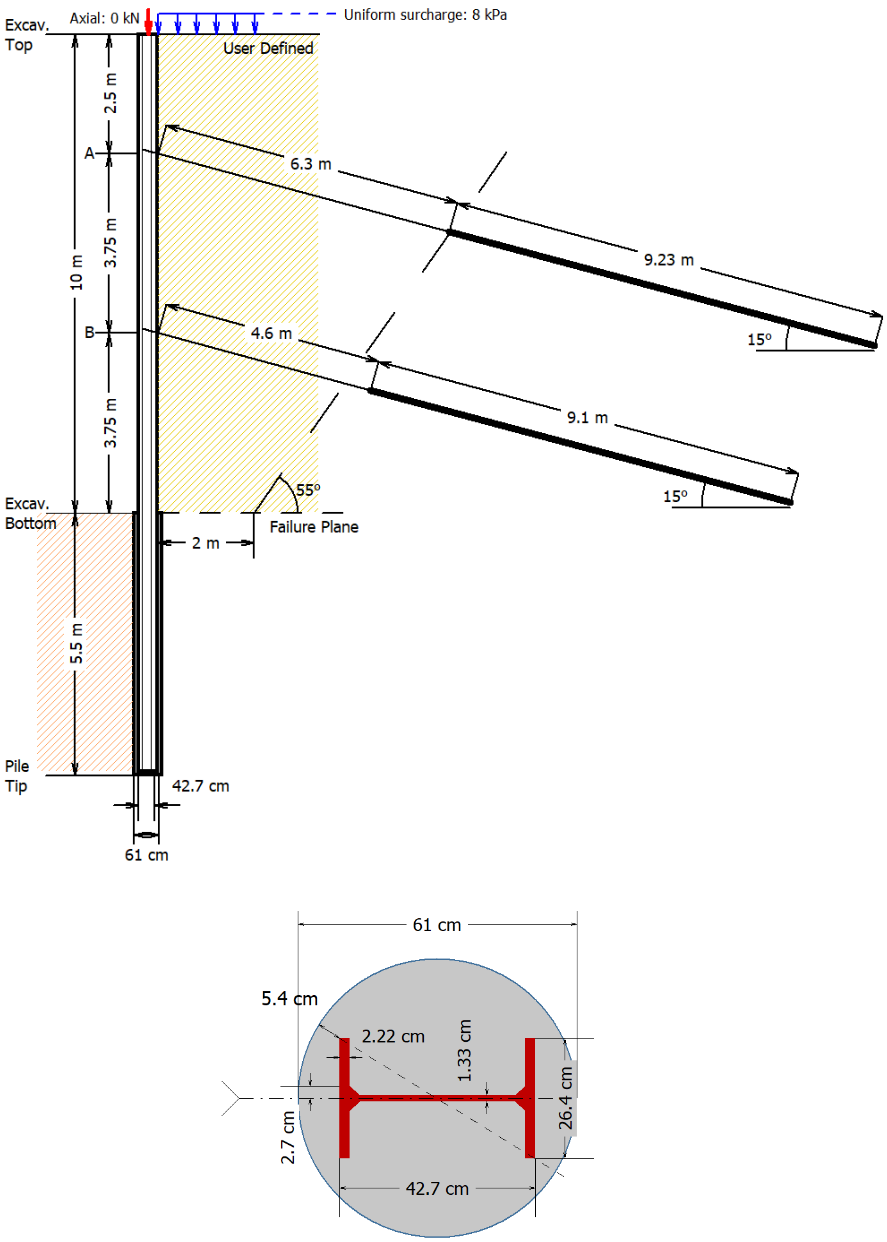
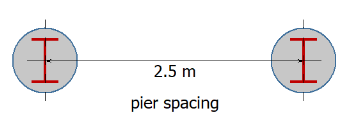
Anchored or Braced Shoring Analysis

Organization:	**FEC**
Project Name:	**FHWA CIRCULAR 4 1999**
Design by:	**EXAMPLE 1 Page 207**
Job #:	
Date:	**1/1/2022**





Inputs

General Data

Units	SI
Analysis Method	Apparent Earth Press.
Installation Method	Drilled
Pile Type	Soldier Beam (King Pile)
Reinforcement	I-Beam
Shored Height, H	10.00 m
Pile Spacing, S	2.50 m
Pile Width or Pier Diameter, B	0.61 m

