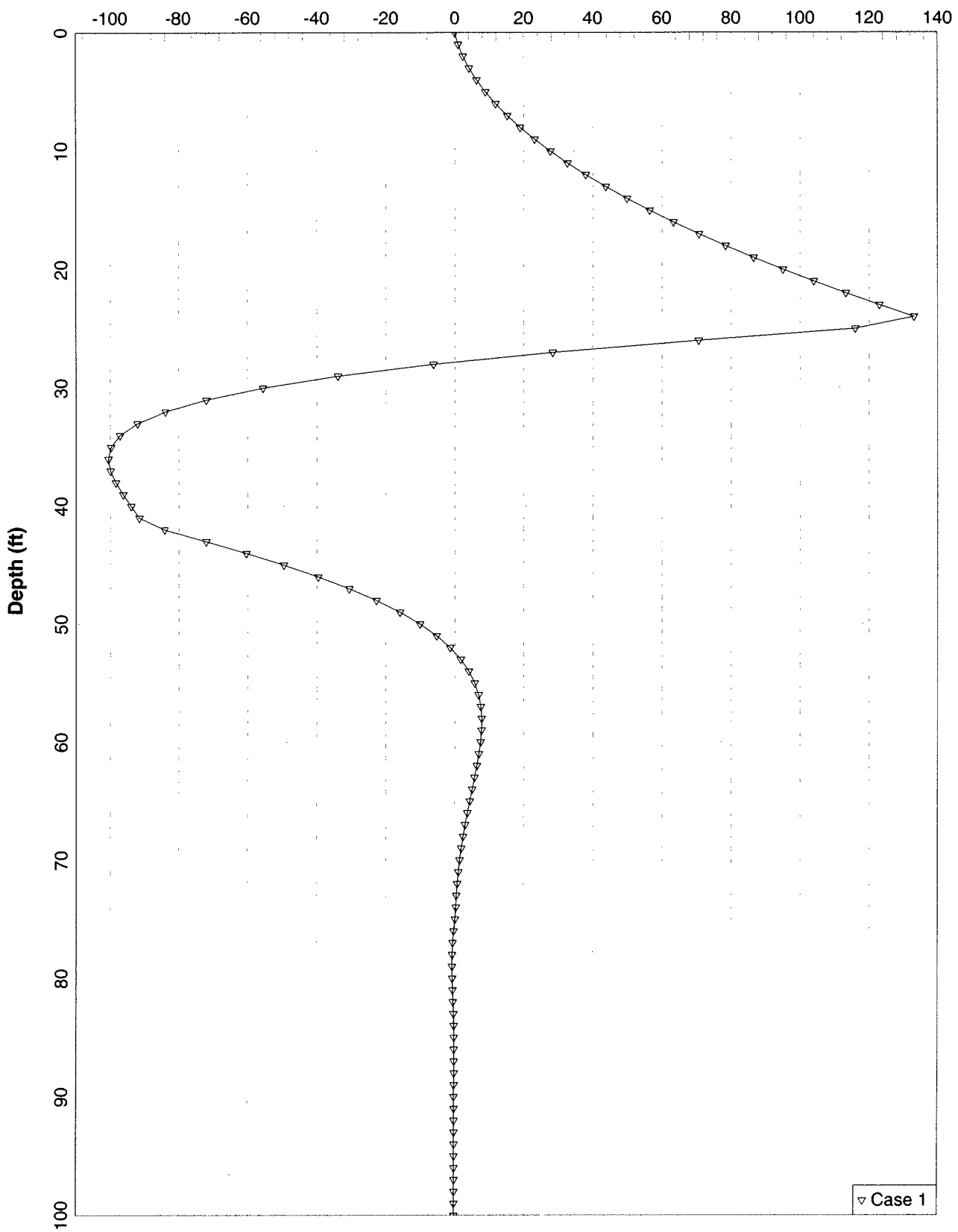


Profile A, 25 ft 48-inch Diameter Drilled Shafts on 120-inch Centers

Unfactored Bending Moment (in-kips)



Profile A, 25 ft 48-inch Diameter Drilled Shafts on 120-inch Centers



Drilled Shaft Retaining Wall Loads
LRFD

Load Factors	γ_p	**Limit State**	*Refer to Table 3.4.1-1*
EH	**1.5**	STRENGTH I	
LS	**1.75**	STRENGTH I	

Wall Height =	**25**	feet	Retained height
Unit Weight of Backfill =	**120**	pcf	Unit wieght
Traffic Surcharge =	**240**	psf	
Ka =	**0.33**		

From Live Load Surcharge

Δp =	79.2	psf	Uniform increase in horizontal earth pressure due to live load surchange
Δp =	138.6	psf	FACTORED Uniform increase in horizontal earth pressure due to live load surchange
P_{LSH} =	**1,980**	**lbs/ft width**	**Total horizontal force from live load surcharge**
P_{LSH} =	**3,465**	**lbs/ft width**	**FACTORED Total horizontal force from live load surcharge**

From Earth Pressure

p_{EH0} =	0	psf	Pressure from horizontal earth pressure at the top of the wall
p_{EH1} =	990	psf	Pressure from horizontal earth pressure at the bottom of the wall
p_{EH0} =	0	psf	FACTORRED Pressure from horizontal earth pressure at the top of the wall
p_{EH1} =	1,485	psf	FACTORED Pressure from horizontal earth pressure at the bottom of the wall
P_{EH} =	**12,375**	**lbs/ft width**	**Total horizontal force from earth pressure**
P_{EH} =	**18,563**	**lbs/ft width**	**FACTORED Total horizontal force from earth pressure**

Combined pressure distribution along retained height

Unfactored

0		0		0	
0	p_0 =	79.2	psf	6.6	lbs/in per foot width
25	p_1 =	1,069	psf	89.1	lbs/in per foot width

Factored

0		0		0	
0	p_0 =	138.6	psf	11.55	lbs/in per foot width
25	p_1 =	1,624	psf	135.3	lbs/in per foot width

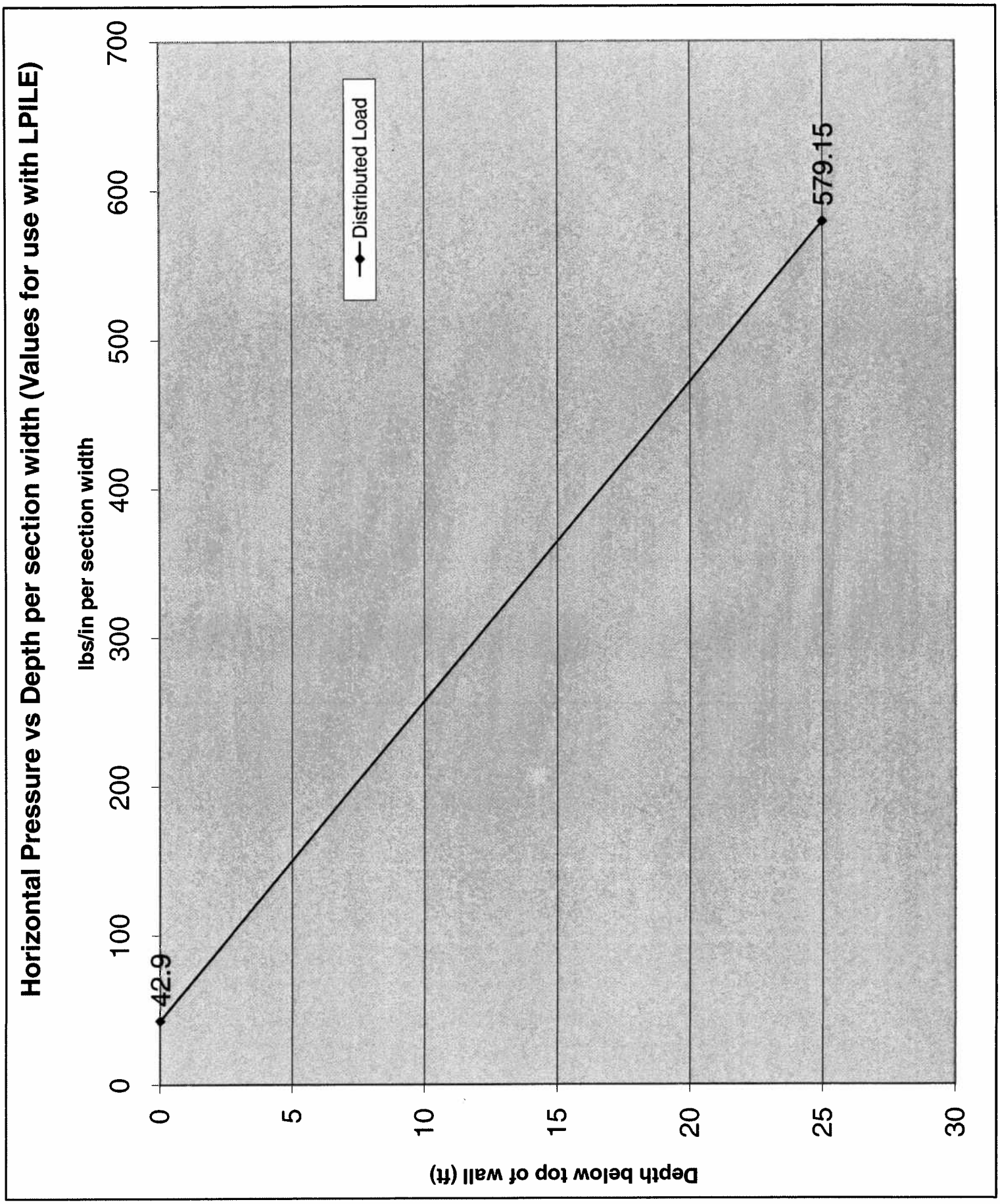
LPILE input

Section Width **6.5** ft
Factored (Y/N) **n**

Distributed Loads

p_0 =	**42.9**	lbs/in per section width	0 inches	0 ft
p_1 =	**579.15**	lbs/in per section width	300 inches	25 ft

sheet 20 of 45
SJK 3-25-09
AGT 3-25-09


Horizontal Pressure vs Depth per section width (Values for use with LPILE)
lbs/in per section width
0
100
200
300
400
500
600
700
Distributed Load
42.9
579.15
Depth below top of wall (ft)
0
5
10
15
20
25
30

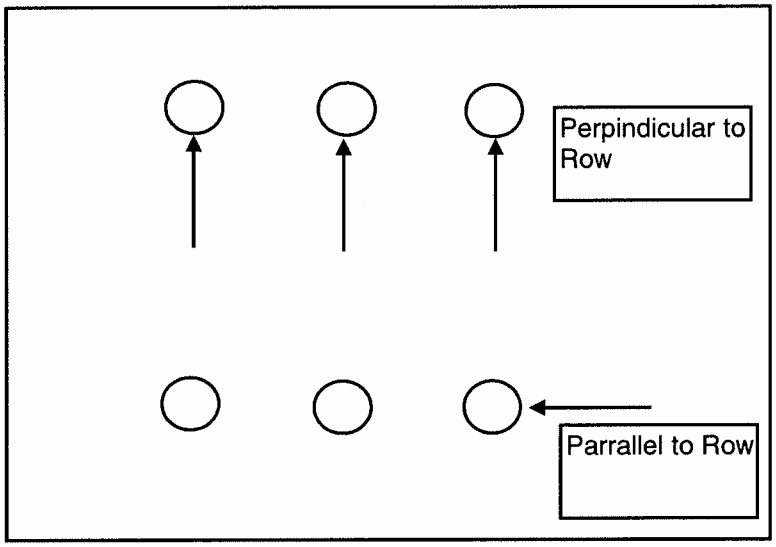
SJR 3-25-09
Chk'd 3-25-09

p-y reduction factor
Reference: Reese, GROUP 6.0 Technical Manual

Pile Diameter =	4.0	feet	S/B =	1.625
B	48	inches		
Pile Spacing =	6.5	feet		
S	78	inches		
Loading Perpr to Row [1]	Y	Y/N	β_a =	**0.697**

$$\beta_a \approx 0.5292\left(\frac{s}{b}\right)^{0.5659} \quad \text{for} \cdot 1.0 \le \frac{s}{b} \le 3.28$$

$$\beta_a \approx 1.0 \rightarrow \text{for} \cdot \frac{s}{b} \ge 3.28$$

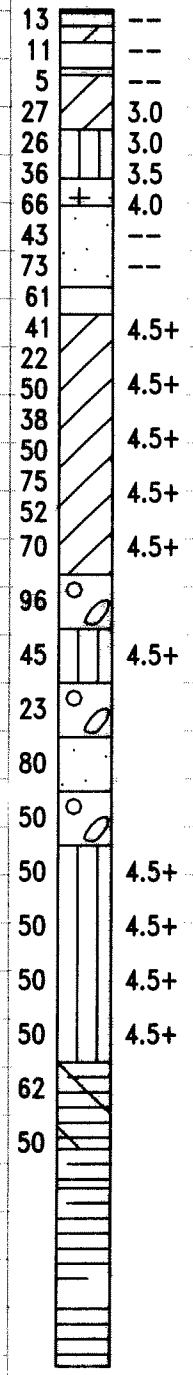


DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT MS Consultants
PROJECT FRA-70-8.93
SUBJECT Retaining Wall Evaluations
Profile - B

PROJECT NO. 0221-1004.01
SHEET NO. 23 OF 43
COMP. BY SJR DATE 3-19-09
CHECKED BY GNT DATE 3-25-09

Profile - B Based upon boring B-046 - East Trench



Depth
0 Ex. Ground Surface

Sand / Silty Clay
$\gamma = 125$ pcf

Base of 25' Wall
25' ▽ 25'
28' Fine Sand $N_{60} = 50$ $\phi' = 36°$ $\gamma = 125$

Silt and Clay (Till)
$\gamma = 125$ pcf
*Use Direct Test Data
52'

57' Gravel $N_{60} = 96$ $\phi' = 38°$ $\gamma = 130$

62' Sandy Silt (Till) *Use Direct Test Data $\gamma = 125$

Sand and Gravel
$N_{60} = 52$ bpf $\phi' = 36°$ $\gamma = 125$
77'

Sandy Silt (Till)
$N_{60} \approx 50+$ bpf
97'

Shale Bedrock

Wall Heights Evaluated:
25' & 35'

*Assume Groundwater level at the base of the proposed retaining wall.

* 28'-35'; Curve No. 3
* 35'-40'; Curve No. 4
* 40'-45'; Curve No. 5
* 45-52'; Curve No. 6
* 57'-62'; Curve No. 7

Use p-y Curve No. 8

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

LPILE Plus for Windows, Version 5.0 (5.0.5)

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

This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations: M:\proj\0221\1004\01\geotechnical\Retaining Walls\LPILE\Final DS Wall\
Name of input data file: Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpd
Name of output file: Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo
Name of plot output file: Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpp
Name of runtime file: Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpr

Time and Date of Analysis

Date: March 25, 2009 Time: 9:43:12

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 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- User-specified p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo
- 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 = 1150.00 in
Depth of ground surface below top of pile = 300.00 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	48.00000000	260576.0000	1810.0000	3605000.
2	1150.0000	48.00000000	260576.0000	1810.0000	3605000.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 9 layers

Layer 1 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 = 336.000 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

Layer 2 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 336.000 in
Distance from top of pile to bottom of layer = 420.000 in

Layer 3 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 420.000 in
Distance from top of pile to bottom of layer = 480.000 in

Layer 4 is modelled using user-specied p-y curves

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 540.000 in

Layer 5 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 540.000 in
Distance from top of pile to bottom of layer = 624.000 in

Layer 6 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 624.000 in
Distance from top of pile to bottom of layer = 684.000 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

Layer 7 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 684.000 in
Distance from top of pile to bottom of layer = 744.000 in

Layer 8 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 744.000 in
Distance from top of pile to bottom of layer = 924.000 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

Layer 9 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 924.000 in
Distance from top of pile to bottom of layer = 1164.000 in

(Depth of lowest layer extends 14.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 18 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	300.00	.03600
2	336.00	.03600
3	336.00	.03600
4	420.00	.03600
5	420.00	.03600
6	480.00	.03600
7	480.00	.03600
8	540.00	.03600
9	540.00	.03600
10	624.00	.03600
11	624.00	.03900
12	684.00	.03900
13	684.00	.03600
14	744.00	.03600
15	744.00	.03600
16	924.00	.03600
17	924.00	.03600
18	1164.00	.03600

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	300.000	.00000	36.00	------	------
2	336.000	.00000	36.00	------	------
3	624.000	.00000	38.00	------	------
4	684.000	.00000	38.00	------	------
5	744.000	.00000	36.00	------	------
6	924.000	.00000	36.00	------	------

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.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	300.000	.6970	1.0000
2	1164.000	.6970	1.0000

User-specified p-y Curves

User-specified p-y curves defined using 12 curves.

User-specified curve number 1 at depth = 336.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0400	530.000
3	.1500	2060.000
4	.3700	4750.000
5	.6200	7440.000
6	.9200	10130.000
7	1.3200	12820.000
8	1.7900	15500.000
9	2.3600	18100.000
10	3.0600	20780.000

Curve No. 3

11	3.8600	23380.000
12	4.8500	25970.000
13	5.9700	28560.000
14	7.3400	31060.000
15	10.1800	35280.000
16	100.0000	37580.000

User-specified curve number 2 at depth = 420.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0400	530.000
3	.1500	2060.000
4	.3700	4750.000
5	.6200	7440.000
6	.9200	10130.000
7	1.3200	12820.000
8	1.7900	15500.000
9	2.3600	18100.000
10	3.0600	20780.000
11	3.8600	23380.000
12	4.8500	25970.000
13	5.9700	28560.000
14	7.3400	31060.000
15	10.1800	35280.000
16	100.0000	37580.000

Curve No. 3

User-specified curve number 3 at depth = 420.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0200	670.000
3	.1000	2210.000
4	.2500	4900.000
5	.4200	7580.000
6	.6200	10370.000
7	.8200	13060.000
8	1.0700	15740.000
9	1.3700	18430.000
10	1.6700	21120.000
11	2.0700	23810.000
12	2.4400	26590.000
13	2.9400	29180.000
14	3.4600	31870.000
15	4.0300	34560.000
16	6.9700	45120.000
17	100.0000	56450.000

Curve No. 4

User-specified curve number 4 at depth = 480.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0200	670.000
3	.1000	2210.000
4	.2500	4900.000
5	.4200	7580.000

6	.6200	10370.000
7	.8200	13060.000
8	1.0700	15740.000
9	1.3700	18430.000
10	1.6700	21120.000
11	2.0700	23810.000
12	2.4400	26590.000
13	2.9400	29180.000
14	3.4600	31870.000
15	4.0300	34560.000
16	6.9700	45120.000
17	100.0000	56450.000

Curve No. 4

User-specified curve number 5 at depth = 480.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0400	720.000
3	.1200	2260.000
4	.2700	4940.000
5	.4200	7630.000
6	.5700	10420.000
7	.7700	13100.000
8	.9500	15890.000
9	1.1700	18580.000
10	1.4200	21260.000
11	1.6400	23950.000
12	1.9200	26740.000
13	2.2400	29420.000
14	2.5900	32110.000
15	2.9600	34800.000
16	4.9300	45550.000
17	100.0000	62160.000

User-specified curve number 6 at depth = 540.000in

Curve No. 5

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0400	720.000
3	.1200	2260.000
4	.2700	4940.000
5	.4200	7630.000
6	.5700	10420.000
7	.7700	13100.000
8	.9500	15890.000
9	1.1700	18580.000
10	1.4200	21260.000
11	1.6400	23950.000
12	1.9200	26740.000
13	2.2400	29420.000
14	2.5900	32110.000
15	2.9600	34800.000
16	4.9300	45550.000
17	100.0000	62160.000

User-specified curve number 7 at depth = 540.000in

Point y p,

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

No.	y in	p, lbs/in
1	.0000	.000
2	.0500	770.000
3	.1400	2300.000
4	.3000	4990.000
5	.4700	7680.000
6	.6700	10460.000
7	.9200	13150.000
8	1.1900	15840.000
9	1.5200	18530.000
10	1.8700	21220.000
11	2.2700	23900.000
12	2.7400	26590.000
13	3.2900	29280.000
14	3.9100	31870.000
15	4.6300	34560.000
16	6.3500	39840.000
17	100.0000	51650.000

Curve No. 6

User-specified curve number 8 at depth = 624.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0500	770.000
3	.1400	2300.000
4	.3000	4990.000
5	.4700	7680.000
6	.6700	10460.000
7	.9200	13150.000
8	1.1900	15840.000
9	1.5200	18530.000
10	1.8700	21220.000
11	2.2700	23900.000
12	2.7400	26590.000
13	3.2900	29280.000
14	3.9100	31870.000
15	4.6300	34560.000
16	6.3500	39840.000
17	100.0000	51650.000

User-specified curve number 9 at depth = 684.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0500	900.000
3	.0700	1190.000
4	.1500	2530.000
5	.3200	5220.000
6	.5500	7910.000
7	.7700	10600.000
8	1.0500	13380.000
9	1.3900	15970.000
10	1.8400	18660.000
11	2.3600	21350.000
12	2.9600	23940.000
13	3.6600	26630.000
14	4.5300	29220.000

Curve No 7

15	5.5500	31810.000
16	8.3100	36810.000
17	100.0000	41700.000

User-specified curve number 10 at depth = 744.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0500	900.000
3	.0700	1190.000
4	.1500	2530.000
5	.3200	5220.000
6	.5500	7910.000
7	.7700	10600.000
8	1.0500	13380.000
9	1.3900	15970.000
10	1.8400	18660.000
11	2.3600	21350.000
12	2.9600	23940.000
13	3.6600	26630.000
14	4.5300	29220.000
15	5.5500	31810.000
16	8.3100	36810.000
17	100.0000	41700.000

Curve No. 7

User-specified curve number 11 at depth = 924.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	8720.000
3	1.2400	15430.000
4	1.8700	19730.000
5	2.4900	23080.000
6	3.1100	26170.000
7	3.7300	28580.000
8	4.3600	30530.000
9	4.9800	32210.000
10	5.6000	33820.000
11	100.0000	35220.000

User-specified curve number 12 at depth = 1164.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.6200	8720.000
3	1.2400	15430.000
4	1.8700	19730.000
5	2.4900	23080.000
6	3.1100	26170.000
7	3.7300	28580.000
8	4.3600	30530.000
9	4.9800	32210.000
10	5.6000	33820.000
11	100.0000	35220.000

Curve No. 8

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

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Loading Type

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

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Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	42.90000
2	300.000	579.20000

Pile-head Loading and Pile-head Fixity Conditions

$P_u = (23\ ^k/_{ft})(6.5\ ft) = 149{,}500$ lbs

$e = 3''$

$P_u \cdot e = 448{,}500$ lb-in

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = .000 lbs
Bending moment at pile head = 448500.000 in-lbs ⟶ $P_u e$
Axial load at pile head = 149500.000 lbs ⟶ P_u

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

---

Number of pile sections = 1

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 48.0000 In

Material Properties:

Compressive Strength of Concrete = 4.000 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2
Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 16

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

Area of Single Bar = 2.25000 In**2
Number of Rows of Reinforcing Bars = 9
Cover Thickness (edge to bar center) = 3.000 In

Unfactored Axial Squash Load Capacity = 8190.10 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	2.250000	21.0000
2	4.500000	19.4015
3	4.500000	14.8492
4	4.500000	8.0364
5	4.500000	.0000
6	4.500000	-8.0364
7	4.500000	-14.8492
8	4.500000	-19.4015
9	2.250000	-21.0000

Axial Thrust Force = 149500.00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
1170609.	1.170609E+12	.00000100	.00004387	43.86639404
5789450.	1.157890E+12	.00000500	.00014050	28.10064697
5789450.	6.432722E+11	.00000900	.00018021	20.02313232
6702062.	5.155432E+11	.00001300	.00024156	18.58172607
8320488.	4.894405E+11	.00001700	.00030239	17.78778076
9927461.	4.727362E+11	.00002100	.00036325	17.29742432
11522552.	4.609021E+11	.00002500	.00042392	16.95684814
13110144.	4.520739E+11	.00002900	.00048532	16.73529053
14686009.	4.450306E+11	.00003300	.00054636	16.55621338
16252803.	4.392649E+11	.00003700	.00060770	16.42437744
17809686.	4.343826E+11	.00004100	.00066935	16.32550049
19357014.	4.301559E+11	.00004500	.00073132	16.25152588
20894611.	4.264206E+11	.00004900	.00079363	16.19659424
22421921.	4.230551E+11	.00005300	.00085628	16.15631104
23938675.	4.199768E+11	.00005700	.00091928	16.12774658
25444535.	4.171235E+11	.00006100	.00098263	16.10870361
26939399.	4.144523E+11	.00006500	.00104635	16.09771729
28423158.	4.119298E+11	.00006900	.00111046	16.09368896
29815721.	4.084345E+11	.00007300	.00117385	16.08013916
30909616.	4.014236E+11	.00007700	.00123341	16.01824951
31714356.	3.915353E+11	.00008100	.00129048	15.93182373
32504593.	3.824070E+11	.00008500	.00134583	15.83331299
33288719.	3.740306E+11	.00008900	.00140147	15.74688721
33765338.	3.630681E+11	.00009300	.00145145	15.60699463
34193705.	3.525124E+11	.00009700	.00150077	15.47186279
34618333.	3.427558E+11	.00010100	.00155030	15.34954834
37124658.	2.833943E+11	.00013100	.00191120	14.58929443
38390261.	2.384488E+11	.00016100	.00224623	13.95172119
39551004.	2.070733E+11	.00019100	.00259134	13.56719971
39987179.	1.809375E+11	.00022100	.00290358	13.13836670
40243943.	1.603344E+11	.00025100	.00321381	12.80401611
40342060.	1.435661E+11	.00028100	.00357220	12.71246338
40399690.	1.299025E+11	.00031100	.00392066	12.60662842

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 40066.98257 In-Kip

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = .000 lbs
Specified moment at pile head = 448500.000 in-lbs
Specified axial load at pile head = 149500.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	2.817	4.49E+05	3.27E-08	-.007670	123.905	1.17E+12	0.000
11.500	2.729	4.65E+05	611.559	-.007665	125.381	1.17E+12	0.000
23.000	2.641	4.89E+05	1459.538	-.007660	127.628	1.17E+12	0.000
34.500	2.553	5.24E+05	2543.935	-.007655	130.898	1.17E+12	0.000
46.000	2.465	5.74E+05	3864.751	-.007650	135.442	1.17E+12	0.000
57.500	2.377	6.40E+05	5421.986	-.007644	141.508	1.17E+12	0.000
69.000	2.289	7.25E+05	7215.641	-.007637	149.348	1.17E+12	0.000
80.500	2.201	8.32E+05	9245.713	-.007630	159.213	1.17E+12	0.000
92.000	2.113	9.64E+05	11512.	-.007621	171.351	1.17E+12	0.000
103.500	2.026	1.12E+06	14015.	-.007611	186.013	1.17E+12	0.000
115.000	1.938	1.31E+06	16754.	-.007599	203.450	1.17E+12	0.000
126.500	1.851	1.53E+06	19730.	-.007585	223.912	1.17E+12	0.000
138.000	1.764	1.79E+06	22942.	-.007568	247.649	1.17E+12	0.000
149.500	1.677	2.09E+06	26391.	-.007549	274.910	1.16E+12	0.000
161.000	1.590	2.42E+06	30076.	-.007527	305.946	1.16E+12	0.000
172.500	1.504	2.81E+06	33997.	-.007501	341.006	1.16E+12	0.000
184.000	1.418	3.23E+06	38155.	-.007471	380.341	1.16E+12	0.000
195.500	1.332	3.71E+06	42549.	-.007437	424.199	1.16E+12	0.000
207.000	1.247	4.24E+06	47180.	-.007397	472.832	1.16E+12	0.000
218.500	1.162	4.82E+06	52047.	-.007352	526.488	1.16E+12	0.000
230.000	1.078	5.46E+06	57151.	-.007301	585.416	1.16E+12	0.000
241.500	.993977	6.16E+06	62491.	-.007213	649.867	5.80E+11	0.000
253.000	.911728	6.92E+06	68067.	-.007074	720.080	5.11E+11	0.000
264.500	.831271	7.75E+06	73880.	-.006907	796.300	4.97E+11	0.000
276.000	.752875	8.64E+06	79929.	-.006715	878.773	4.85E+11	0.000
287.500	.676834	9.61E+06	86215.	-.006496	967.747	4.76E+11	0.000
299.000	.603465	1.06E+07	92737.	-.006249	1063.466	4.67E+11	0.000
310.500	.533113	1.18E+07	95495.	-.005970	1166.178	4.59E+11	-97.787
322.000	.466149	1.29E+07	93699.	-.005660	1267.651	4.53E+11	-214.512
333.500	.402938	1.39E+07	90553.	-.005318	1366.460	4.48E+11	-332.542
345.000	.343841	1.50E+07	70886.	-.004945	1461.162	4.44E+11	-3087.815
356.500	.289204	1.56E+07	38054.	-.004548	1518.191	4.42E+11	-2622.170
368.000	.239234	1.59E+07	10348.	-.004138	1543.215	4.41E+11	-2196.311
379.500	.194025	1.58E+07	-12695.	-.003725	1541.421	4.41E+11	-1811.019
391.000	.153569	1.56E+07	-31539.	-.003315	1517.502	4.42E+11	-1466.239
402.500	.117779	1.51E+07	-46430.	-.002916	1475.660	4.43E+11	-1123.452
414.000	.086502	1.45E+07	-57606.	-.002532	1420.070	4.46E+11	-820.229
425.500	.059533	1.38E+07	-68057.	-.002168	1354.431	4.49E+11	-997.421

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

437.000	.036635	1.30E+07	-77761.	-.001827	1276.586	4.53E+11	-690.184
448.500	.017522	1.20E+07	-84082.	-.001511	1190.282	4.58E+11	-409.141
460.000	.001884	1.10E+07	-86688.	-.001223	1098.946	4.64E+11	-43.999
471.500	-.010609	1.00E+07	-85516.	-.000964	1007.031	4.72E+11	247.721
483.000	-.020289	9.07E+06	-82628.	-.000733	918.095	4.81E+11	254.550
494.500	-.027474	8.14E+06	-79183.	-.000530	832.225	4.92E+11	344.688
506.000	-.032470	7.25E+06	-74858.	-.000352	750.524	5.05E+11	407.365
517.500	-.035566	6.42E+06	-69950.	-.000202	673.758	5.46E+11	446.217
529.000	-.037109	5.64E+06	-64707.	-.000106	602.406	1.16E+12	465.567
540.500	-.038007	4.93E+06	-59685.	-5.36E-05	536.717	1.16E+12	407.954
552.000	-.038341	4.27E+06	-54973.	-7.93E-06	475.988	1.16E+12	411.549
563.500	-.038189	3.67E+06	-50249.	3.14E-05	420.266	1.16E+12	409.912
575.000	-.037618	3.12E+06	-45570.	6.50E-05	369.531	1.16E+12	403.788
586.500	-.036693	2.62E+06	-40984.	9.34E-05	323.710	1.16E+12	393.853
598.000	-.035469	2.17E+06	-36530.	.000117	282.682	1.16E+12	380.720
609.500	-.033999	1.78E+06	-32243.	.000137	246.288	1.17E+12	364.936
621.000	-.032327	1.43E+06	-28149.	.000152	214.337	1.17E+12	346.986
632.500	-.030492	1.13E+06	-26017.	.000165	186.610	1.17E+12	23.750
644.000	-.028530	8.31E+05	-25589.	.000175	159.170	1.17E+12	50.807
655.500	-.026475	5.40E+05	-24873.	.000181	132.348	1.17E+12	73.672
667.000	-.024358	2.59E+05	-23919.	.000185	106.422	1.17E+12	92.187
678.500	-.022212	-10613.	-22778.	.000187	83.574	1.17E+12	106.320
690.000	-.020067	-2.66E+05	-20719.	.000185	107.082	1.17E+12	251.761
701.500	-.017952	-4.88E+05	-17976.	.000181	127.523	1.17E+12	225.229
713.000	-.015893	-6.80E+05	-15535.	.000176	145.220	1.17E+12	199.388
724.500	-.013910	-8.46E+05	-13385.	.000168	160.487	1.17E+12	174.511
736.000	-.012022	-9.88E+05	-11514.	.000159	173.627	1.17E+12	150.833
747.500	-.010247	-1.11E+06	-8672.476	.000149	184.928	1.17E+12	343.322
759.000	-.008597	-1.19E+06	-4992.653	.000138	192.046	1.17E+12	296.647
770.500	-.007081	-1.23E+06	-1841.194	.000126	195.548	1.17E+12	251.433
782.000	-.005704	-1.23E+06	801.945	.000114	195.986	1.17E+12	208.243
793.500	-.004465	-1.21E+06	2962.554	.000102	193.885	1.17E+12	167.515
805.000	-.003364	-1.16E+06	4670.764	9.01E-05	189.742	1.17E+12	129.565
816.500	-.002394	-1.10E+06	5959.726	7.89E-05	184.019	1.17E+12	94.602
828.000	-.001548	-1.03E+06	6864.420	6.85E-05	177.142	1.17E+12	62.736
839.500	-.000819	-9.44E+05	7420.596	5.88E-05	169.500	1.17E+12	33.990
851.000	-.000196	-8.56E+05	7663.854	5.00E-05	161.441	1.17E+12	8.315
862.500	.000331	-7.67E+05	7628.875	4.20E-05	153.280	1.17E+12	-14.399
874.000	.000770	-6.81E+05	7348.816	3.49E-05	145.294	1.17E+12	-34.307
885.500	.001133	-5.99E+05	6854.877	2.86E-05	137.724	1.17E+12	-51.595
897.000	.001428	-5.23E+05	6176.037	2.31E-05	130.781	1.17E+12	-66.464
908.500	.001664	-4.57E+05	5338.962	1.83E-05	124.648	1.17E+12	-79.115
920.000	.001849	-4.00E+05	4368.078	1.41E-05	119.477	1.17E+12	-89.735
931.500	.001988	-3.56E+05	3740.040	1.04E-05	115.399	1.17E+12	-19.489
943.000	.002087	-3.14E+05	3510.334	7.06E-06	111.558	1.17E+12	-20.460
954.500	.002151	-2.75E+05	3271.471	4.17E-06	107.965	1.17E+12	-21.082
966.000	.002183	-2.39E+05	3027.207	1.64E-06	104.629	1.17E+12	-21.399
977.500	.002188	-2.06E+05	2780.822	-5.49E-07	101.553	1.17E+12	-21.451
989.000	.002170	-1.75E+05	2535.147	-2.42E-06	98.738	1.17E+12	-21.275
1001.	.002133	-1.47E+05	2292.610	-4.01E-06	96.182	1.17E+12	-20.905
1012.	.002078	-1.23E+05	2055.266	-5.33E-06	93.880	1.17E+12	-20.372
1024.	.002010	-1.00E+05	1824.835	-6.43E-06	91.826	1.17E+12	-19.703
1035.	.001930	-80512.	1602.734	-7.31E-06	90.012	1.17E+12	-18.923
1047.	.001842	-63319.	1390.115	-8.02E-06	88.429	1.17E+12	-18.054
1058.	.001746	-48512.	1187.893	-8.57E-06	87.065	1.17E+12	-17.115
1070.	.001645	-35968.	996.781	-8.98E-06	85.909	1.17E+12	-16.122
1081.	.001539	-25555.	817.317	-9.29E-06	84.950	1.17E+12	-15.089
1093.	.001431	-17138.	649.892	-9.50E-06	84.175	1.17E+12	-14.028
1104.	.001321	-10575.	494.779	-9.63E-06	83.571	1.17E+12	-12.948
1116.	.001209	-5724.826	352.153	-9.71E-06	83.124	1.17E+12	-11.856
1127.	.001097	-2442.325	222.118	-9.75E-06	82.822	1.17E+12	-10.758
1139.	.000985	-582.566	104.728	-9.77E-06	82.650	1.17E+12	-9.657

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo
1150. .000873 0.000 0.000 -9.77E-06 82.597 1.17E+12 -8.556

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	2.81709279 in
Computed slope at pile head	=	-.00766961
Maximum bending moment	=	15858419. lbs-in
Maximum shear force	=	95494.53821 lbs
Depth of maximum bending moment	=	368.00000 in
Depth of maximum shear force	=	310.50000 in
Number of iterations	=	20
Number of zero deflection points	=	2

Summary of Pile-Head 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 = rotational stiffness of pile-headin-lbs/rad

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 0.000	M= 4.49E+05	149500.	2.8171	1.5858E+07	95494.5382

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

Shear	=	0. lbs
Moment	=	448500. in-lbs
Axial Load	=	149500. lbs

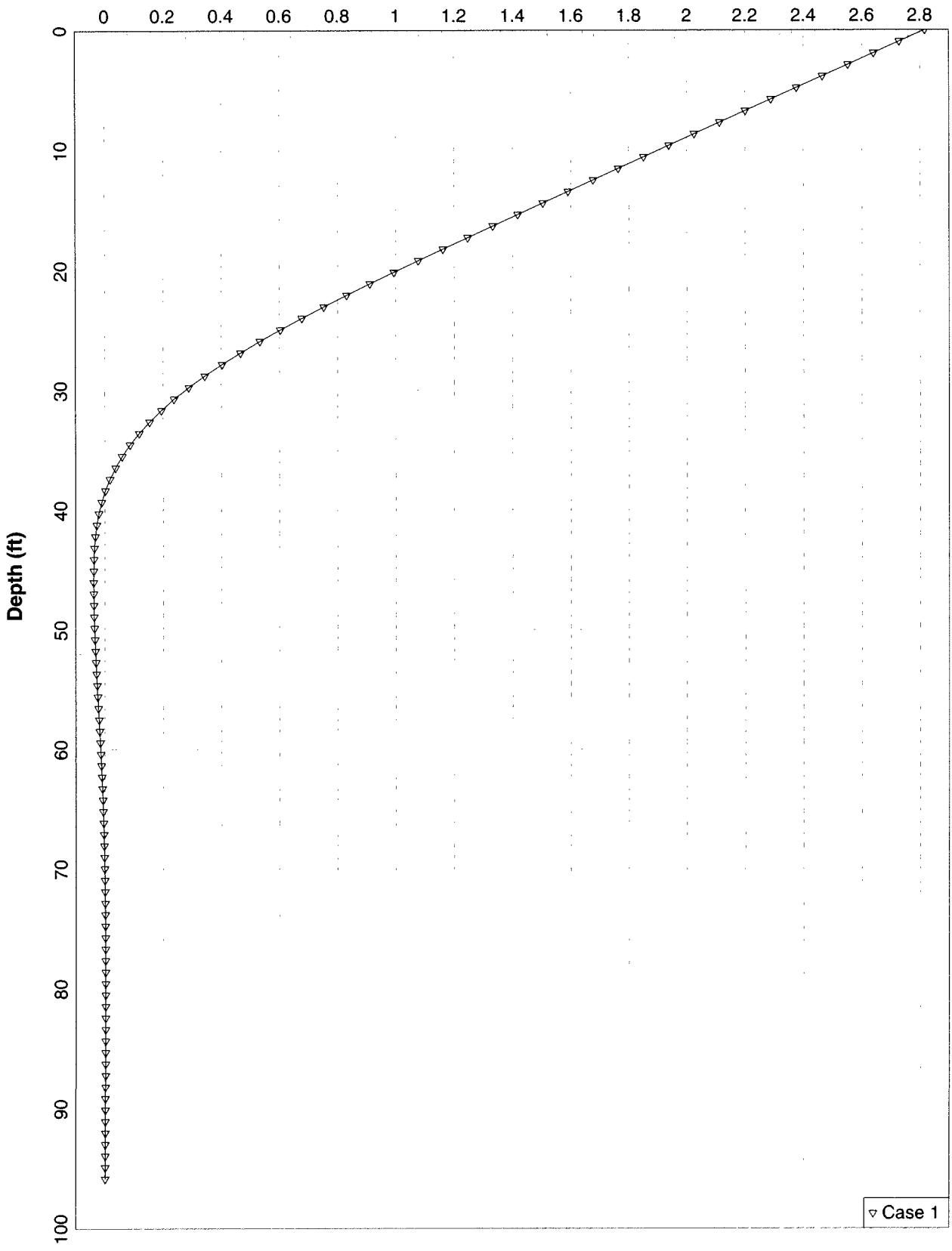
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs

Profile B, 25 ft 48 in on 78 in centers SERVICE w Axial Load.lpo

1150.000	2.81709279	15858419.	95494.53821
1092.500	2.63030496	15257881.	93258.67265
1035.000	2.66222749	15311488.	90396.61648
977.500	2.69370022	15494161.	92089.56998
920.000	2.67358975	15408234.	92573.80086
862.500	2.57776603	15062265.	91823.61012
805.000	2.65772400	15364118.	94179.98165
747.500	2.63529491	15279052.	94681.11545
690.000	2.65868893	15343480.	93283.55556
632.500	2.76818508	15598728.	93459.09734

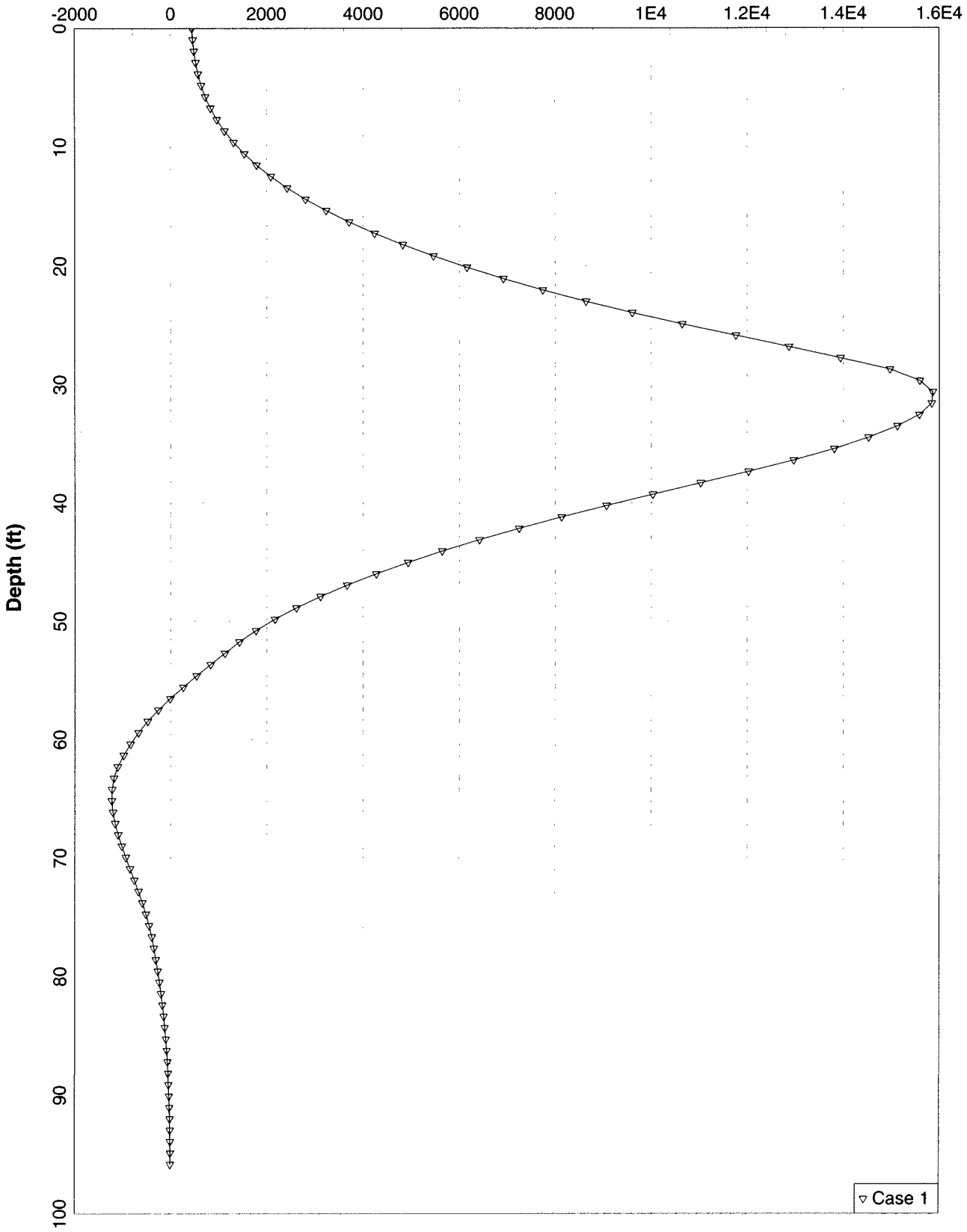
The analysis ended normally.