Earth Pressure and Surcharges

Loading Height		10.00 m
Pile Spacing		2.50 m
Soil Unit Weight		18.0 kN/m^3
Vert. Uniform Surcharge		8.00 kPa
Water Depth, GWT		17.00 m
Soil/Bedrock Type		User Defined
Unfactored Earth Pressure	**x, m**	**w, kPa**
	0.00	0.00
	1.67	43.60
	7.50	43.60
	10.00	0.00
Act. Earth Press. Coeff, Ka		0.40
Seismic Horiz. Accel, Kh		0.00 g

Structural data

I-Beam

Beam Type	North America
Beam Size	W410X132
Beam Diagonal Length (<61 cm - 10 cm, O.K.)	50.20 cm
Pipe Filled with Concrete	No
Steel Modulus of Elasticity	200000 MPa
Yield Strength, Fy	345 MPa
Conc. Compress. Str, f'c	21.00 MPa

Loads Applied to the Pile

Axial Load	0.00 kN
Vertical Axial Component	308.33 kN
Total Vertical Load	308.33 kN
Drilled Pier Diameter	0.61 m
Drilled Pier Embedment	5.50 m
Allow. Skin Friction	40.00 kPa
Allow. End Bearing	200.00 kPa

Anchors or Braces

Shored Height, H	10.00 m
Loading Height, L	10.00 m
Number of Anchors	2
Anchor Spacing	2.50 m

Level	Anchor Type	Ht. from Prev, m	Angle, deg
A	Tieback	2.50	15.00
B	Tieback	3.75	15.00
C		3.75	

Embedment Ratio	0.55 H
Required Embedment	5.50 m
Unbraced Length, Lb	3.75 m
Modification Factor, Cb	1.00
Total Vertical Load, P	308.33 kN
Effect. Length Factor, K	1.00
Max Deflection	12.50 mm
Test Load Factor	1.33

Bond Length

Soil/Bedrock Type	User Defined
Allow. Load Transfer	65.00 kN/m

Lagging Design

Plate Type	Timber Lagging
Arching Factor	0.6
Allowed Stress	
Bending, Fb	15.00 MPa
Shear Parallel to Grain, Fv	2.50 MPa
Reference Modulus of El, E	1.5 GPa
Lumber is Southern Pine	No
Lumber Grade	#2

Results

Anchored or Braced Results		
Unfactored Earth Pressure	**x, m**	**w, kN/m**
	0.00	8.00
	1.67	117.00
	7.50	117.00
	10.00	8.00

Level	Depth, m	Tension (T) or Compression (C), kN	Unbonded Tieback Length, m	Test Load, kN
A	2.50	451.1 (T)	6.3	600.0
			2 #10 bars Grade 150	
B	6.25	444.6 (T)	4.6	591.3
			2 #10 bars Grade 150	

Static Check

Sum of Reactions	-942.9 kN
Sum of Loads	942.9 kN
Lateral Torsional Buckling Check, Mn/Omega	584 kN-m
Axially-Loaded Member Check, Pn/Omega	2699 kN
Combined Forces Utilization	38 %

Design

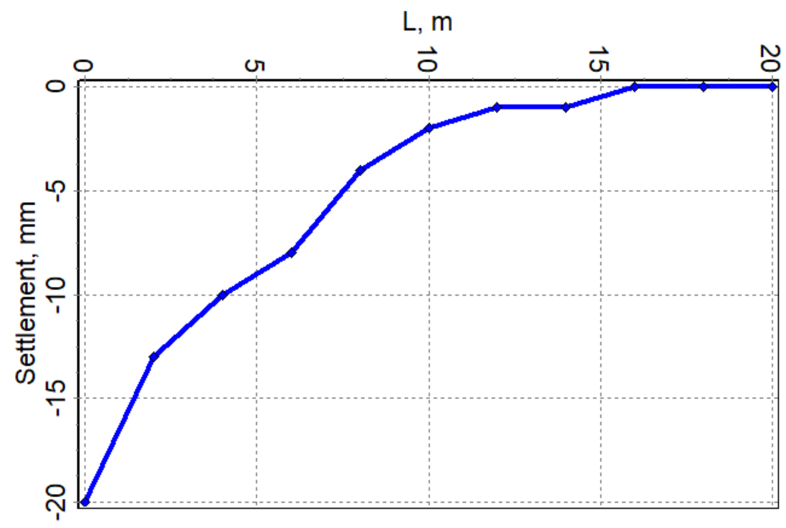
Max. Shear	-234.1 kN @ 2.50 m	
Max. Moment	189.0 kN-m @ 2.50 m	
Max. Deflection	-3.75 mm @ 0.00 m	
	Required	Provided
Aw (Adequate for Shear)	1696.40	56.79 mm^2
Zx (Adequate for Bending)	914.90	2870.00 cm^3
Utilized Ix (Adequate for Deflection)		30 %
Slenderness Ratio, kL/r		59