Profile B, 25 ft, 48-inch Diameter Drilled Shafts on 120-inch Centers

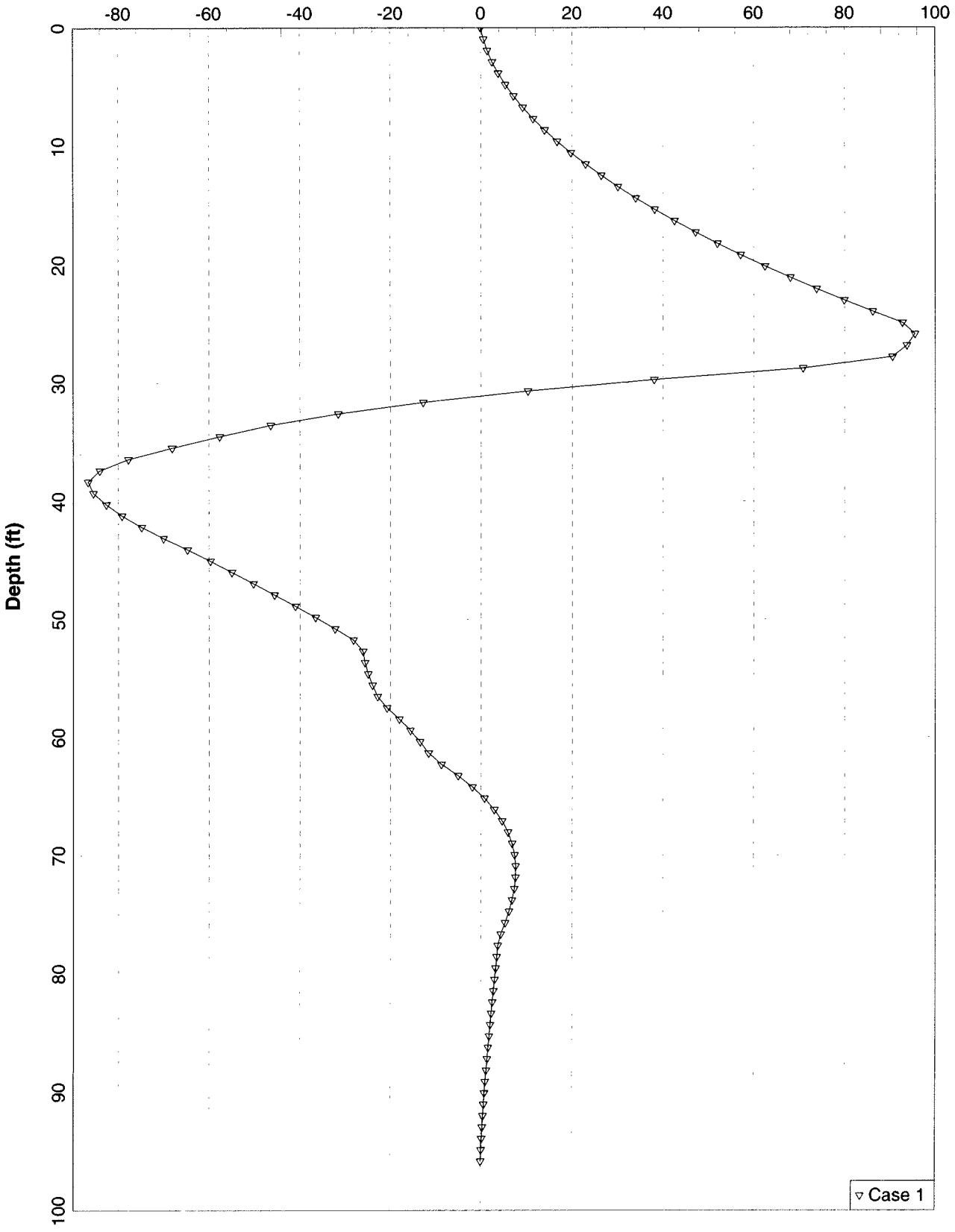


Profile B, 25 ft, 48-inch Diameter Drilled Shafts on 120-inch Centers

Unfactored Bending Moment (in-kips)

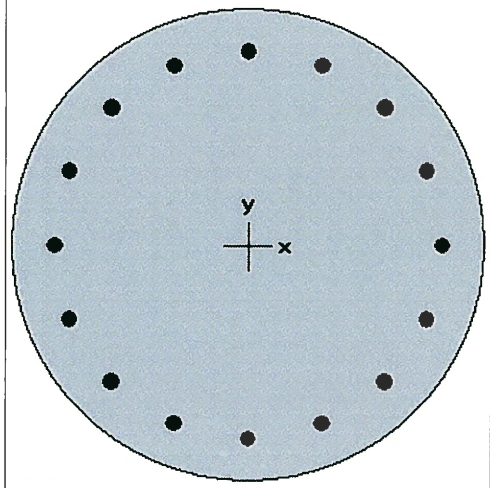


Profile B, 25 ft, 48-inch Diameter Drilled Shafts on 120-inch Centers



Drilled Shaft Interaction Diagram

Profile A, 25-foot Wall, 48-inch Diameter Drilled Shafts on 120-inch c-c Spacing



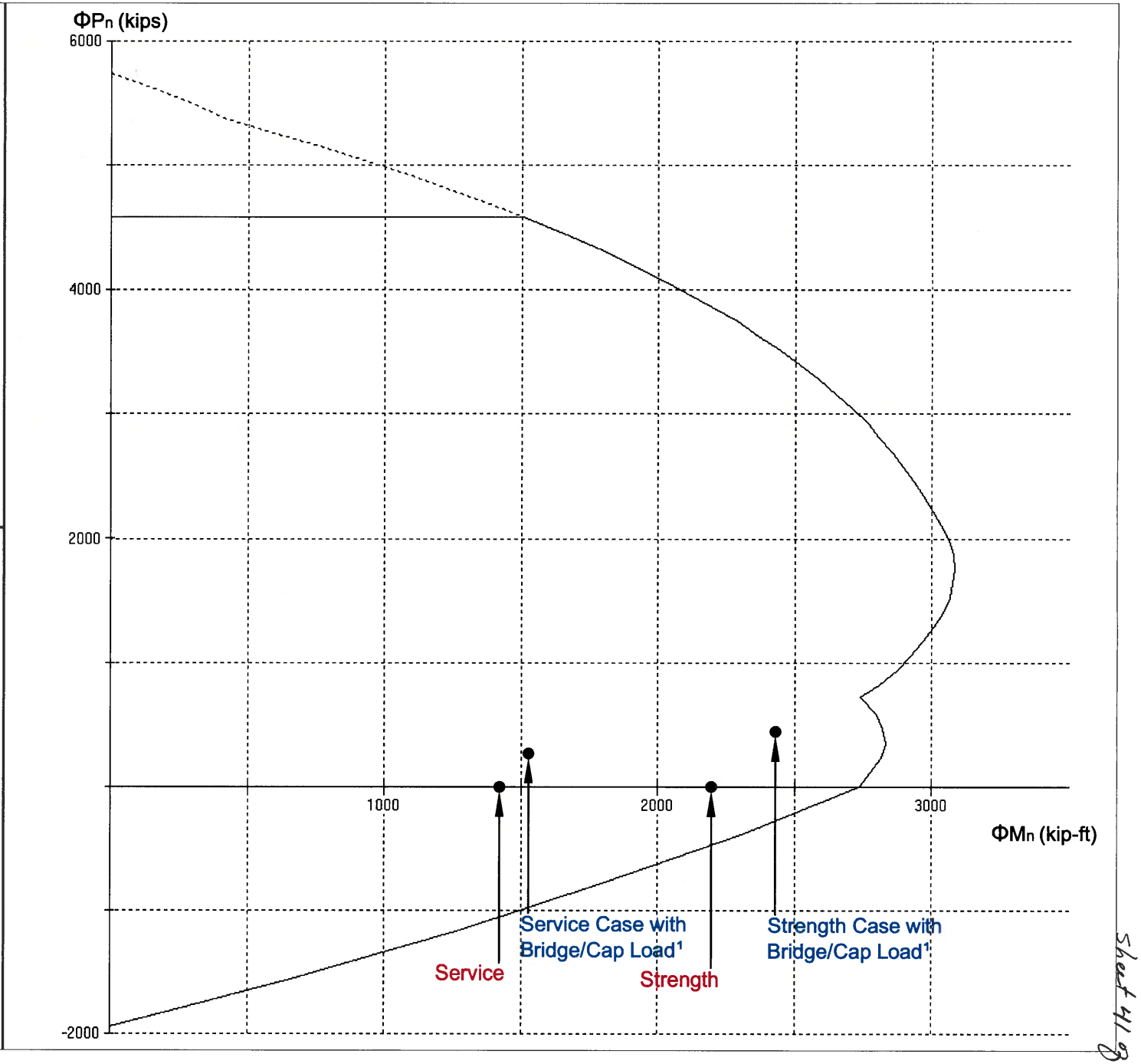
48 in diam.
1.99% reinf.

MATERIAL:
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
Ag = 1809.56 in^2
Ix = 260576 in^4
Iy = 260576 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
16 #14 bars @ 1.989%
As = 36 in^2
Confinement: Tied
Clear Cover = 3.5 in
Spacing = 5.97542 in

SLENDERNESS:
N/A

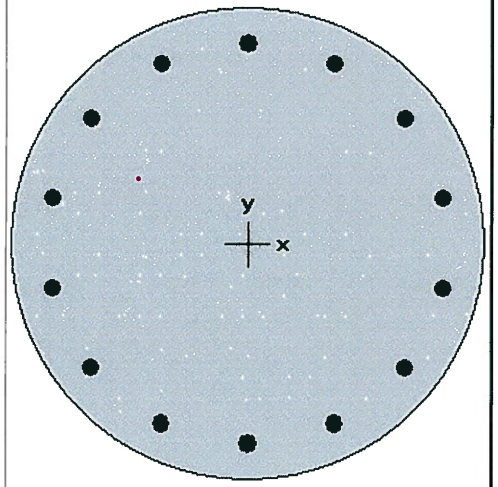


[1]Load cases which include vertical loads from bridges/caps assume a worst case eccentricity for the load placement. Eccentricity is equal to 1 percent of the retained height.

Sheet 41 of 43

Drilled Shaft Interaction Diagram

Profile A, 35-foot Wall, 60-inch Diameter Drilled Shafts on 84-inch c-c Spacing



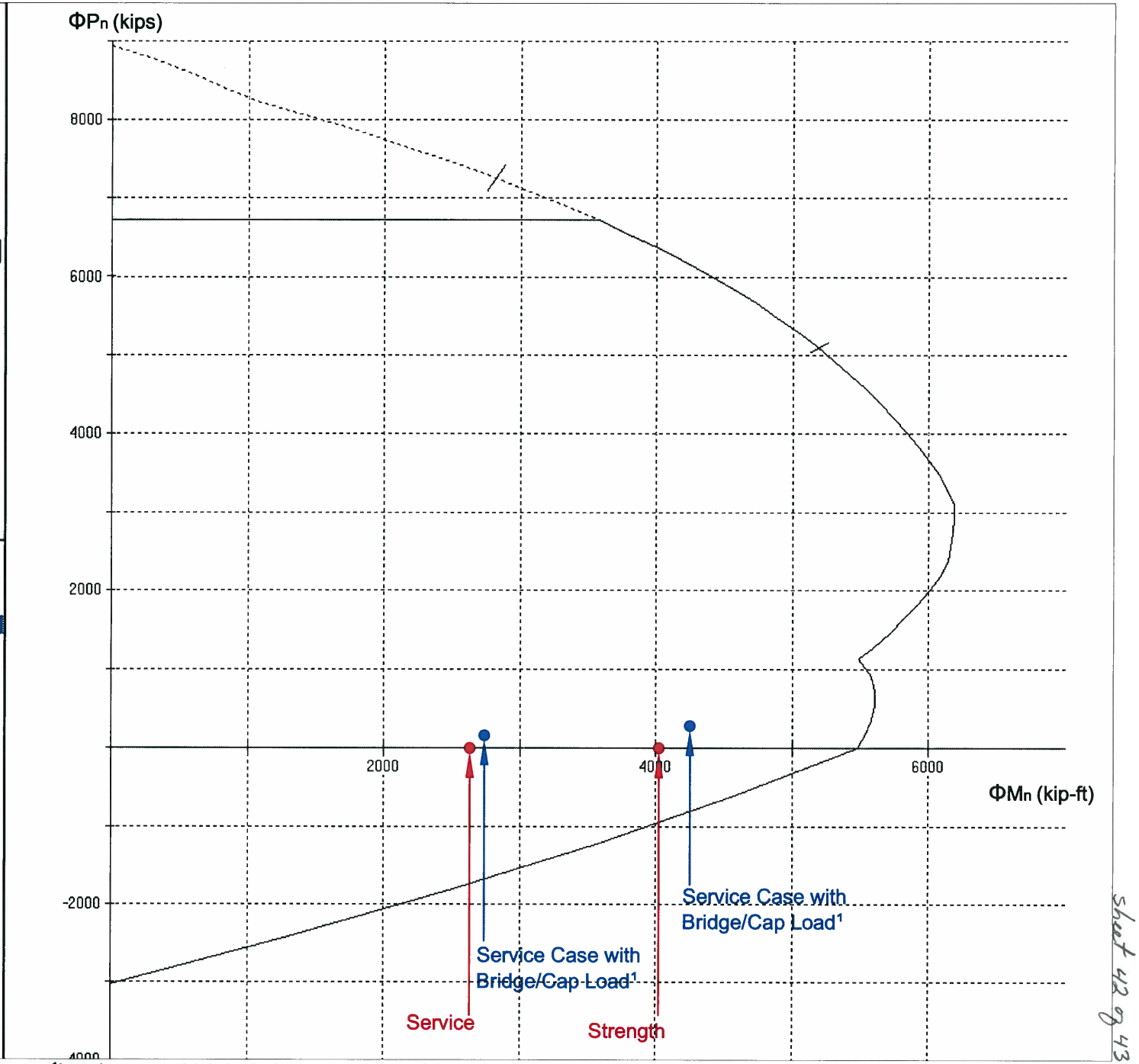
60 in diam.
1.98% reinf.

MATERIAL:
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.9
fy = 60 ksi
Es = 29000 ksi

SECTION:
Ag = 2827.43 in^2
Ix = 636173 in^4
Iy = 636173 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
14 #18 bars @ 1.981%
As = 56 in^2
Confinement: Other
Clear Cover = 3.5 in
Spacing = 9.03438 in

SLENDERNESS:
N/A



[1]Load cases which include vertical loads from bridges/caps assume a worst case eccentricity for the load placement. Eccentricity is equal to 1 percent of the retained height.

shaft 42 of 43

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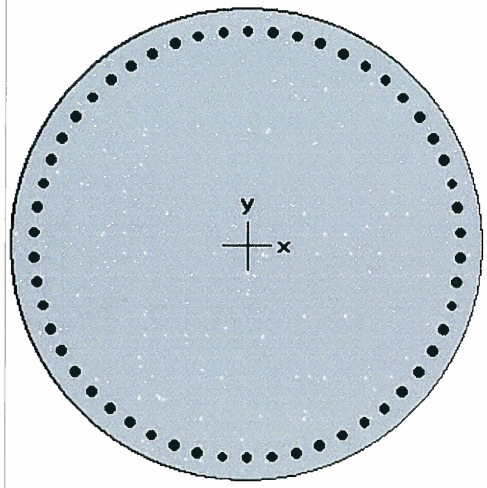
Drilled Shaft Interaction Diagram

Profile A, 45-foot Wall, 96-inch Diameter Drilled Shafts on 120-inch c-c Spacing



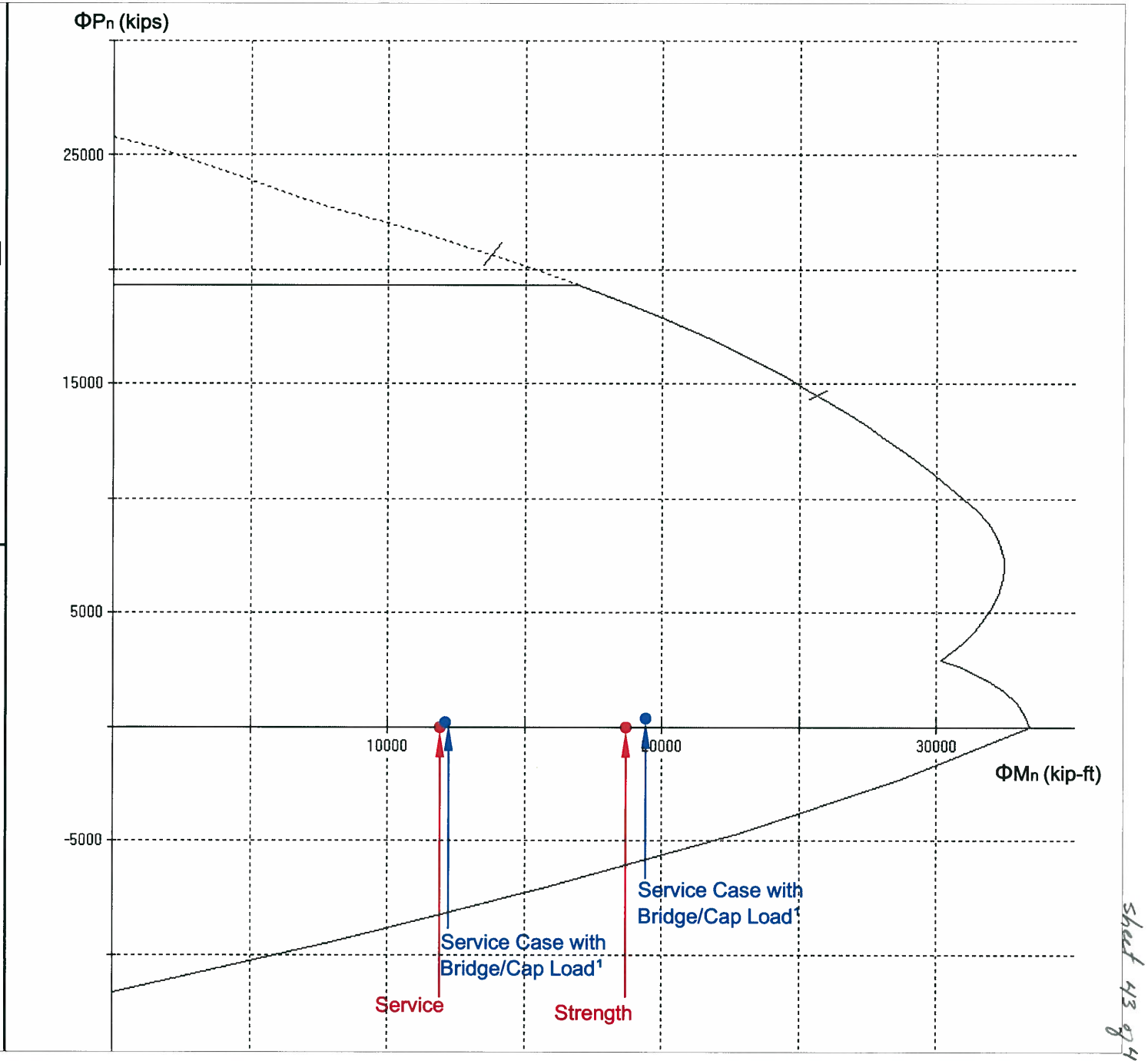
96 in diam.
2.98% reinf.

MATERIAL:
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.9
fy = 60 ksi
Es = 29000 ksi

SECTION:
Ag = 7238.23 in^2
Ix = 4.16922e+006 in^4
Iy = 4.16922e+006 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
54 #18 bars @ 2.984%
As = 216 in^2
Confinement: Other
Clear Cover = 3.5 in
Spacing = 2.78666 in

SLENDERNESS:
N/A



[1]Load cases which include vertical loads from bridges/caps assume a worst case eccentricity for the load placement. Eccentricity is equal to 1 percent of the retained height.

sheet 413 of 413

Calculations: Soldier Pile Retaining Wall Using Ground Anchors (tiebacks)

Summary of Analyses for Profile A and Profile B
Example Calculations

Summary of Analyses for Soldier-Pile Walls Using Ground Anchors (LRFD)

Evaluation of internal shear and moment forces are determined by speadsheet solution for wall elements above the base of the wall

Evaluation of geotechnical resistance (and embedment depth) is determined by LPILE analyses
It is assumed that a 36-inch diameter shaft will be used below the bottom of the excavation.

Based on lateral analyses

Profile	Wall Height (ft)	Section	c-c Spacing (ft)	Anchor Loads (kips) Row 1	Row 2	Mu (k-ft) per Section	Pu (k) per Section	Vu (k) per Section	Embedment (ft)	
A	25	2C12x30	8	98.64	98.42	80.9	373.0	56.9	23	44
A	35†	2C15x50	9	225.60	201.31	282.2	473.0	130.1	25	54
B	25†	2C12x30	8	98.64	98.42	111‡	373.0	56.9	25	44
B	35	2C15x50	9	225.60	201.31	282.2	473.0	130.1	28	54

† *Example Calculations Included*

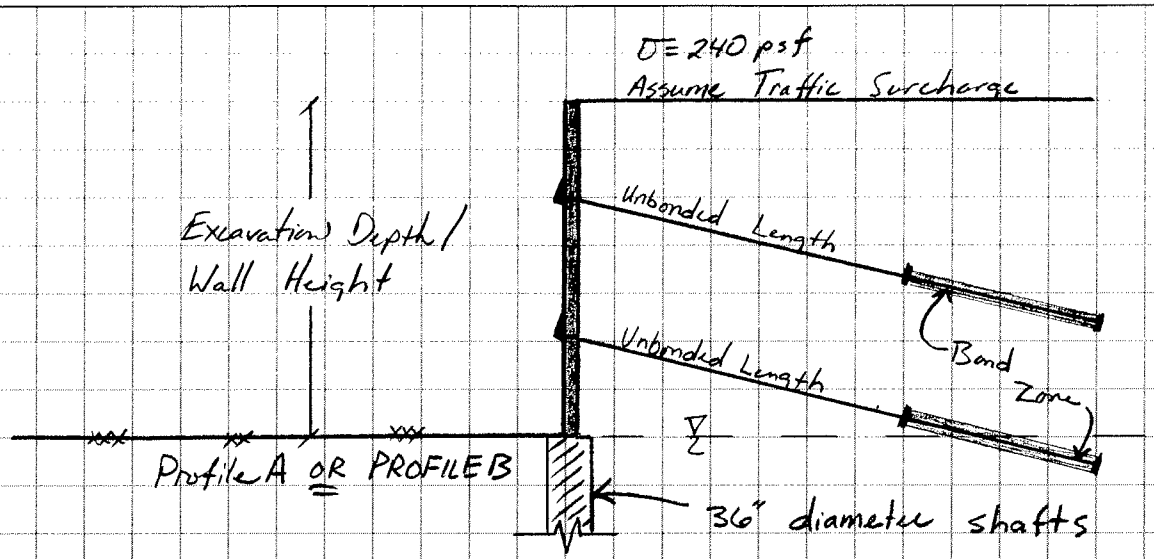
‡ *Maximum Value is Below Base of Wall*

Based on required axial resistance

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT ms Consultants
PROJECT FRA-70-8.93
SUBJECT Anchored Retaining Wall

PROJECT NO. 0221-1004.01
SHEET NO. 1 OF 39
COMP. BY SAR DATE 3-25-09
CHECKED BY CWT DATE 3-27-09



Assumptions:

1) Vertical structural elements use double C-shapes.
2) Assume sandy silt material in bond zone. - Med. dense
3) For efficiency, multiple rows of anchors are considered.
4) Assume groundwater at bottom of excavation
5) Assume 15° inclination for soil anchors.
6) Below base of wall, it is assumed that 36" diameter shaft is used.
   Piles should be preBored. Annular space should be filled with concrete up to the base of the wall.
7) Soil Profiles A and B were used to determine the required embedment depths. Refer to drilled shaft retaining wall calculations for details of the soil profiles.

Because discrete foundation elements (bored piles) are proposed below the bottom of excavation (base of wall), LPILE Plus 5.0 was used to determine the pile response below the bottom of the excavation.

Deflections should be limited to approximately 1-2 inches.

CLIENT	ms consultants, ODOT-District 6	JOB NUMBER	0221-1004.01		
PROJECT	FRA-70-8.93	SHEET NO	2	OF	39
SUBJECT	Anchored Retaining Wall Analysis	COMP BY	SJR	DATE	3/25/09
		CHKD BY	[illegible]	DATE	3-31-09

LRFD - ANCHORED RETAINING WALL CALCULATIONS (SOLDIER PILE AND LAGGING TYPE)

WALL AND BACKFILL PARAMETERS

25.0 ft — H - RETAINED WALL HEIGHT — k_a = 0.3333

$$k_a = \tan^2(45 - \Phi'/2)$$

30 deg — Φ' - RETAINIED SOIL

120 pcf — γ' - RETAINED SOIL — **p_a = 813 psf**

Appearent Earth Pressures from AASHTO 3.11.5.7

240 psf — LS - Live Load Surcharge *(AASHTO Table 3.11.6.4-2)*

*for a single row of anchors

$$p_a = k_a \gamma' H$$

*for 2 rows of anchors

$$p_a = \frac{k_a \gamma' H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}}$$

ANCHORS

2 ROWS OF ANCHORS

5.5 ft — H_1 - DISTANCE FROM GROUND SURFACE TO TOP ROW OF ANCHORS

11.5 ft — H_2 - DISTANCE BETWEEN BOTTOM AND TOP ROW OF ANCHORS

8.0 ft — H_3 - DISTANCE BETWEEN BOTTOM ROW OF ANCHORS AND BOTTOM OF EXCAVATION

15 deg — INCLINATION OF ANCHORS (10-15 degrees is recommended)

Load Factors and Load Combinations

Group	EH	LS
Strength I	1.35	1.75
Service I	1.00	1.00

AASHTO Table 3.4.1-1 and 3.4.1-2

322 kips — Factored Axial Load from Structure

Anchor Load using Triburary Area Method

Unfactored - Service I

T_1 = 8,858 lbs / ft width OR 70,861 lbs per section

T_2 = 8,874 lbs /ft width OR 70,990 lbs per section

Factored - Strength I a/b

T_{n1} = 12,331 lbs /ft width OR 98,644 lbs per section

T_{n2} = 12,302 lbs /ft width OR 98,420 lbs per section

STRUCTURAL ELEMENTS

8.0 ft — S - Center to center spacing of vertical wall elements

Provided Section Properties **2-C12x30** — E = **29** x10^6 psi — f_y = **50** ksi

Z_x = **67.6** in^3 — I_x = **324** in^4 — A_s = **17.62** in^2 — d = **12.0** in

Z_x = 8.5 in^3 per ft — I_x = 40.5 in^4 per ft — t_w = **1.020** in — t_f = **0.501** in

CHECK CAPACITY

Φ_f = **1.00** — Φ_c = **0.70** — Φ_v = **1.00** — *AASHTO 6.5.4.2*

LPILE → 111 k-ft

Maximum Factored Bending Moment, M_u = 10.1 k-ft per foot OR **80.9** k-ft per section

Maximum Factored Axial Force, P_u = 51.0 (anchor load) + 322 (axial load) = **373** k per section

Maximum Factored Shear Force, V_u = 7.1 kips per foot OR **56.9** k per section

Factored Flexural Resistance, $\Phi_f M_n$ =	**281.7** k-ft per section	M_u	is less than $\Phi_f M_n$	**OK**
Factored Axial Resistance, $\Phi_c P_n$ =	**572.0** k per section	P_u	is less than $\Phi_c P_n$	**OK**
Factored Shear Resistance, $\Phi_v V_n$ =	**325.3** k per section	V_u	is less than $\Phi_v V_n$	**OK**

FACTORED SHEAR AND MOMENT AT GROUND SURFACE (for determining geotechnical resistance)

R = V = **17.7** kips per section — M = **51** k-ft per section — 611,573 lb-in per section

Reference: AASHTO LRFD Bridge Design Specifications, 4th Edition 2007 (with 2008 Interim Revisions)

Using Apparent Earth Pressures

AASHTO 3.11.5.7

Maximum pressure ordinate, p_a:

$$k_a = \tan^2(45 - \Phi'/2)$$

$$p_a = k_a \gamma' H \quad \text{for a single row of anchors}$$

$$p_a = \frac{k_a \gamma H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}} \quad \text{for multiple rows of anchors}$$

Live Load Surcharge AASHTO 3.11.6.4

$LS = 2.0\gamma$ for walls in excess of 20 feet in height

Anchor Loads and Reaction from Subgrade AASHTO 11.9.5.1

Calculate anchor loads and resultant using triburary area method

Calculate Capacity of Selected Section

Calculate Resistance to flexure: Calculate as per AASHTO 6.12.2

$$\phi_f M_n = \phi_f \left(\frac{F_y Z}{12} \right)$$

If section is compact, $M_n = M_p$ (6.12.2.2.1-1)

Check Combined Axial Compression and Flexure: Calculate as per AASHTO 6.9.2.2

If $\frac{P_u}{P_r} < 0.2$ then, $\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \le 1.0$ (6.9.2.2-1)

If $\frac{P_u}{P_r} \ge 0.2$ then, $\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \le 1.0$ (6.9.2.2-2)

Resistance to compression: Calculate as per AASHTO 6.9.4

$P_r = \phi_c P_n$ (6.9.2.1-1)

If $\lambda \le 2.25$ $P_n = 0.66^{\lambda} F_y A_s$ (6.9.4.1-1)

If $\lambda > 2.25$ $P_n = \left(\frac{0.88}{\lambda} \right) F_y A_s$ (6.9.4.1-2)

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

K is assumed to be 1.0; for pinned-pinned connection between anchor points

r_s, radius of gyration in plane of buckling (assumed in strong axis)

ℓ , unbraced length (anchor points are considered as bracing)

Calculate Capacity of Selected Section

Resistance to Shear: Calculate as per AASHTO 6.10.9

$\phi_v V_n = \phi_v C \cdot 0.58 F_y \cdot D \cdot t_w$ where D=depth of web (6.10.9.2-2)

If $\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_{yw}}}$, then; C=1.0 where k=5 (6.10.9.3.2-4)

If $1.12\sqrt{\frac{Ek}{F_{yw}}} < \frac{D}{t_w} \le 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then;

$$C = \frac{1.12}{\left(\frac{D}{t_w}\right)}\sqrt{\frac{Ek}{F_{yw}}} \quad (6.10.9.3.2\text{-}5)$$

If $\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then;

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \quad (6.10.9.3.2\text{-}6)$$

Anchor Pullout Capacity AASHTO 11.9.4.2

Calculate nominal anchor pullout capacity

$$\phi Q_n = \phi \pi d \tau_n L_b \quad (11.9.4.2\text{-}1)$$

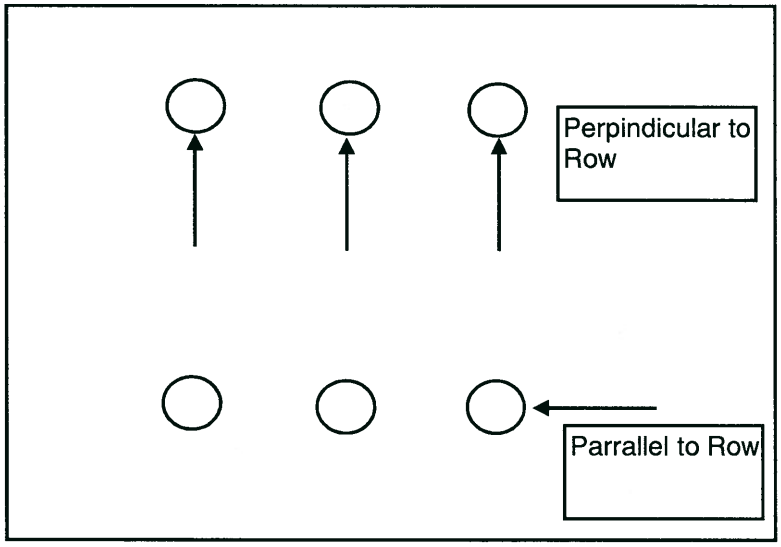
SJK 3/25/09
QA/QC 3-27-09

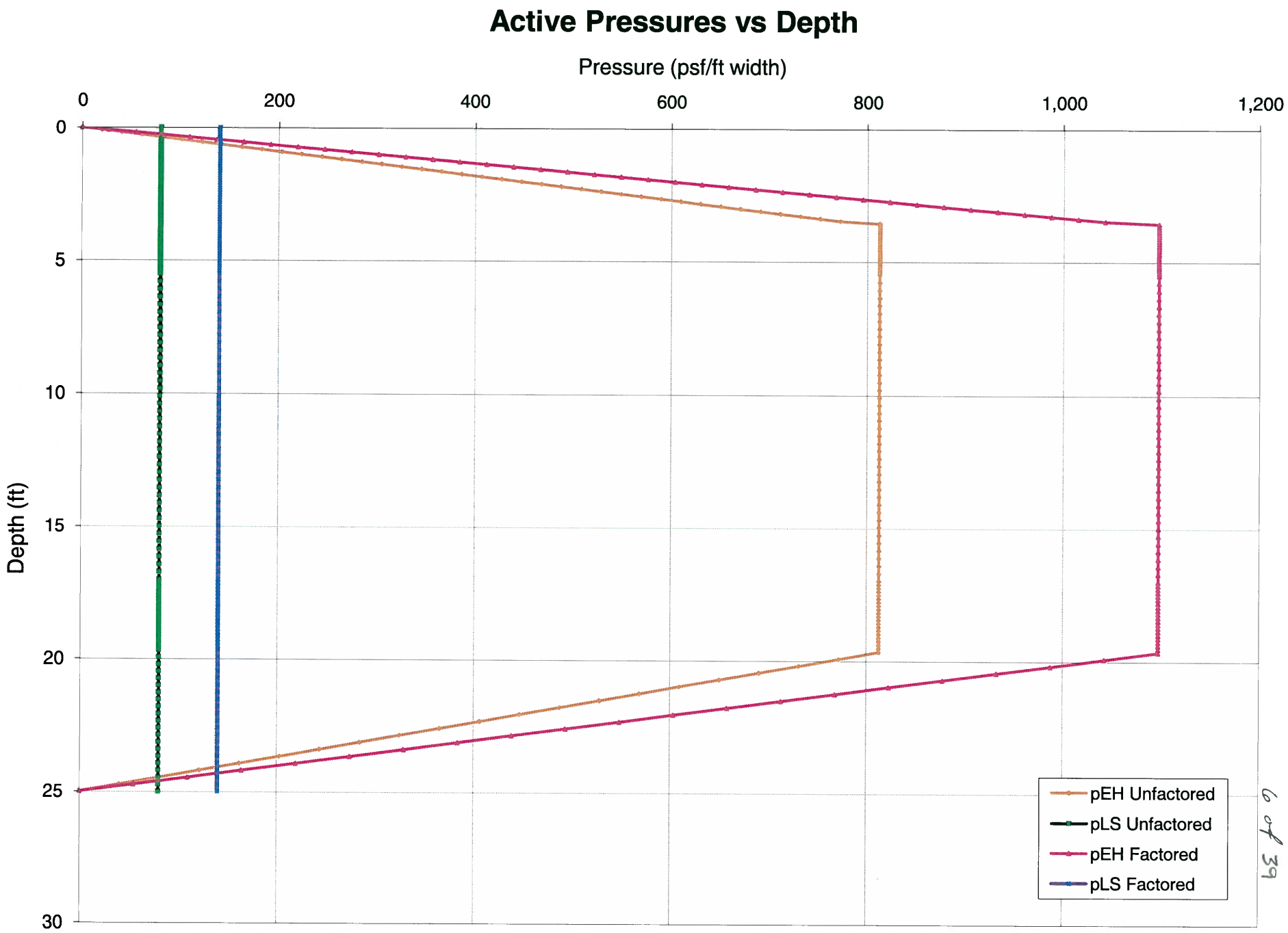
p-y reduction factor
Reference: Reese, GROUP 6.0 Technical Manual

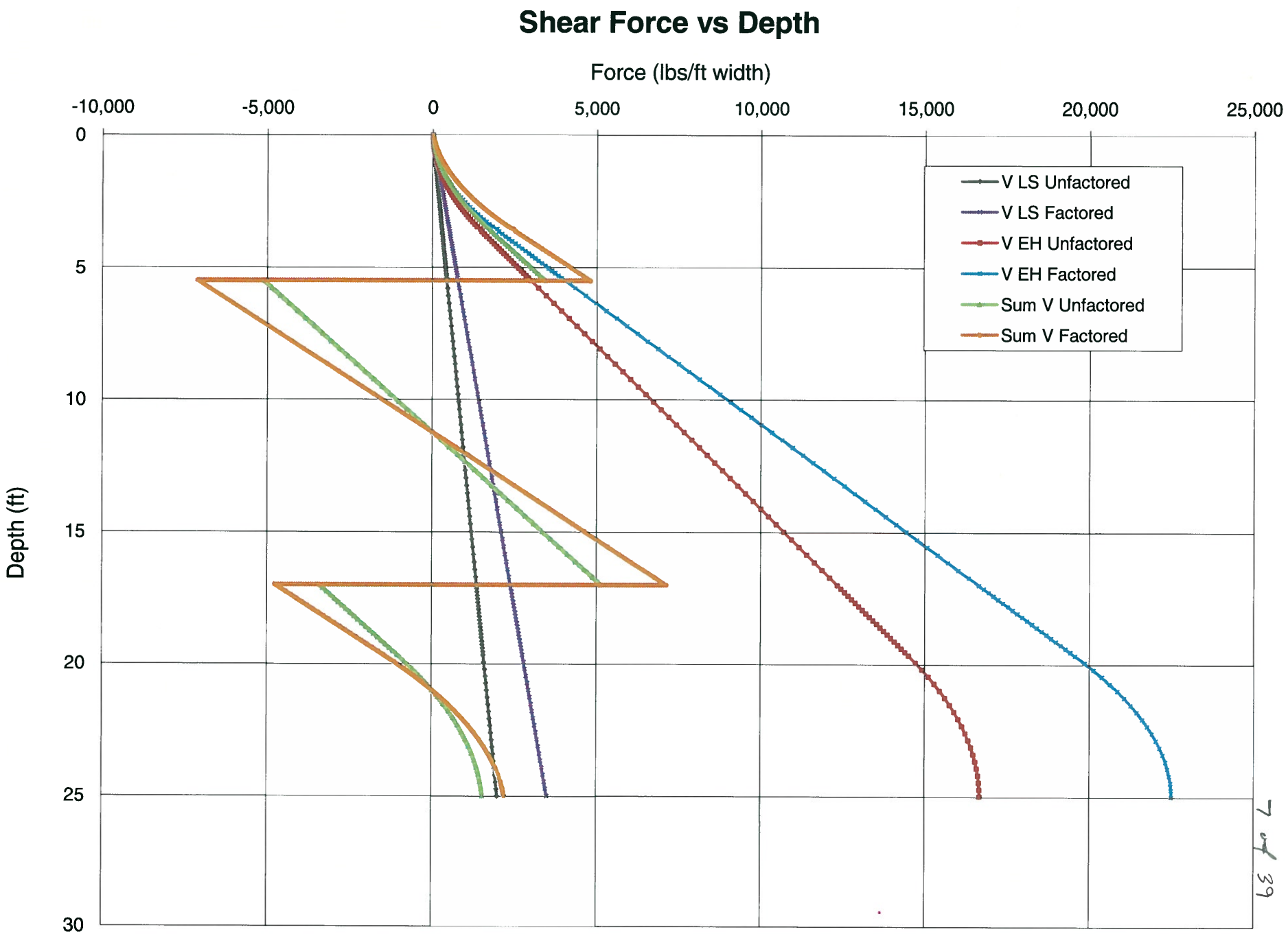
Pile Diameter =	3.0	feet	S/B =	2.666667
B	36	inches		
Pile Spacing =	8.0	feet		
S	96	inches		
Loading Perpr to Row [1]	Y	Y/N	β_a =	**0.922**

$$\beta_a \approx 0.5292\left(\frac{s}{b}\right)^{0.5659} \quad \text{for} \cdot 1.0 \le \frac{s}{b} \le 3.28$$

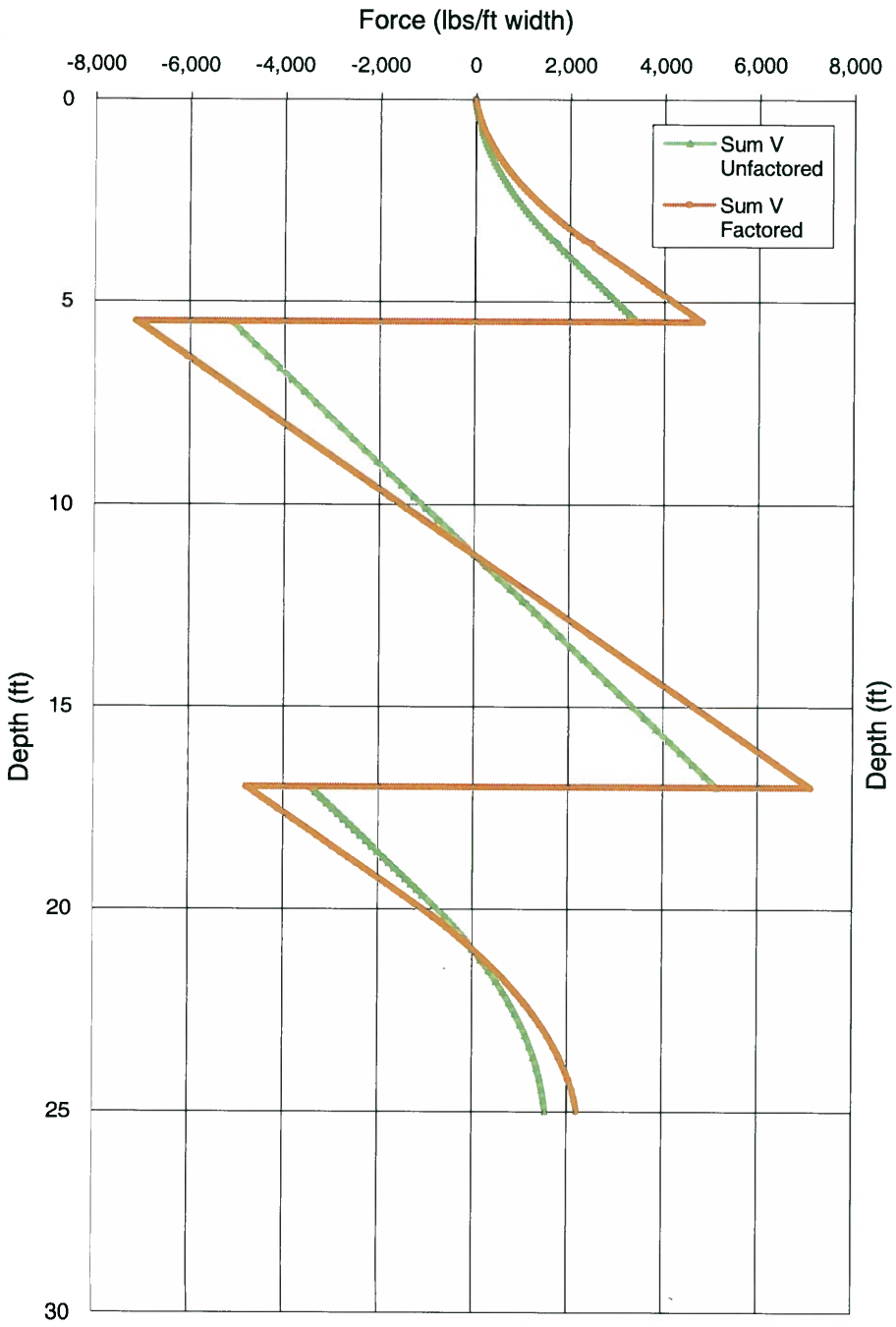
$$\beta_a \approx 1.0 \rightarrow \text{for} \cdot \frac{s}{b} \ge 3.28$$



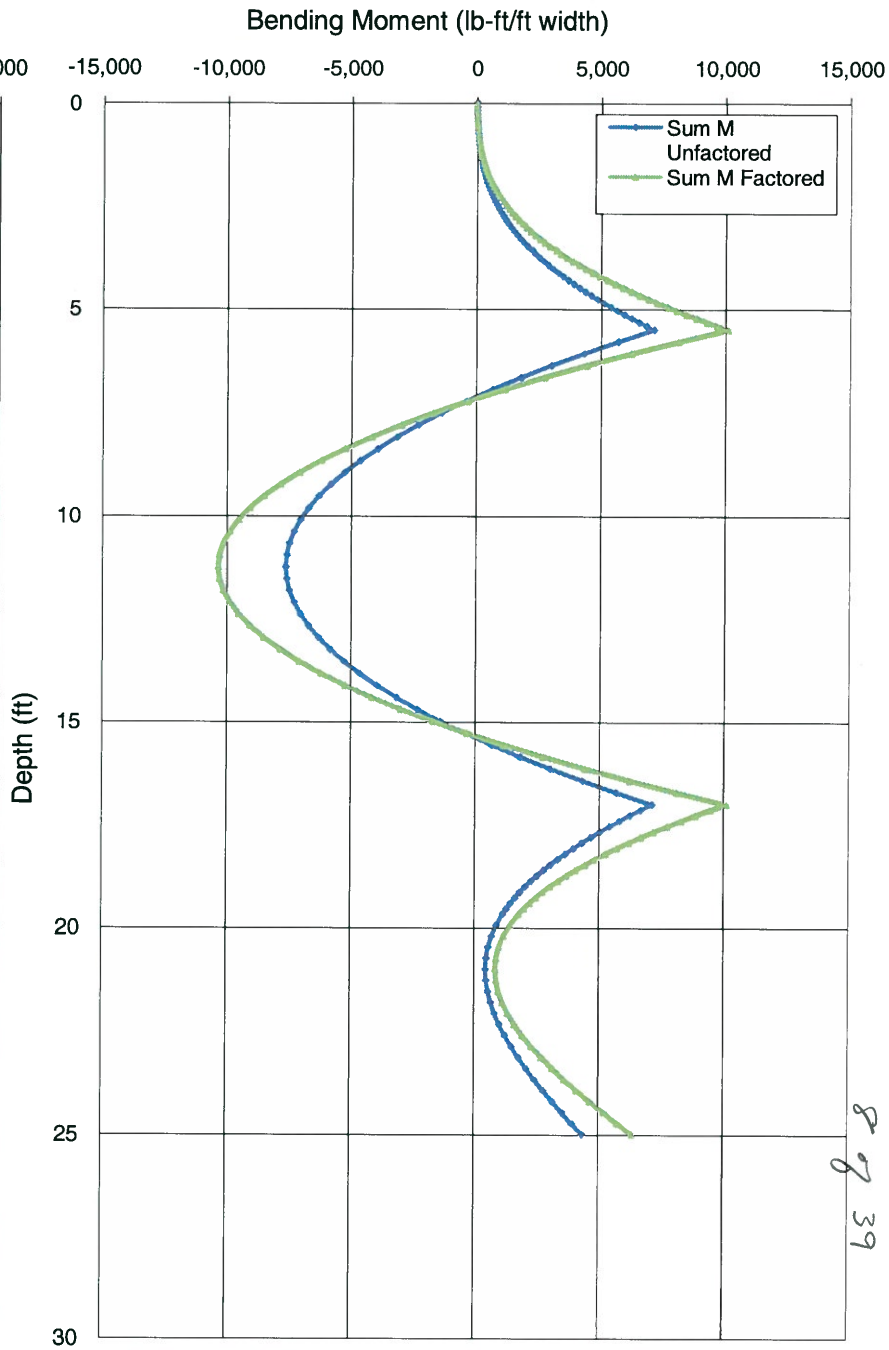

Active Pressures vs Depth
Pressure (psf/ft width)
0
200
400
600
800
1,000
1,200
Depth (ft)
0
5
10
15
20
25
30
pEH Unfactored
pLS Unfactored
pEH Factored
pLS Factored


Shear Force vs Depth
Force (lbs/ft width)
-10,000
-5,000
0
5,000
10,000
15,000
20,000
25,000
Depth (ft)
0
5
10
15
20
25
30
V LS Unfactored
V LS Factored
V EH Unfactored
V EH Factored
Sum V Unfactored
Sum V Factored

Shear Force vs Depth



Bending Moment vs Depth



8 of 39

9 of 39
SJR 3/25/09
GWR 3-27-09

STRENGTH-I Determine Embedment Depth L' = 25'

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo

LPILE Plus for Windows, Version 5.0 (5.0.5)

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

This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations: M:\proj\0221\1004\01\geotechnical\Retaining Walls\LPILE\Final Anchored Wall\
Name of input data file: TB Profile B, 25 ft 36 in on 96 in centers

STRENGTH.lpd
Name of output file: TB Profile B, 25 ft 36 in on 96 in centers

STRENGTH.lpo
Name of plot output file: TB Profile B, 25 ft 36 in on 96 in centers

STRENGTH.lpp
Name of runtime file: TB Profile B, 25 ft 36 in on 96 in centers

STRENGTH.lpr

Time and Date of Analysis

Date: March 25, 2009 Time: 15:25:19

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:
- User-specified p-y curves used in analysis

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- 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 = 300.00 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	36.00000000	324.0000	17.6200	29000000.
2	1000.0000	36.00000000	324.0000	17.6200	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 9 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 = 36.000 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

Layer 2 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 36.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 3 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 4 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 5 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 324.000 in

Layer 6 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 324.000 in
Distance from top of pile to bottom of layer = 384.000 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

Layer 7 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 384.000 in
Distance from top of pile to bottom of layer = 444.000 in

Layer 8 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 444.000 in
Distance from top of pile to bottom of layer = 624.000 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

Layer 9 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = 624.000 in
Distance from top of pile to bottom of layer = 1000.000 in

(Depth of lowest layer extends 700.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 18 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.03600
2	36.00	.03600
3	36.00	.03600
4	120.00	.03600
5	120.00	.03600
6	180.00	.03600
7	180.00	.03600
8	240.00	.03600
9	240.00	.03600
10	324.00	.03600
11	324.00	.03900
12	384.00	.03900
13	384.00	.03600
14	444.00	.03600
15	444.00	.03600
16	624.00	.03600
17	624.00	.03600
18	1000.00	.03600

Shear Strength of Soils

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	36.00	------	------
2	36.000	.00000	36.00	------	------
3	324.000	.00000	38.00	------	------
4	384.000	.00000	38.00	------	------
5	444.000	.00000	36.00	------	------
6	624.000	.00000	36.00	------	------

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.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.9220	1.0000
2	1000.000	.9220	1.0000

User-specified p-y Curves

User-specified p-y curves defined using 12 curves.

User-specified curve number 1 at depth = 36.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0280	400.000
3	.1120	1550.000
4	.2800	3560.000
5	.4670	5580.000
6	.6910	7600.000
7	.9900	9610.000
8	1.3400	11630.000
9	1.7700	13570.000
10	2.3000	15590.000
11	2.8900	17530.000
12	3.6400	19480.000

13	4.4800	21420.000
14	5.5100	23290.000
15	7.6400	26460.000
16	100.0000	28190.000

User-specified curve number 2 at depth = 120.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0280	400.000
3	.1120	1550.000
4	.2800	3560.000
5	.4670	5580.000
6	.6910	7600.000
7	.9900	9610.000
8	1.3400	11630.000
9	1.7700	13570.000
10	2.3000	15590.000
11	2.8900	17530.000
12	3.6400	19480.000
13	4.4800	21420.000
14	5.5100	23290.000
15	7.6400	26460.000
16	100.0000	28190.000

User-specified curve number 3 at depth = 120.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0190	500.000
3	.0750	1660.000
4	.1870	3670.000
5	.3170	5690.000
6	.4670	7780.000
7	.6160	9790.000
8	.8000	11810.000
9	1.0300	13820.000
10	1.2500	15840.000
11	1.5500	17860.000
12	1.8300	19940.000
13	2.2000	21890.000
14	2.6000	23900.000
15	3.0200	25920.000
16	5.2300	33840.000
17	100.0000	42340.000

User-specified curve number 4 at depth = 180.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0190	500.000
3	.0750	1660.000
4	.1870	3670.000
5	.3170	5690.000
6	.4670	7780.000
7	.6160	9790.000

8	.8000	11810.000
9	1.0300	13820.000
10	1.2500	15840.000
11	1.5500	17860.000
12	1.8300	19940.000
13	2.2000	21890.000
14	2.6000	23900.000
15	3.0200	25920.000
16	5.2300	33840.000
17	100.0000	42340.000

User-specified curve number 5 at depth = 180.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0280	540.000
3	.0930	1690.000
4	.2050	3710.000
5	.3170	5720.000
6	.4290	7810.000
7	.5790	9830.000
8	.7100	11920.000
9	.8800	13930.000
10	1.0600	15950.000
11	1.2300	17960.000
12	1.4400	20050.000
13	1.6800	22070.000
14	1.9400	24080.000
15	2.2200	26100.000
16	3.7000	34160.000
17	100.0000	46620.000

User-specified curve number 6 at depth = 240.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0280	540.000
3	.0930	1690.000
4	.2050	3710.000
5	.3170	5720.000
6	.4290	7810.000
7	.5790	9830.000
8	.7100	11920.000
9	.8800	13930.000
10	1.0600	15950.000
11	1.2300	17960.000
12	1.4400	20050.000
13	1.6800	22070.000
14	1.9400	24080.000
15	2.2200	26100.000
16	3.7000	34160.000
17	100.0000	46620.000

User-specified curve number 7 at depth = 240.000in

Point No.	y in	p, lbs/in

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo

1	.0000	.000
2	.0370	580.000
3	.1030	1730.000
4	.2240	3740.000
5	.3550	5760.000
6	.5040	7850.000
7	.6910	9860.000
8	.9000	11880.000
9	1.1400	13900.000
10	1.4000	15910.000
11	1.7000	17930.000
12	2.0500	19940.000
13	2.4600	21960.000
14	2.9300	23900.000
15	3.4700	25920.000
16	4.7600	29880.000
17	100.0000	38740.000

User-specified curve number 8 at depth = 324.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0370	580.000
3	.1030	1730.000
4	.2240	3740.000
5	.3550	5760.000
6	.5040	7850.000
7	.6910	9860.000
8	.9000	11880.000
9	1.1400	13900.000
10	1.4000	15910.000
11	1.7000	17930.000
12	2.0500	19940.000
13	2.4600	21960.000
14	2.9300	23900.000
15	3.4700	25920.000
16	4.7600	29880.000
17	100.0000	38740.000

User-specified curve number 9 at depth = 384.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0370	680.000
3	.0560	890.000
4	.1120	1900.000
5	.2430	3920.000
6	.4110	5930.000
7	.5790	7950.000
8	.7800	10040.000
9	1.0500	11980.000
10	1.3800	14000.000
11	1.7700	16010.000
12	2.2200	17960.000
13	2.7400	19970.000
14	3.4000	21920.000
15	4.1600	23860.000
16	6.2400	27600.000

17	100.0000	31280.000

User-specified curve number 10 at depth = 444.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.0370	680.000
3	.0560	890.000
4	.1120	1900.000
5	.2430	3920.000
6	.4110	5930.000
7	.5790	7950.000
8	.7800	10040.000
9	1.0500	11980.000
10	1.3800	14000.000
11	1.7700	16010.000
12	2.2200	17960.000
13	2.7400	19970.000
14	3.4000	21920.000
15	4.1600	23860.000
16	6.2400	27600.000
17	100.0000	31280.000

User-specified curve number 11 at depth = 624.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.4670	6540.000
3	.9330	11570.000
4	1.4000	14790.000
5	1.8670	17310.000
6	2.3340	19630.000
7	2.8000	21440.000
8	3.2670	22900.000
9	3.7340	24150.000
10	4.2010	25360.000
11	100.0000	26420.000

User-specified curve number 12 at depth = 1000.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.4670	6540.000
3	.9330	11570.000
4	1.4000	14790.000
5	1.8670	17310.000
6	2.3340	19630.000
7	2.8000	21440.000
8	3.2670	22900.000
9	3.7340	24150.000
10	4.2010	25360.000
11	100.0000	26420.000

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo
Loading Type

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

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Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 17700.000 lbs
Bending moment at pile head = 611573.000 in-lbs
Axial load at pile head = 373000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 17700.000 lbs
Specified moment at pile head = 611573.000 in-lbs
Specified axial load at pile head = 373000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.474188	611573.	17700.0000	-.0104303	55145.4038	0.0000
3.000	.443190	676235.	17659.9332	-.0102247	58737.7513	-26.7112
6.000	.412840	740416.	17537.9080	-.0099986	62303.3207	-54.6390
9.000	.383199	803839.	17331.0339	-.0097520	65826.8742	-83.2770
12.000	.354328	866227.	17037.9396	-.0094854	69292.8339	-112.1192
15.000	.326287	927295.	16658.7693	-.0091991	72685.5404	-140.6610
18.000	.299133	986767.	16195.1753	-.0088935	75989.5106	-168.4017
21.000	.272925	1044370.	15648.9386	-.0085693	79189.6937	-195.7561
24.000	.247718	1099839.	15022.6304	-.0082270	82271.2693	-221.7827
27.000	.223564	1152918.	14320.8241	-.0078673	85220.1229	-246.0882
30.000	.200514	1203371.	13549.2481	-.0074912	88023.0483	-268.2957
33.000	.178617	1250979.	12714.7343	-.0070993	90667.9403	-288.0468
36.000	.157918	1295547.	9379.2274	-.0066928	93143.9783	-1935.6244
39.000	.138460	1322233.	3894.3173	-.0062749	94626.4890	-1720.9823
42.000	.120268	1332956.	-967.6230	-.0058510	95222.2637	-1520.3112
45.000	.103354	1329521.	-5228.0313	-.0054260	95031.4251	-1319.9610
48.000	.087712	1313732.	-8891.7652	-.0050040	94154.2166	-1122.5283
51.000	.073330	1287370.	-11987.0266	-.0045888	92689.6678	-940.9793
54.000	.060180	1252079.	-14560.9888	-.0041834	90729.0765	-774.9955

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo

57.000	.048230	1209366.	-16659.7073	-.0037904	88356.1350	-624.1501
60.000	.037438	1160604.	-18327.8227	-.0034121	85647.1140	-487.9268
63.000	.027757	1107035.	-19608.1162	-.0030500	82671.0929	-365.6023
66.000	.019137	1049781.	-20534.6176	-.0027057	79490.2973	-252.0653
69.000	.011523	989883.	-21140.3748	-.0023801	76162.6320	-151.7728
72.000	.004857	928266.	-21463.9874	-.0020739	72739.4323	-63.9689
75.000	-.000920	865741.	-21541.7552	-.0017875	69265.8230	12.1237
78.000	-.005868	803015.	-21407.6290	-.0015211	65781.0917	77.2938
81.000	-.010047	740699.	-21093.1889	-.0012746	62319.0683	132.3329
84.000	-.013516	679309.	-20627.6491	-.0010479	58908.5093	178.0270
87.000	-.016335	619278.	-20037.8823	-.0008406	55573.4803	215.1508
90.000	-.018560	560963.	-19348.4636	-.0006522	52333.7348	244.4616
93.000	-.020248	504647.	-18581.7286	-.0004821	49205.0856	266.6951
96.000	-.021453	450552.	-17757.8432	-.0003296	46199.7672	282.5618
99.000	-.022226	398838.	-16894.8841	-.0001940	43326.7868	292.7442
102.000	-.022617	349616.	-16008.9256	-7.4533E-05	40592.2621	297.8948
105.000	-.022673	302951.	-15114.1317	2.9645E-05	37999.7452	298.6345
108.000	-.022439	258865.	-14222.8520	.0001193	35550.5324	295.5520
111.000	-.021957	217347.	-13345.7185	.0001954	33243.9573	289.2036
114.000	-.021267	178354.	-12491.7434	.0002585	31077.6700	280.1132
117.000	-.020406	141818.	-11668.4149	.0003096	29047.8992	268.7725
120.000	-.019409	107650.	-10881.7922	.0003495	27149.6996	255.6426
123.000	-.018309	75745.0562	-9831.9800	.0003787	25377.1847	444.2322
126.000	-.017136	47810.8142	-8541.9568	.0003985	23825.2823	415.7833
129.000	-.015918	23601.5404	-7338.9467	.0004099	22480.3227	386.2234
132.000	-.014677	2859.8446	-6225.4394	.0004141	21328.0062	356.1149
135.000	-.013434	-14677.8392	-5202.3572	.0004122	21984.5615	325.9399
138.000	-.012204	-29276.8195	-4269.2881	.0004052	22795.6160	296.1061
141.000	-.011002	-41200.3850	-3424.7000	.0003939	23458.0363	266.9527
144.000	-.009840	-50706.6564	-2666.1358	.0003793	23986.1625	238.7568
147.000	-.008727	-58046.0003	-1990.3917	.0003619	24393.9038	211.7393
150.000	-.007669	-63458.9517	-1393.6763	.0003425	24694.6233	186.0709
153.000	-.006672	-67174.5924	-871.7540	.0003217	24901.0478	161.8773
156.000	-.005739	-69409.3373	-420.0708	.0002998	25025.2003	139.2449
159.000	-.004873	-70366.0798	-33.8652	.0002775	25078.3526	118.2256
162.000	-.004074	-70233.6520	291.7356	.0002551	25070.9955	98.8416
165.000	-.003342	-69186.5568	561.6328	.0002328	25012.8236	81.0899
168.000	-.002677	-67384.9337	780.6868	.0002110	24912.7334	64.9461
171.000	-.002076	-64974.7201	953.6586	.0001899	24778.8327	50.3684
174.000	-.001537	-62087.9765	1085.1625	.0001696	24618.4580	37.3008
177.000	-.001058	-58843.3431	1179.6278	.0001503	24438.2006	25.6761
180.000	-.000635	-55346.6012	1241.2704	.0001321	24243.9372	15.4190
183.000	-.000266	-51691.3144	1271.4872	.0001150	24040.8657	4.7255
186.000	5.45E-05	-47975.0294	1277.1229	9.9081E-05	23834.4054	-.9683926
189.000	.000329	-44250.3195	1266.9025	8.4358E-05	23627.4771	-5.8452
192.000	.000561	-40562.4068	1243.1821	7.0818E-05	23422.5930	-9.9684
195.000	.000754	-36949.7174	1208.1285	5.8444E-05	23221.8881	-13.4007
198.000	.000911	-33444.4327	1163.7220	4.7206E-05	23027.1500	-16.2036
201.000	.001037	-30073.0319	1111.7610	3.7066E-05	22839.8500	-18.4370
204.000	.001134	-26856.8195	1053.8683	2.7977E-05	22661.1715	-20.1581
207.000	.001205	-23812.4349	991.4984	1.9888E-05	22492.0390	-21.4218
210.000	.001253	-20952.3392	925.9456	1.2742E-05	22333.1448	-22.2800
213.000	.001281	-18285.2776	858.3538	6.4779E-06	22184.9747	-22.7813
216.000	.001292	-15816.7143	789.7252	1.0338E-06	22047.8323	-22.9711
219.000	.001287	-13549.2400	720.9312	-3.6542E-06	21921.8616	-22.8916
222.000	.001270	-11482.9490	652.7220	-7.6504E-06	21807.0676	-22.5812
225.000	.001241	-9615.7864	585.7371	-1.1019E-05	21703.3363	-22.0753
228.000	.001204	-7943.8665	520.5156	-1.3822E-05	21610.4519	-21.4057
231.000	.001159	-6461.7594	457.5060	-1.6122E-05	21528.1126	-20.6007
234.000	.001107	-5162.7501	397.0764	-1.7977E-05	21455.9454	-19.6857
237.000	.001051	-4039.0672	339.5239	-1.9446E-05	21393.5186	-18.6827
240.000	.000990	-3082.0857	285.0834	-2.0583E-05	21340.3530	-17.6110

243.000	.000927	-2282.5017	238.5660	-2.1440E-05	21295.9316	-13.4006
246.000	.000862	-1602.7076	199.7823	-2.2060E-05	21258.1653	-12.4552
249.000	.000795	-1034.4378	163.8680	-2.2481E-05	21226.5948	-11.4876
252.000	.000727	-569.1870	130.8781	-2.2737E-05	21200.7475	-10.5057
255.000	.000658	-198.2841	100.8456	-2.2859E-05	21180.1418	-9.5159
258.000	.000590	87.0463	73.7867	-2.2877E-05	21173.9619	-8.5234
261.000	.000521	295.6352	49.7035	-2.2816E-05	21185.5502	-7.5321
264.000	.000453	436.3300	28.5882	-2.2699E-05	21193.3665	-6.5448
267.000	.000385	517.9656	10.4256	-2.2547E-05	21197.9019	-5.5636
270.000	.000318	549.3435	-4.8042	-2.2377E-05	21199.6451	-4.5896
273.000	.000251	539.2189	-17.1234	-2.2203E-05	21199.0826	-3.6232
276.000	.000184	496.2931	-26.5544	-2.2037E-05	21196.6978	-2.6642
279.000	.000118	429.2121	-33.1190	-2.1890E-05	21192.9711	-1.7121
282.000	5.30E-05	346.5686	-36.8361	-2.1766E-05	21188.3798	-.7659836
285.000	-1.21E-05	256.9074	-37.7221	-2.1670E-05	21183.3986	.1753624
288.000	-7.70E-05	168.7326	-35.7893	-2.1602E-05	21178.5000	1.1132
291.000	-.000142	90.5160	-31.0467	-2.1560E-05	21174.1547	2.0486
294.000	-.000206	30.7045	-23.4995	-2.1541E-05	21170.8318	2.9828
297.000	-.000271	-2.2728	-13.1505	-2.1536E-05	21169.2523	3.9166
300.000	-.000336	0.0000	0.0000	-2.1537E-05	21169.1260	4.8504

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.47418840 in
Computed slope at pile head	=	-.01043030
Maximum bending moment	=	1332956. lbs-in → 1111 k-ft
Maximum shear force	=	-21541.75524 lbs
Depth of maximum bending moment	=	42.00000000 in
Depth of maximum shear force	=	75.00000000 in
Number of iterations	=	8
Number of zero deflection points	=	3

Summary of Pile-Head 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 = rotational stiffness of

pile-headin-lbs/rad

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 17700.	M= 6.12E+05	373000.	.4741884	1332956.	-21541.7552

TB Profile B, 25 ft 36 in on 96 in centers STRENGTH.lpo

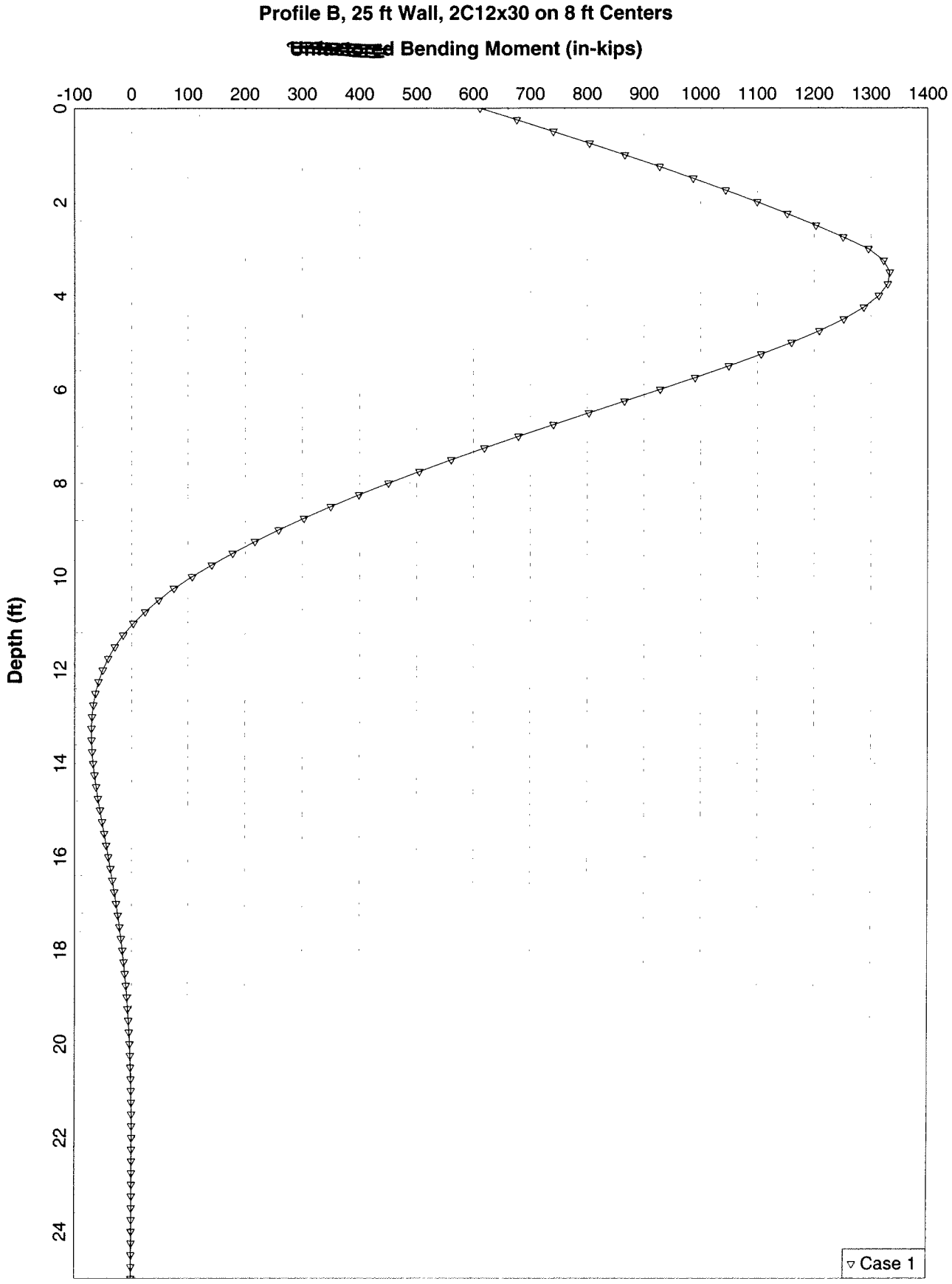
Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

Shear = 17700. lbs
Moment = 611573. in-lbs
Axial Load = 373000. lbs

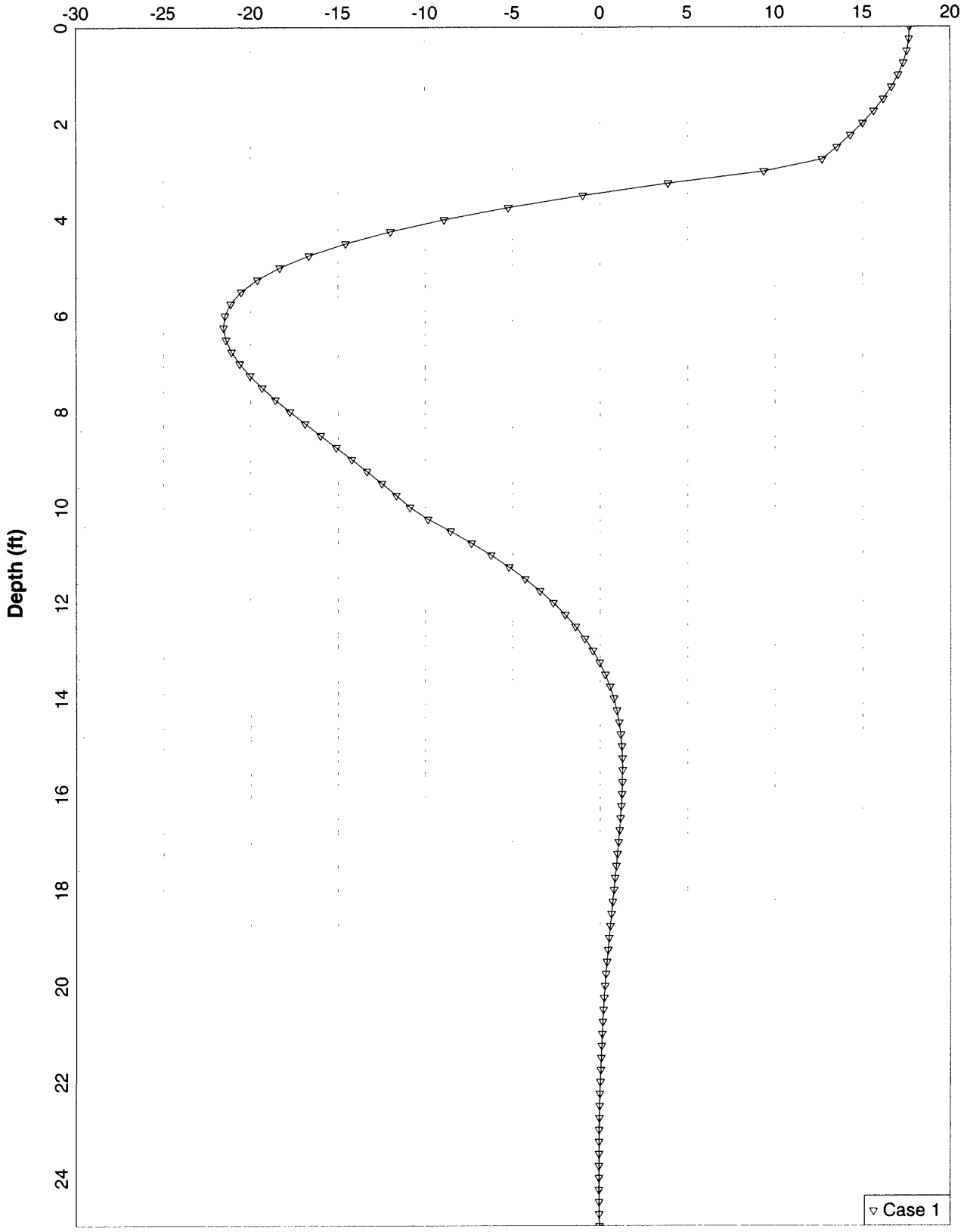
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
300.000	.47418840	1332956.	-21541.75524
285.000	.48888581	1347759.	-21776.95153
270.000	.49959364	1358091.	-21987.71166
255.000	.50665414	1364317.	-22096.90449
240.000	.50964779	1367731.	-22147.86542
225.000	.50857599	1367008.	-22145.38634
210.000	.50382197	1362008.	-22051.05003
195.000	.49503467	1352589.	-21881.59509
180.000	.50632999	1363805.	-21974.23864
165.000	.48743332	1344998.	-21539.30424

The analysis ended normally.


Profile B, 25 ft Wall, 2C12x30 on 8 ft Centers
[redacted] Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
Case 1

Profile B, 25 ft Wall, 2C12x30 on 8 ft Centers

Shear Force (kips)



CLIENT	ms consultants, ODOT-District 6	JOB NUMBER	0221-1004.01		
PROJECT	FRA-70-8.93	SHEET NO	23	OF	39
SUBJECT	Anchored Retaining Wall Analysis	COMP BY	SJR	DATE	3/25/09
		CHKD BY	[illegible]	DATE	3-31-09

LRFD - ANCHORED RETAINING WALL CALCULATIONS (SOLDIER PILE AND LAGGING TYPE)

WALL AND BACKFILL PARAMETERS

35.0 ft — H - RETAINED WALL HEIGHT — k_a = 0.3333

$$k_a = \tan^2(45 - \Phi'/2)$$

30 deg — Φ' - RETAINIED SOIL

120 pcf — γ' - RETAINED SOIL — **p_a = 1,120 psf**

*for a single row of anchors

Appearent Earth Pressures from AASHTO 3.11.5.7

$$p_a = k_a \gamma' H$$

240 psf — LS - Live Load Surcharge *(AASHTO Table 3.11.6.4-2)*

*for 2 rows of anchors

$$p_a = \frac{k_a \gamma' H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}}$$

ANCHORS

2 — ROWS OF ANCHORS

8.5 ft — H_1 - DISTANCE FROM GROUND SURFACE TO TOP ROW OF ANCHORS

17.5 ft — H_2 - DISTANCE BETWEEN BOTTOM AND TOP ROW OF ANCHORS

9.0 ft — H_3 - DISTANCE BETWEEN BOTTOM ROW OF ANCHORS AND BOTTOM OF EXCAVATION

15 deg — INCLINATION OF ANCHORS (10-15 degrees is recommended)

Load Factors and Load Combinations

Group	EH	LS
Strength I	1.35	1.75
Service I	1.00	1.00

AASHTO Table 3.4.1-1 and 3.4.1-2

Anchor Load using Triburary Area Method

Unfactored - Service I

T_1 = 18,145 lbs / ft width OR 163,304 lbs per section

T_2 = 16,243 lbs /ft width OR 146,191 lbs per section

Factored - Strength I a/b

T_{n1} = 25,067 lbs /ft width OR 225,604 lbs per section

362 kips — Factored Axial Load from Structure — T_{n2} = 22,368 lbs /ft width OR 201,309 lbs per section

STRUCTURAL ELEMENTS

9.0 ft — S - Center to center spacing of vertical wall elements

Provided Section Properties **2-C15x50**

E = **29** x10^6 psi — f_y = **50** ksi

Z_x = **137** in^3 — I_x = **808** in^4 — A_s = **29.4** in^2 — d = **15.0** in

Z_x = 15.2 in^3 per ft — I_x = 89.8 in^4 per ft — t_w = **1.432** in — t_f = **0.650** in

CHECK CAPACITY

Φ_f = **1.00** — Φ_c = **0.70** — Φ_v = **1.00** — *AASHTO 6.5.4.2*

Maximum Factored Bending Moment, M_u = 31.4 k-ft per foot OR **282.2** k-ft per section

Maximum Factored Axial Force, P_u = 110.5 (anchor load) + 362.3 (axial load) = **473** k per section

Maximum Factored Shear Force, V_u = 14.5 kips per foot OR **130.1** k per section

Factored Flexural Resistance, $\Phi_f M_n$ =	**570.8**	k-ft per section	M_u	is less than $\Phi_f M_n$	**OK**
Factored Axial Resistance, $\Phi_c P_n$ =	**915.9**	k per section	P_u	is less than $\Phi_c P_n$	**OK**
Factored Shear Resistance, $\Phi_v V_n$ =	**568.9**	k per section	V_u	is less than $\Phi_v V_n$	**OK**

FACTORED SHEAR AND MOMENT AT GROUND SURFACE (for determining geotechnical resistance)

R = V = **28.6** kips per section — M = **224** k-ft per section — 2,682,288 lb-in per section

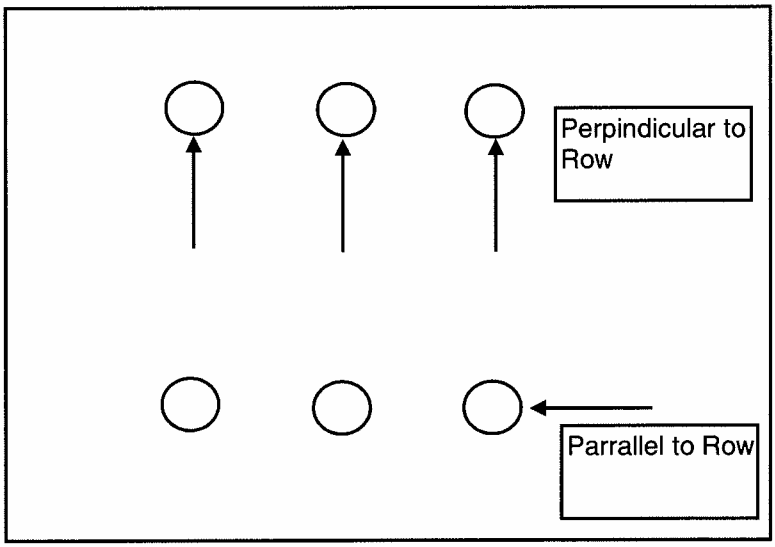
Reference: AASHTO LRFD Bridge Design Specifications, 4th Edition 2007 (with 2008 Interim Revisions)

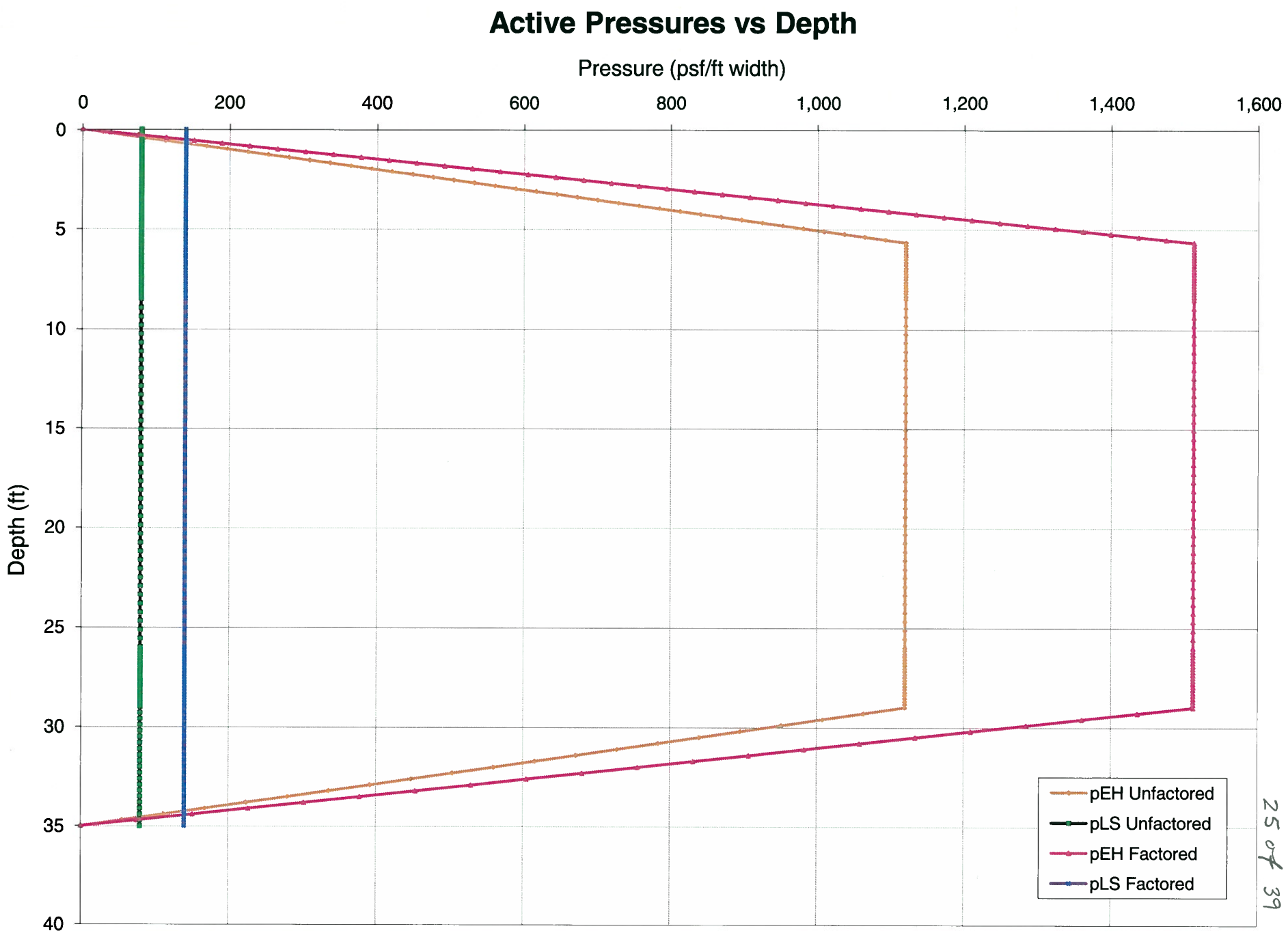
p-y reduction factor
Reference: Reese, GROUP 6.0 Technical Manual

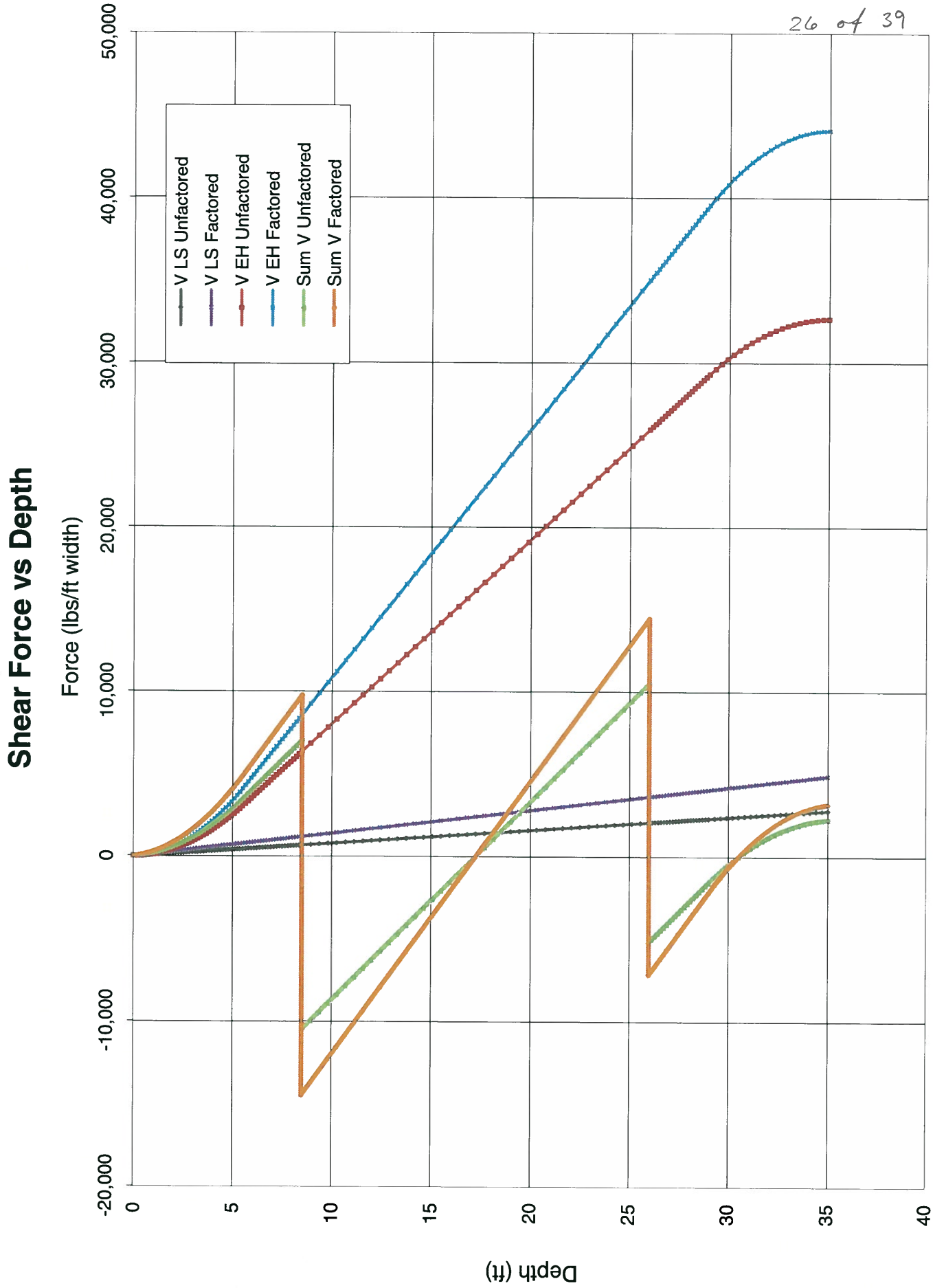
Pile Diameter =	3.0	feet	S/B =	3
B	36	inches		
Pile Spacing =	9.0	feet		
S	108	inches		
Loading Perpr to Row [1]	Y	Y/N	β_a =	**0.985**

$$\beta_a \approx 0.5292\left(\frac{s}{b}\right)^{0.5659} \quad \text{for} \cdot 1.0 \le \frac{s}{b} \le 3.28$$

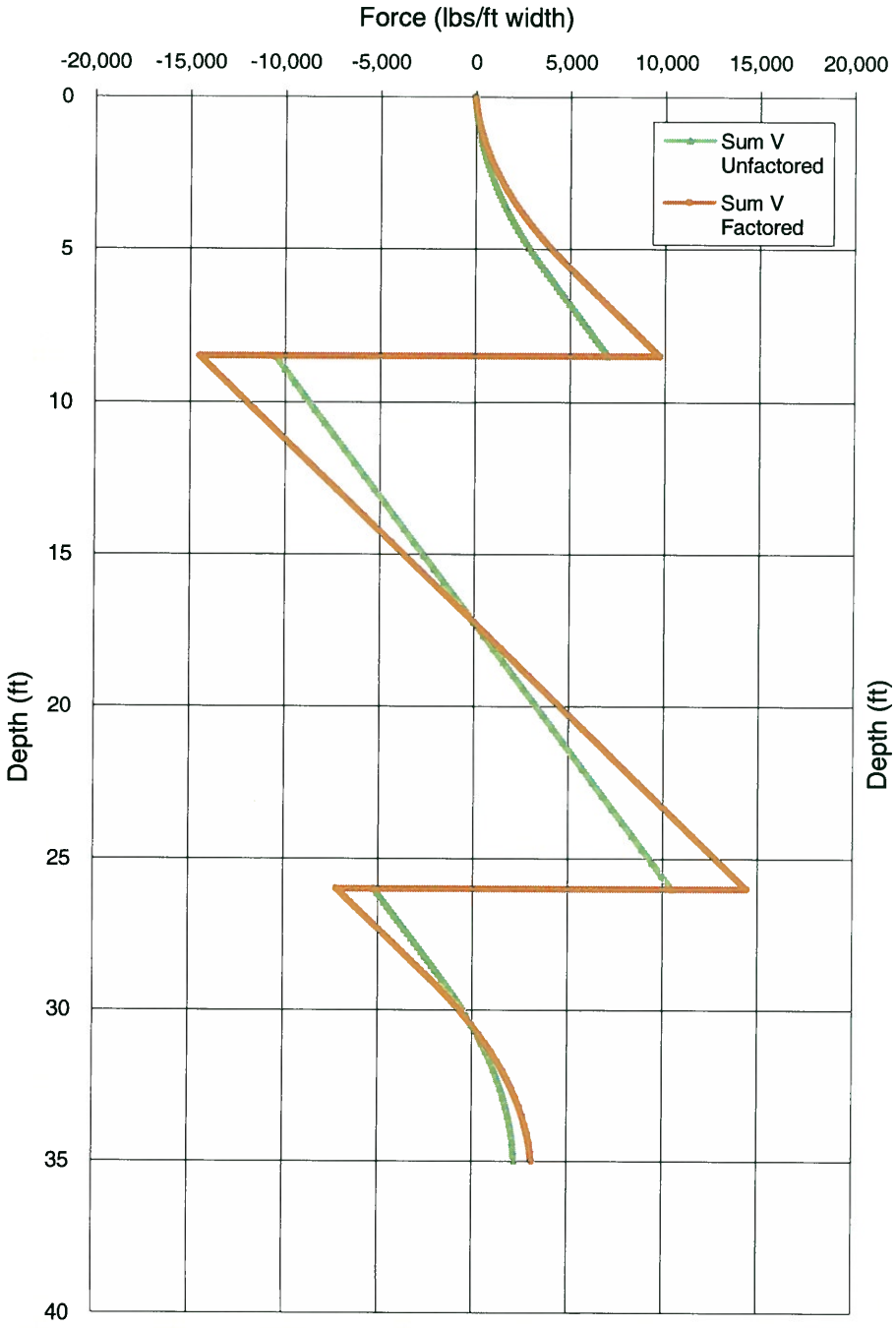
$$\beta_a \approx 1.0 \rightarrow \text{for} \cdot \frac{s}{b} \ge 3.28$$



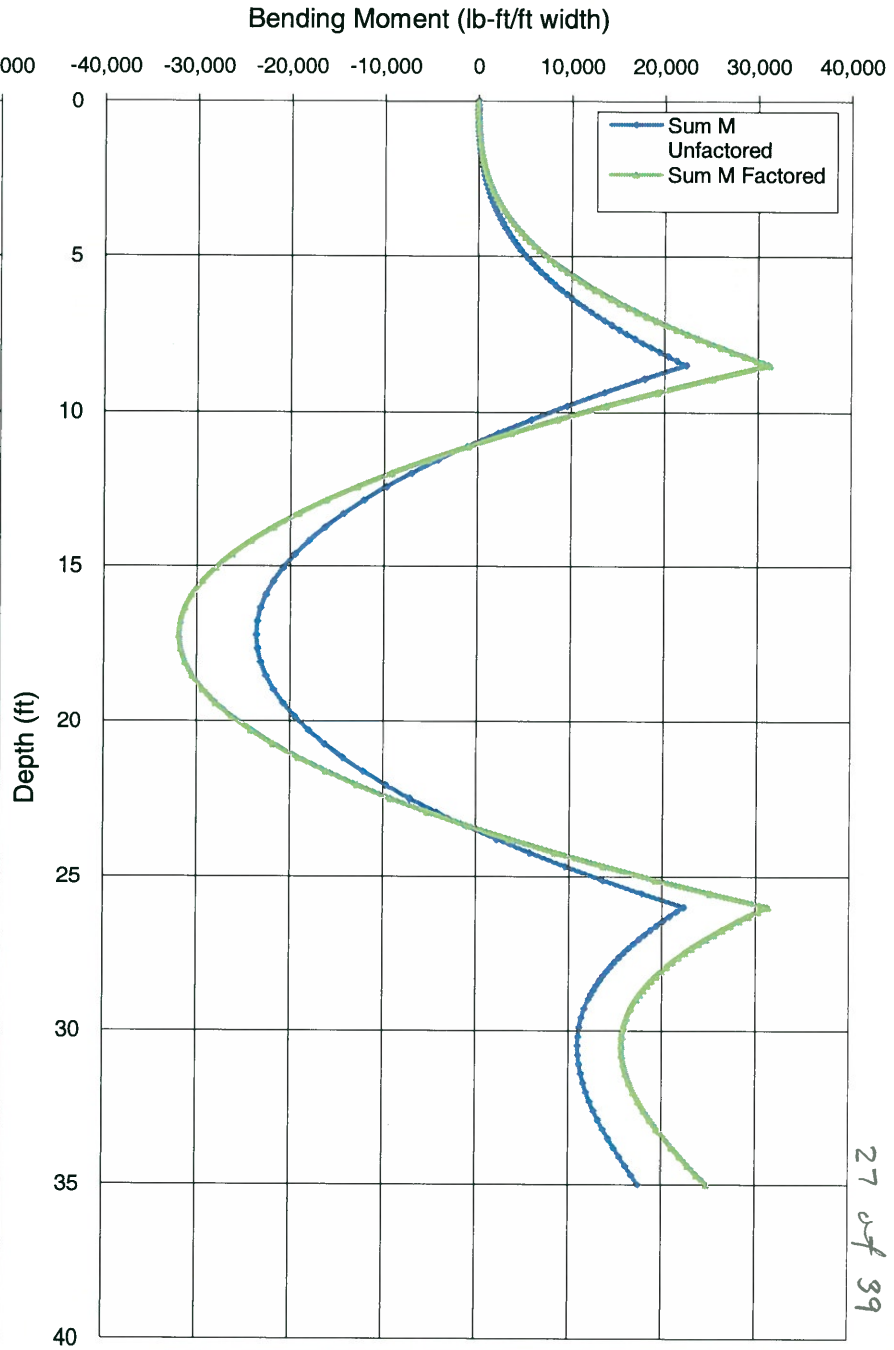

Active Pressures vs Depth
Pressure (psf/ft width)
0
200
400
600
800
1,000
1,200
1,400
1,600
Depth (ft)
0
5
10
15
20
25
30
35
40
pEH Unfactored
pLS Unfactored
pEH Factored
pLS Factored


Shear Force vs Depth
Force (lbs/ft width)
50,000
40,000
30,000
20,000
10,000
0
-10,000
-20,000
0
5
10
15
20
25
30
35
40
Depth (ft)
V LS Unfactored
V LS Factored
V EH Unfactored
V EH Factored
Sum V Unfactored
Sum V Factored

Shear Force vs Depth



Bending Moment vs Depth



STRENGTH-I Determine Embedment Depth L'= 25' SJR 3/25/09 EJT 3-27-09

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo

LPILE Plus for Windows, Version 5.0 (5.0.5)

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

This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations: M:\proj\0221\1004\01\geotechnical\Retaining Walls\LPILE\Final Anchored Wall\
Name of input data file: TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpd
Name of output file: TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo
Name of plot output file: TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpp
Name of runtime file: TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpr

Time and Date of Analysis

Date: March 25, 2009 Time: 15:22:37

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:
- User-specified p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo
- 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 = 300.00 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	36.00000000	808.0000	29.4000	29000000.
2	1000.0000	36.00000000	808.0000	29.4000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is modelled using user-specied p-y curves
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 84.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 84.000 in
Distance from top of pile to bottom of layer = 468.000 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

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 468.000 in
Distance from top of pile to bottom of layer = 588.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 588.000 in
Distance from top of pile to bottom of layer = 1000.000 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 700.00 in below pile tip)

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.03600
2	84.00	.03600
3	84.00	.03900
4	468.00	.03900
5	468.00	.03600
6	588.00	.03600
7	588.00	.03900
8	1000.00	.03900

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 6 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	84.000	.00000	38.00	------	------
2	468.000	.00000	38.00	------	------
3	468.000	10.40000	.00	------	------
4	588.000	10.40000	.00	------	------
5	588.000	.00000	38.00	------	------
6	1000.000	.00000	38.00	------	------

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.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.9850	1.0000
2	1000.000	.9850	1.0000

User-specified p-y Curves

User-specified p-y curves defined using 2 curves.

User-specified curve number 1 at depth = .000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.4700	5870.000
3	.9300	11730.000
4	1.4000	16720.000
5	1.8700	20410.000
6	2.3300	23170.000
7	2.8000	25340.000
8	3.2700	27100.000
9	3.7300	28860.000
10	4.2000	30620.000
11	100.0000	32260.000

User-specified curve number 2 at depth = 84.000in

Point No.	y in	p, lbs/in
1	.0000	.000
2	.4700	5870.000
3	.9300	11730.000
4	1.4000	16720.000
5	1.8700	20410.000
6	2.3300	23170.000
7	2.8000	25340.000
8	3.2700	27100.000
9	3.7300	28860.000
10	4.2000	30620.000
11	100.0000	32260.000

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 Shear and Moment (BC Type 1)
Shear force at pile head = 28600.000 lbs

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo
Bending moment at pile head = 2682288.000 in-lbs
Axial load at pile head = 473000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 28600.000 lbs
Specified moment at pile head = 2682288.000 in-lbs
Specified axial load at pile head = 473000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.248170	2682288.	28600.0000	-.0076409	75842.3760	-3052.9964
3.000	.225763	2764948.	19854.4965	-.0072922	77683.8172	-2777.3392
6.000	.204417	2822110.	11916.3678	-.0069345	78957.2254	-2514.7466
9.000	.184156	2856127.	4746.0149	-.0065710	79715.0177	-2265.4887
12.000	.164991	2869235.	-1696.8074	-.0062045	80007.0313	-2029.7262
15.000	.146929	2863554.	-7452.6784	-.0058375	79880.4826	-1807.5211
18.000	.129966	2841086.	-12562.2298	-.0054723	79379.9453	-1598.8466
21.000	.114095	2803711.	-17065.8942	-.0051110	78547.3489	-1403.5964
24.000	.099300	2753195.	-21003.6797	-.0047553	77421.9911	-1221.5940
27.000	.085563	2691185.	-24414.9715	-.0044067	76040.5670	-1052.6006
30.000	.072860	2619212.	-27338.3575	-.0040668	74437.2099	-896.3234
33.000	.061162	2538696.	-29811.4757	-.0037366	72643.5439	-752.4221
36.000	.050440	2450947.	-31870.8835	-.0034172	70688.7464	-620.5164
39.000	.040659	2357169.	-33551.9455	-.0031094	68599.6189	-500.1916
42.000	.031784	2258460.	-34888.7400	-.0028139	66400.6655	-391.0047
45.000	.023776	2155822.	-35913.9808	-.0025314	64114.1772	-292.4892
48.000	.016596	2050160.	-36658.9546	-.0022621	61760.3211	-204.1601
51.000	.010203	1942288.	-37153.4719	-.0020065	59357.2346	-125.5182
54.000	.004556	1832934.	-37425.8297	-.0017649	56921.1215	-56.0537
57.000	-.000386	1722742.	-37502.7854	-.0015372	54466.3516	4.7499
60.000	-.004667	1612280.	-37409.5403	-.0013238	52005.5617	57.4135
63.000	-.008329	1502042.	-37169.7317	-.0011244	49549.7578	102.4589
66.000	-.011413	1392453.	-36805.4329	-.0009391	47108.4172	140.4070
69.000	-.013963	1283874.	-36337.1588	-.0007678	44689.5921	171.7757
72.000	-.016020	1176609.	-35783.8784	-.0006103	42300.0110	197.0779
75.000	-.017625	1070903.	-35163.0305	-.0004664	39945.1811	216.8206
78.000	-.018818	966954.	-34490.5448	-.0003359	37629.4884	231.5032
81.000	-.019640	864913.	-33780.8648	-.0002187	35356.2976	241.6168
84.000	-.020130	764889.	-32606.0094	-.0001143	33128.0492	541.6201
87.000	-.020326	669601.	-30944.7263	-2.2509E-05	31005.2974	565.9019
90.000	-.020265	579285.	-29217.0101	5.7439E-05	28993.2960	585.9089
93.000	-.019982	494136.	-27435.4878	.0001262	27096.4220	601.7726
96.000	-.019508	414314.	-25612.3915	.0001843	25318.2006	613.6249
99.000	-.018876	339939.	-23759.5488	.0002326	23661.3311	621.6036
102.000	-.018113	271097.	-21888.3598	.0002717	22127.7142	625.8557
105.000	-.017246	207838.	-20009.7678	.0003024	20718.4812	626.5390

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo

108.000	-.016299	150180.	-18134.2265	.0003253	19434.0249	623.8219
111.000	-.015294	98109.0944	-16271.6676	.0003412	18274.0340	617.8840
114.000	-.014252	51581.5251	-14431.4698	.0003508	17237.5288	608.9145
117.000	-.013189	10524.8153	-12622.4320	.0003547	16322.8991	597.1107
120.000	-.012123	-25159.8101	-10852.7515	.0003538	16648.9262	582.6762
123.000	-.011067	-55595.7784	-9130.0102	.0003486	17326.9552	565.8179
126.000	-.010031	-80929.2848	-7492.2084	.0003399	17891.3155	526.0500
129.000	-.009027	-101514.	-5988.0420	.0003282	18349.8778	476.7277
132.000	-.008062	-117789.	-4629.8449	.0003142	18712.4478	428.7370
135.000	-.007142	-130184.	-3413.0553	.0002983	18988.5814	382.4560
138.000	-.006272	-139114.	-2332.0842	.0002811	19187.5075	338.1914
141.000	-.005456	-144974.	-1380.5196	.0002629	19318.0650	296.1849
144.000	-.004695	-148143.	-551.3142	.0002441	19388.6521	256.6187
147.000	-.003991	-148975.	163.0459	.0002251	19407.1888	219.6214
150.000	-.003345	-147804.	770.3887	.0002061	19381.0896	185.2738
153.000	-.002755	-144938.	1278.7208	.0001874	19317.2458	153.6143
156.000	-.002220	-140663.	1696.1080	.0001691	19222.0164	124.6439
159.000	-.001740	-135241.	2030.5711	.0001514	19101.2273	98.3315
162.000	-.001312	-128909.	2289.9964	.0001345	18960.1758	74.6187
165.000	-.000933	-121883.	2482.0599	.0001184	18803.6420	53.4237
168.000	-.000601	-114353.	2614.1642	.0001033	18635.9036	34.6459
171.000	-.000313	-106491.	2693.3873	8.9183E-05	18460.7562	18.1695
174.000	-6.63E-05	-98445.7974	2726.4421	7.6064E-05	18281.5348	3.8670
177.000	.000143	-90348.0619	2719.6461	6.3979E-05	18101.1397	-8.3977
180.000	.000318	-82309.4916	2678.8998	5.2926E-05	17922.0627	-18.7665
183.000	.000461	-74424.8667	2609.6730	4.2892E-05	17746.4151	-27.3846
186.000	.000575	-66773.1823	2516.9989	3.3854E-05	17575.9568	-34.3981
189.000	.000664	-59418.9499	2405.4734	2.5776E-05	17412.1249	-39.9522
192.000	.000730	-52413.4926	2279.2606	1.8617E-05	17256.0627	-44.1896
195.000	.000775	-45796.2198	2142.1027	1.2330E-05	17108.6482	-47.2490
198.000	.000804	-39595.8680	1997.3334	6.8633E-06	16970.5215	-49.2639
201.000	.000817	-33831.6974	1847.8953	2.1628E-06	16842.1118	-50.3616
204.000	.000817	-28514.6346	1696.3589	-1.8283E-06	16723.6624	-50.6626
207.000	.000806	-23648.3550	1544.9448	-5.1675E-06	16615.2552	-50.2801
210.000	.000786	-19230.3005	1395.5456	-7.9124E-06	16516.8332	-49.3193
213.000	.000758	-15252.6260	1249.7509	-1.0120E-05	16428.2216	-47.8772
216.000	.000725	-11703.0751	1108.8711	-1.1845E-05	16349.1474	-46.0427
219.000	.000687	-8565.7821	973.9625	-1.3143E-05	16279.2573	-43.8964
222.000	.000646	-5822.0004	845.8518	-1.4064E-05	16218.1334	-41.5108
225.000	.000603	-3450.7579	725.1603	-1.4658E-05	16165.3087	-38.9503
228.000	.000558	-1429.4408	612.3274	-1.4970E-05	16120.2794	-36.2716
231.000	.000513	265.6911	507.6337	-1.5044E-05	16094.3542	-33.5241
234.000	.000468	1659.0575	411.2225	-1.4921E-05	16125.3946	-30.7500
237.000	.000423	2775.3727	323.1206	-1.4637E-05	16150.2630	-27.9846
240.000	.000380	3639.3220	243.2577	-1.4227E-05	16169.5094	-25.2573
243.000	.000338	4275.2943	171.4847	-1.3720E-05	16183.6771	-22.5914
246.000	.000298	4707.1674	107.5903	-1.3145E-05	16193.2980	-20.0048
249.000	.000259	4958.1420	51.3171	-1.2526E-05	16198.8890	-17.5106
252.000	.000222	5050.6199	2.3750	-1.1886E-05	16200.9492	-15.1174
255.000	.000188	5006.1235	-39.5456	-1.1242E-05	16199.9579	-12.8297
258.000	.000155	4845.2506	-74.7627	-1.0611E-05	16196.3741	-10.6484
261.000	.000124	4587.6620	-103.5924	-1.0007E-05	16190.6358	-8.5714
264.000	9.50E-05	4252.0972	-126.3404	-9.4415E-06	16183.1603	-6.5940
267.000	6.75E-05	3856.4144	-143.2948	-8.9224E-06	16174.3456	-4.7090
270.000	4.14E-05	3417.6501	-154.7197	-8.4567E-06	16164.5711	-2.9076
273.000	1.67E-05	2952.0965	-160.8505	-8.0490E-06	16154.1999	-1.1796
276.000	-6.86E-06	2475.3902	-161.8905	-7.7015E-06	16143.5802	.4862550
279.000	-2.95E-05	2002.6105	-158.0085	-7.4149E-06	16133.0480	2.1018
282.000	-5.13E-05	1548.3828	-149.3375	-7.1876E-06	16122.9290	3.6789
285.000	-7.26E-05	1126.9836	-135.9752	-7.0163E-06	16113.5414	5.2293
288.000	-9.34E-05	752.4436	-117.9851	-6.8960E-06	16105.1977	6.7641
291.000	-.000114	438.6439	-95.3990	-6.8197E-06	16098.2071	8.2933

TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo

294.000	-.000134	199.4041	-68.2211	-6.7789E-06	16092.8775	9.8253
297.000	-.000155	48.5560	-36.4329	-6.7630E-06	16089.5171	11.3668
300.000	-.000175	0.0000	0.0000	-6.7599E-06	16088.4354	12.9218

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.24817031 in
Computed slope at pile head	=	-.00764087
Maximum bending moment	=	2869235. lbs-in → 239 k-ft
Maximum shear force	=	-37502.78543 lbs
Depth of maximum bending moment	=	12.00000000 in
Depth of maximum shear force	=	57.00000000 in
Number of iterations	=	7
Number of zero deflection points	=	3

Summary of Pile-Head 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 = rotational stiffness of pile-headin-lbs/rad

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 28600.	M= 2.68E+06	473000.	.2481703	2869235.	-37502.7854

Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

Shear	=	28600. lbs
Moment	=	2682288. in-lbs
Axial Load	=	473000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
300.000	.24817031	2869235.	-37502.78543
285.000	.24817852	2868366.	-37409.05820
270.000	.24819722	2869019.	-37355.08093

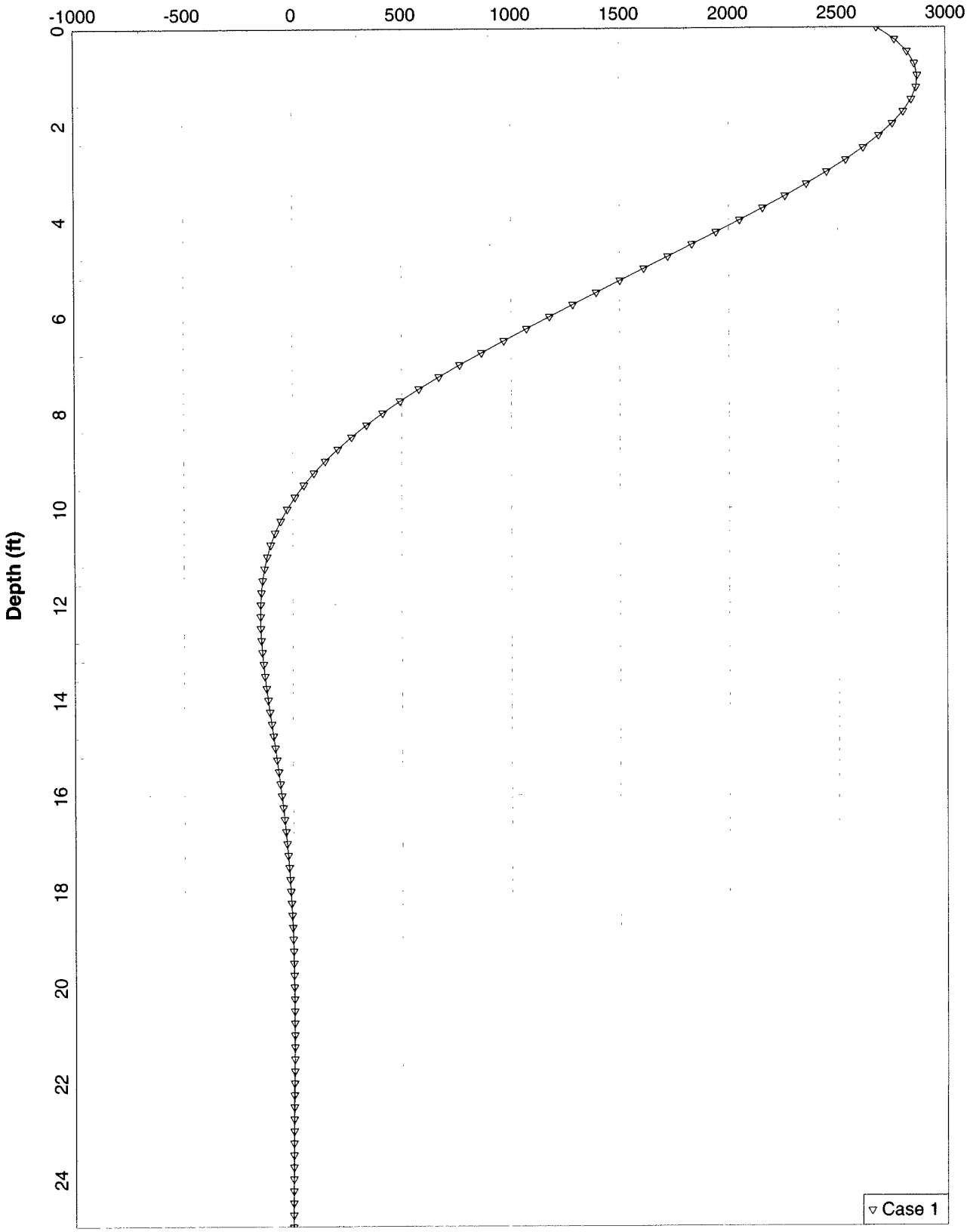
TB Profile A, 35 ft 36 in on 108 in centers STRENGTH.lpo

255.000	.24825537	2869912.	-37491.33936
240.000	.24825343	2869829.	-37345.91998
225.000	.24830347	2869346.	-37415.57519
210.000	.24836480	2870326.	-37381.05950
195.000	.24847844	2869593.	-37375.33808
180.000	.24860892	2870407.	-37344.35269
165.000	.24861496	2870211.	-37146.04333

The analysis ended normally.

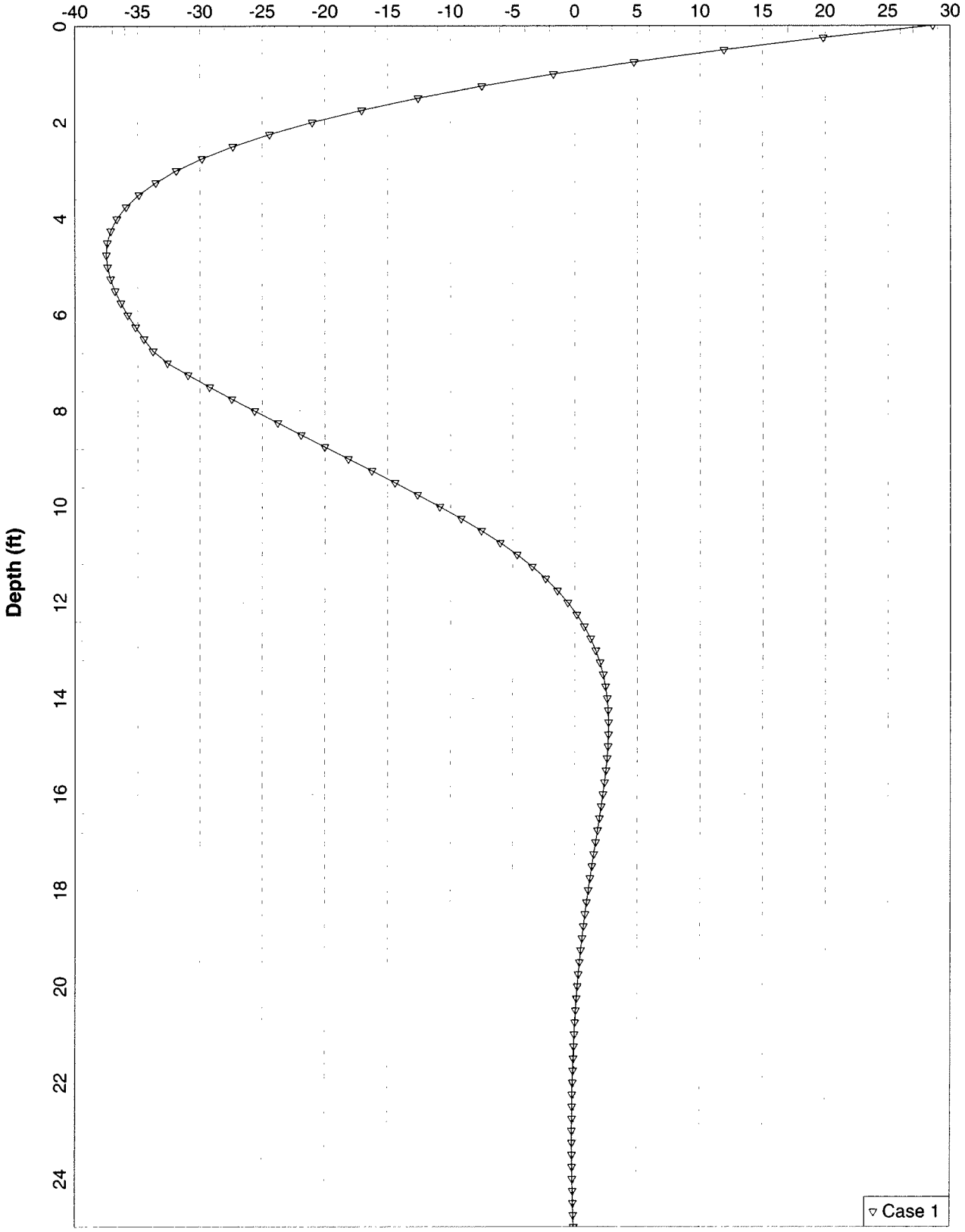
Profile A, 35 ft Wall 2C15x51 on 9ft Centers

~~Unfactored~~ Bending Moment (in-kips)



Profile A, 35 ft Wall 2C15x51 on 9ft Centers

Shear Force (kips)



Check Combined Axial Compression and Flexural Resistance:
AASHTO 6.9.2.2

- Consider 25-foot wall:

$P_u = 51 + 322 = 373^{k}$

* Use $M_u = 111$ k-ft from LPILE analyses

Axial Resistance: $P_r = \phi P_n$

$$\lambda = \left(\frac{Kl}{r_s \pi}\right)^2 \left(\frac{F_y}{E}\right) = \left(\frac{1.0(11.5' \cdot 12)}{(4.29)\cdot \pi}\right)^2 \left(\frac{50{,}000\ psi}{29\times10^6\ psi}\right) = 0.18$$

for $\lambda \le 2.25 \longrightarrow P_n = 0.66^{\lambda} F_y A_s$

$P_n = 0.66^{0.18}(50{,}000\ psi)(17.62\ in^2) = 817^{k}$

$P_r = \phi_c P_n = 0.70(817^{k}) = \boxed{572^{k}}$ ✓

$$\frac{P_u}{P_r} = \frac{373}{572} = 0.65 \longrightarrow \frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$

$$\frac{373}{572} + \frac{8}{9}\left(\frac{111}{282}\right) = 1.00 \le 1.0 \quad ✓ \boxed{OK}$$

- Consider 35-foot wall:

$$\frac{P_u}{P_r} = \frac{473}{916} = 0.52 \longrightarrow \frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$

$$\frac{473}{916} + \frac{8}{9}\left(\frac{282}{571}\right) = 0.96 < 1.0 \quad ✓ \boxed{OK}$$

- Free Stressing Length: Use minimum length of 20'

- Assume medium dense sandy silt / glacial till in bond zone.
  From table C11.9.4.2-2;
  Assume $^{*}T_n \simeq 4.5$ ksf
  * Presumptive value assuming pressure grouting

$$\phi Q_n = \phi \pi d\, T_a\, L_b \qquad \phi = 0.65 \ [11.5.6\text{-}1]$$
$$\{\text{Eq}^{n}\ 11.9.4.2\text{-}1\}$$

$$\phi Q_n / L_b = \phi \pi d \cdot T_a = 0.65 \cdot \pi (0.5')(4.5\ \text{ksf}) = 4.59\ \text{k/ft}$$

Bond Length - L_b

- Consider 25-foot wall;

  Factored Anchor Loads: $T_1 = 98.6^{k}$, $T_2 = 98.4^{k}$ } Say 99^{k}

$$L_b = \frac{99^{k}}{4.59\ \text{k/ft}} = 22\ \text{ft}$$

- Consider 35-foot wall;

  Factored Anchor Loads: $T_1 = 225.6^{k}$, $T_2 = 201.3^{k}$ } Say 226^{k}

$$L_b = \frac{226^{k}}{4.59\ \text{k/ft}} = 49\ \text{ft}$$

* During final design, possibly consider adding a third row to lower anchor loads.

Calculations: Check of Axial Capacity of Drilled Shafts

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT ms consultants | PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 | SHEET NO. 1 OF 5
SUBJECT Drilled Shaft Resistance | COMP. BY SJK DATE 3/26/09
Check Axial Capacity of Drilled Shafts | CHECKED BY DAA DATE 3/30/09

Drilled Shafts

- Assume shafts are founded in glacial till and alluvium
- Factored Axial Load from bridge or cap is assumed to be 40.25 k/ft (STRENGTH).
  *Assume Load Factor = 1.75 (Conservative)

Average N_{60} Value in boring B-029: $\overline{N_{60}}$-Value = 55 bpf*
*Excludes "50+" N-values, which may have encountered boulders.

Although average N_{60} values are > 50 bpf, do not treat as intermediate geomaterial.

Say $N_{60} = 50$ bpf.

In cohesionless soil:

* Tip Resistance $q_p = 1.2\, N_{60}$ (ksf) where $N_{60} \leq 50$ {10.8.3.5.2c-1}

$q_p = 1.2(50) = \boxed{60 \text{ ksf}}$

* Side Resistance (β-method)

$q_s = \beta \cdot \sigma_v' \leq 4.0$ for $0.25 \leq \beta \leq 1.2$ {10.8.3.5.2b-1}

where $\beta = 1.5 - 0.135\sqrt{z}$ {10.8.3.5.2b-2}

- Assume 45' minimum embedment (from lateral analyses) for 48-inch diameter drilled shafts.
- Factored axial load (48-in on 120-in centers) is 402.5 k.

$\beta = 1.5 - 0.135\sqrt{22.5'} = 0.86$

$\sigma_v' = 22.5(125 - 62.4) = 1409$ psf * conservative estimate

$q_s = 0.86(1409 \text{ psf}) = \boxed{1{,}212 \text{ psf}}$

Drilled Shafts (cont.)

Shafts in cohesive soil:

Consider soil below the bottom of excavation. Nearly all cohesive soil below this depth is glacial till. The till is characterized by very high shear strength.

Say $s_u = 4.5$ ksf

* Tip Resistance

$q_p = N_c \cdot s_u \leq 80.0$ ksf {10.8.3.5.1c-1}

where $N_c = 6\left[1+0.2\left(\frac{Z}{D}\right)\right] \overset{?}{\leq} 9$ {10.8.3.5.1c-2}

Assume 48-inch diameter shafts. Minimum embedment = 45'.

$N_c = 6\left[1+0.2\left(\frac{45}{4}\right)\right] = 19.5 > 9$ Use $N_c = 9$

$q_p = 9 \cdot (4.5 \text{ ksf}) = \boxed{40.5 \text{ ksf}}$

* Side Resistance

$q_s = \alpha \cdot s_u$ {10.8.3.5.16-1}

where $\alpha = 0.55 - 0.1\left(s_u/p_a - 1.5\right)$ {10.8.3.5.16-3}

for $1.5 \leq s_u/p_a \leq 2.5$

p_a = atmospheric pressure = 2.12 ksf

$\alpha = 0.55 - 0.1\left[\left(4.5/2.12\right) - 1.5\right] = 0.49$

$q_s = 0.49\ (4.5 \text{ ksf}) = \boxed{2.2 \text{ ksf}}$

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CLIENT ms consultants
PROJECT FRA-70-8.93
SUBJECT Drilled Shaft Resistance
Check Axial Capacity

PROJECT NO. 0221-1004.01
SHEET NO. 3 OF 5
COMP. BY SJK DATE 3/26/09
CHECKED BY DAA DATE 3/30/09

As a check, use lesser resistance values between cohesive and cohesionless soils;

Use $q_p = 40.5$ ksf & $q_s = 1.2$ ksf

Consider 48" diameter drilled shafts on 10' centers. In terms of axial resistance this is the most critical case.

Resistance factors: $\phi_{qp} = 0.50$ $\phi_{qs} = 0.55$ (P)

$$\phi_{qp} \cdot R_p = 0.50(\pi 2^2)(40.5 \text{ ksf}) = 254 \text{ ksf}$$

$$\phi_{qs} \cdot R_s = 0.55(2 \cdot \pi \cdot 2')(45' - 5')(1.2 \text{ ksf}) = 332 \text{ ksf}$$
(Neglects top 5 feet)

• Factored Axial Resistance
$R_R = \phi R_n = 254 + 332 = 586$ k

• Factored Axial Load = 402.5 k per shaft

∴ Embedment depth is based upon lateral analyses. The axial resistance provided by this embedment depth is greater than the factored axial load.

Axial Resistance for drilled shaft retaining walls is OK ✓

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PROJECT FRA-70-8.93
SUBJECT Soldier Pile Walls - Axial Resistance Check -

PROJECT NO. 0221-1004.01
SHEET NO. 4 OF 5
COMP. BY SJK DATE 3/26/09
CHECKED BY DAA DATE 3/30/09

- Consider embedded shafts for Soldier-Pile walls.

Most critical case is 35-ft high wall on 9-ft Centers.

Factored Axial Load = 473 k (362 + 111 = 473 k)

- Consider 50' Shafts below bottom of excavation.

Use cohesionless side resistance value. (most conservative)

$\beta = 1.5 - 0.135\sqrt{25'} = 0.83$

$\sigma_v' = 25'(125 - 62.4\ pcf) = 1565\ psf$

$q_s = \beta \cdot \sigma_v' = 0.83(1565\ psf) = \boxed{1.30\ ksf}$

$A_p = \pi(1.5)^2 = 7.07\ ft^2$

A_s / ft length $= 2\pi(1.5') = 9.42\ ft^2$ per foot length

$\phi_{qp} \cdot R_p = 0.50(7.07\ ft^2)(40.5\ ksf) = 143\ k$

$\phi_{qs} \cdot R_s = 473\ k - 143\ k = 330\ k$ $\qquad R_s = 330^k / 0.55 = 600\ k$

Determine Required Shaft Length

$L = 600\ k / (9.42\ ft^2 \cdot 1.30\ ksf) = 49\ ft$

Neglect side resistance in upper 5'

$\therefore$ Required Length $= 49' + 5' = \boxed{54'}$

For 35' walls <u>with</u> structure load

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PROJECT FRA-70 - 8.93
SUBJECT Soldier Pile Walls - Axial Resistance Check -

PROJECT NO. 0221-1004.01
SHEET NO. 5 OF 5
COMP. BY SJR DATE 3-26-09
CHECKED BY DAA DATE 3/30/09

- Consider 25-ft high wall on 8-ft Centers.

Factored Axial Load = 373 k (322 + 51 = 373)

Use q_s = 1.30 ksf (see previous page)

$\phi_{qp} \cdot R_p = 143^k$

$\phi_{qs} \cdot R_s = 373 - 143 = 230^k$ $\quad R_s = 230^k / 0.55 = \underline{418^k}$

$L = 418^k / (9.42\ ft^2 \cdot 1.30\ ksf) = 34$ ft.

Reduce shaft length *

- Consider L = 40 ft.

$\beta = 1.5 - 0.135\sqrt{20} = 0.90$

$\sigma_v' = 20'(125 - 62.4\ pcf) = 1252\ psf$

$q_s = 0.90\ (1.25\ ksf) = \boxed{1.13\ ksf}$

$L = 418^k / (9.42\ ft^2 \cdot 1.13\ ksf) = 39$ ft

Required Length = 39 + 5' = $\boxed{44'}$

For 25' walls <u>with</u> structure load

Calculations: Retaining Walls Supported on Spread Footings

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CLIENT ODOT DISTRICT 6 | PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 | SHEET NO. 1 OF 8
SUBJECT Retaining Wall Evaluations Spread Footing Foundations | COMP. BY DAA DATE 3/25/09
CHECKED BY GWT DATE 3-30-09

CALCULATIONS BASED ON AASHTO LRFD MANUAL, 4TH ED, WITH 2008 INTERIM REVISIONS

Consider concrete gravity walls supported on spread footings at sections with shorter wall heights (less than 15 feet) or with primarily new fill behind wall
- west end, approximate Sta. 729+50 to 734+50 (base of wall varies el. 725-735)
- south wall, between Sta. 746+00 to 762+50 (base of wall varies el. 726-735)

A. Spread Footings bearing on cohesive soil - applies to walls at west end and south wall Sta 759+00 to 762+50

Based on B-024, B-025 and B-039, let $\gamma = 130$, $c = 3000$ psf
$c' = 0$
$\phi_f = 0$
$\phi_f' = 32°$

Calculate nominal bearing resistance - use lesser value from total stress vs. effective stress analyses
Neglect shape and inclination factors, assume $B > D$

1. Total stress - assume $c = 3000$ psf, $\phi_f = 0$
$D_f = D_w = 4.0'$
$N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$ [Table 10.6.3.1.2a-1]
$C_{wq} = 1.0$, $C_{w\gamma} = 0.5$ [Table 10.6.3.1.2a-2]

$$q_n = cN_c + \gamma D_f N_q C_{wq} + 0.5\gamma B' N_\gamma C_{w\gamma}$$
$$= 3000(5.14) + 130(4)(1)(1) = 15940 \text{ psf} = 15.9 \text{ ksf}$$

2. Effective stress - assume $c' = 0$, $\phi_f' = 32°$
$N_q = 23.2$, $N_\gamma = 30.2$
C_{wq}, $C_{w\gamma}$, D_f, D_w same as above

$$q_n = c N_c + 130(4)(23.2)(1) + 0.5(130 - 62.4) B'(30.2)(0.5)$$
$$= 12{,}064 + 510 B'$$

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CLIENT ODOT DISTRICT 6 PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 SHEET NO. 2 OF 8
SUBJECT Retaining Wall Evaluations COMP. BY DAA DATE 3/25/09
Spread Footing Foundations CHECKED BY GWT DATE 3-30-09

A. Spread Footings bearing on cohesive soil, cont

for a 10-foot wide footing, $q_n = 12{,}064 + 510(10)$
$= 17{,}164$ psf $> 15{,}940$ psf (Total stress result)

for a 5-foot wide footing, $q_n = 12{,}064 + 510(5)$
$= 14{,}614$ psf $< 15{,}940$ psf

Consequently, use $q_n =$ **12,064 + 510 B' up to a maximum of 15,940 psf**

See following sheet for q_n values for other footing widths

Resistance factor **$(\phi_r) = 1.0$** for service and extreme limit states
$= 0.5$ for strength limit state

Nominal Sliding Resistance (R_r) = the lesser of

- cohesion of the clay = 3000 psf
  OR
- one half the normal stress (maximum value is one half factored bearing resistance)

} Section 10.6.3.4

$= 0.5\,(0.5)(14{,}614) = 3653$ psf > 3000 psf

Consequently, use **$R_r = 3000$ psf** (neglect passive resistance)

Sliding Resistance Factor (ϕ_r) = **0.85** [Table 10.5.5.2.2-1]

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CLIENT ODOT DISTRICT 6 | PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 | SHEET NO. 3 OF 8
SUBJECT Retaining Wall Evaluations, Spread Footing Foundations | COMP. BY DAA DATE 3/25/09
CHECKED BY GWY DATE 3-30-09

B. Spread footings bearing on granular soil – applies to wall on south side Sta. 746+00 to 759+00

Based on B-031, B-035, and B-036, let $\gamma' = 120$ pcf
$c = 0$
$\phi_f = 34°$, $N_{60} = 50+$

$D_f = D_w = 4.0$ feet
$N_q = 29.4$, $N_\gamma = 41.1$ [Table 10.6.3.1.2a-1] for $\phi_f = 34°$

C_{wq}, $C_{w\gamma}$ same as above

Nominal Bearing Resistance (q_n)

$= c N_c^\circ + 120(4)(29.4)(1) + 0.5(120-62.4)(B')(41.1)(0.5)$
$= 14{,}112 + 592\,B'$

for a 10-foot wide footing, $q_n = 20{,}032$ psf = **20.0 ksf**

See attached sheet for q_n values for other values of B'

Resistance factor (ϕ_r) = 1.0 for service limit state
= 0.45 for strength limit state

Nominal sliding resistance (R_τ) = $P_v \tan\delta$ [Equ. 10.6.3.4-2]
$\tan\delta = \tan\phi_f = \tan 34 =$ **0.67**

Sliding resistance factor (ϕ_τ) = **0.8** for strength limit state [Table 10.5.5.2.2-1]

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CLIENT	ODOT District 6	JOB NUMBER	0221-1004.01
PROJECT	FRA-70-8.93	SHEET NO.	4 OF 8
SUBJECT	Retaining Wall Evaluations	COMP. BY	DAA DATE 3/26/2009
	Additional Resistance Values for Spread Footings	CHECKED BY	DATE 3-30-09

A. Additional Nominal Bearing Resistance Values (q_n) for Spread Footings on Cohesive Soil

Effective Footing Width, B' (ft)	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
Nominal Bearing Resistance, q_n (ksf)	14.6	15.1	15.6	15.9*	15.9*	15.9*	15.9*	15.9*	15.9*	15.9*	15.9*

where q_n = (12,064 psf + 510 pcf (B'))/1000 psf/ksf

* - maximum value of 15.9 ksf

B. Additional Nominal Bearing Resistance Values (q_n) for Spread Footings on Granular Soil

Effective Footing Width, B' (ft)	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
Nominal Bearing Resistance, q_n (ksf)	17.1	17.7	18.3	18.8	19.4	20.0	20.6	21.2	21.8	22.4	23.0

where q_n = (14,112 psf + 592 pcf (B'))/1000 psf/ksf

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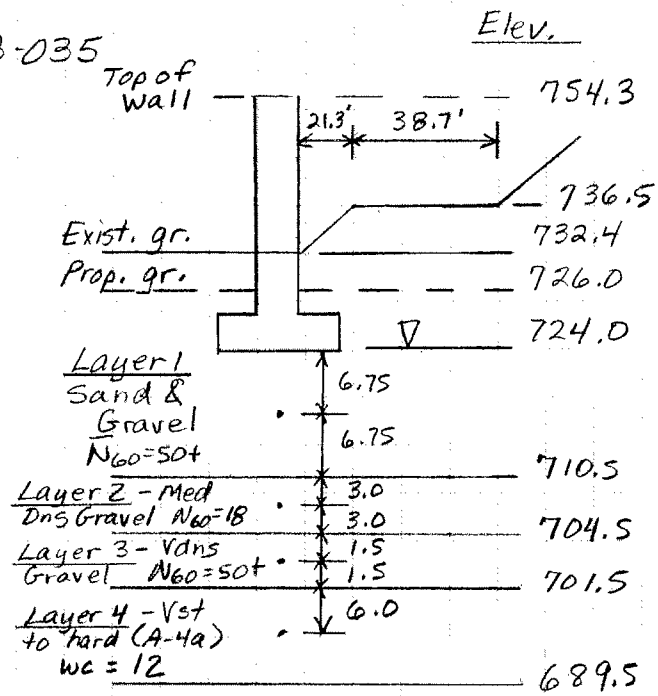
CLIENT ODOT DISTRICT 6 PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 SHEET NO. 5 OF 8
SUBJECT Retaining Wall Evaluations COMP. BY DAA DATE 3/25/09
Spread Footing Foundations CHECKED BY [illegible] DATE 3-30-2009

SETTLEMENT

Consider section at Sta. 752+50 R
- base of wall at el. 726.0 (lowest point of wall)
- 6.4 feet cut to base of wall
- 21.9 feet new fill next to wall - actual height varies
- total wall height of 28.3 feet

Use profile based on B-034, B-035

γ_T - granular layers = 120pcf
γ_T - cohesive layers = 130pcf



— ASSUME INCOMPRESSIBLE —

Look at at effect of new fill within 60 feet of new wall
- in the first 21.3', height of fill varies between 17.8 and 21.9 feet - average 19.9 feet
- in the next 38.7', fill is 17.8 feet

Although height of fill varies, use maximum height (21.9 feet) to determine settlement of retaining wall.

Analyze settlement under wall using EMBANK, end of fill condition.

Estimated Settlement = **0.75 inches** MAXIMUM

(see following sheets for input values and printout)

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CLIENT ODOT DISTRICT 6 PROJECT NO. 0221-1004.01
PROJECT FRA-70-8.93 SHEET NO. 6 OF 8
SUBJECT Retaining Wall Evaluations COMP. BY DAA DATE 3/30/09
Spread Footing Foundations CHECKED BY GWT DATE 3-30-09

SETTLEMENT, cont.

Determine input propeties - assume soil is overconsolidated

Layer 1

for $\bar{N}_{60}$ = 50+, use C' = 170 [Fig. 10.6.2.4.2-1]

but program requires values in terms of e_o, C_c, C_r

$\frac{1}{C'} = \frac{C_r}{1+e_o}$ (let $e_o = 0.33$) [Equ. 10.6.2.4.2-3 & 10.6.2.4.3-1]

$C_r = \frac{1+e_o}{C'} = \frac{1.33}{170} = 0.0078 \simeq 0.008$

$C_c \rightarrow$ value not used, enter zero

Layer 2

for N_{60} = 18, use C' = 65 $\Rightarrow C_r = \frac{1+e_o}{C'} = \frac{1.33}{65} = 0.020$

$C_c \rightarrow$ value not used, enter zero

Layer 3

use same values e_o, C_r, C_c as Layer 1

Layer 4

$C_r = \frac{wc}{1000} = \frac{12}{1000} = 0.012$ (FHWA Soils and Foundations Workshop Manual)

based on B-034, S-20

$C_c = \frac{wc}{100} = \frac{12}{100} = 0.12$

$e_o = \frac{2.75 \times wc}{100} = \frac{2.75 \times 12}{100} = 0.33$

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿
³ INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION ³

Project Name : FRA-70-8.93 Client : ODOT D6
File Name : RET WALL01 Project Manager : D. Adams
Date : 3/30/10 Computed by : DAA

Settlement for X-Direction

Embank. slope, x direc. = 0.10 (ft) Height of fill H = 21.90 (ft)
y direc. = 0.10 (ft) Unit weight of fill = 120.00 (pcf)
Embankment top width = 100.00 (ft) p load/unit area = 2628.00 (psf)
Embankment bottom width = 100.20 (ft) Foundation Elev. = 735.80 (ft)
Ground Surface Elev. = 732.40 (ft)
Water table Elev. = 724.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	8.4	-----	-----	-----	120.00	----	----
2	COMP.	13.5	0.000	0.008	0.000	120.00	2.65	0.33
3	COMP.	6.0	0.000	0.020	0.000	120.00	2.65	0.33
4	COMP.	3.0	0.000	0.008	0.000	120.00	2.65	0.33
5	COMP.	12.0	0.120	0.012	0.012	130.00	2.65	0.33

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	13.50	717.25	1396.80	7602.59
3	6.00	707.50	1958.40	8444.92
4	3.00	703.00	2217.60	8833.69
5	12.00	695.50	2709.60	9481.64

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 10.00 Stress (psf)	Sett. (in.)	X = 20.00 Stress (psf)	Sett. (in.)	X = 30.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	650.80	0.16	1031.10	0.23	1201.75	0.26	1260.10	0.27
3	648.32	0.13	919.40	0.18	1097.53	0.21	1188.08	0.22
4	645.79	0.02	883.01	0.03	1053.23	0.04	1149.98	0.04
5	639.77	0.12	834.19	0.15	986.90	0.18	1085.29	0.19
		0.44		0.60		0.68		0.72

Layer	X = 40.00 Stress (psf)	Sett. (in.)	X = 50.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	1280.14	0.28	1285.20	0.28
3	1227.82	0.23	1239.04	0.23

FRA70RW2.TXT

3	4	1196.17	0.04	1209.76	0.04
3	5	1137.40	0.20	1153.62	0.20
3			------		------
3			0.74		0.75

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