Bond Length	
Soil/Bedrock Type	User Defined
Level A Bond Length	9.23 m
Level B Bond Length	9.10 m

Loads Applied to the Pile	
Allowable Axial Load	480.05 kN

Lagging Design		
Soil Pressure		11.3 kPa
Max Bending Moment		5.063 kN-m/m
Shear		10.7 kN/m
	Adjusted	Actual
Bending Stress	14.9	1.6 MPa
Shear Parallel to Grain	2.5	0.1 MPa
Deflection		5.53 mm
Wood Lagging Size		150mm timber

Surface Settlement



Guardrail Design

Guardrail Design	
Shape	L89X89X12.7
Length of the Guardrail	1.070 m
Lateral Bracing	1.070 m
Max. Spacing of Guardrail	2.500 m
Dist. Load in Any Dir.	0.70 kN/m
Point Load in Any Dir.	0.89 kN
ASD Safety Factor of Comp.	1.67
ASD Safety Factor of Bend.	1.67

Guardrail Design Results	Allowable	Applied
Compression in the Post	180.02	1.75 kN
Yield Mom. Axis of Bending		0.009 kN-m
Elastic lat-tors. Buckling Mom.		0.012 kN-m
Bending moment in the post	5.417	1.873 kN-m
Welding Stress, y-axis	217.15	73.55 MPa
Welding Stress, x-axis	208.95	121.71 MPa

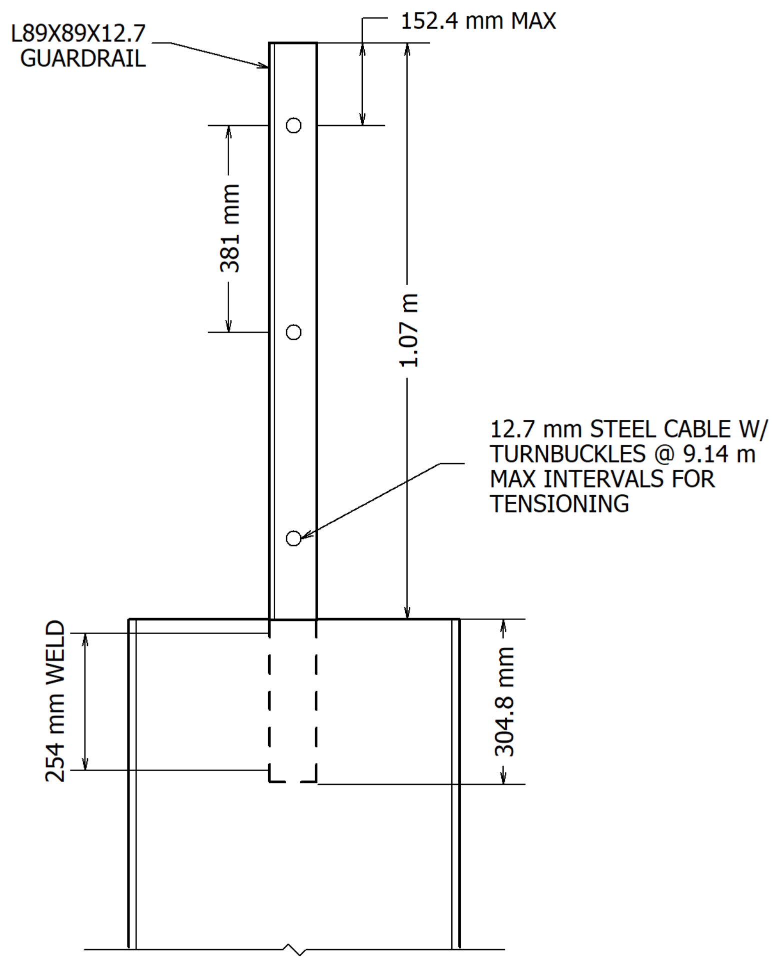
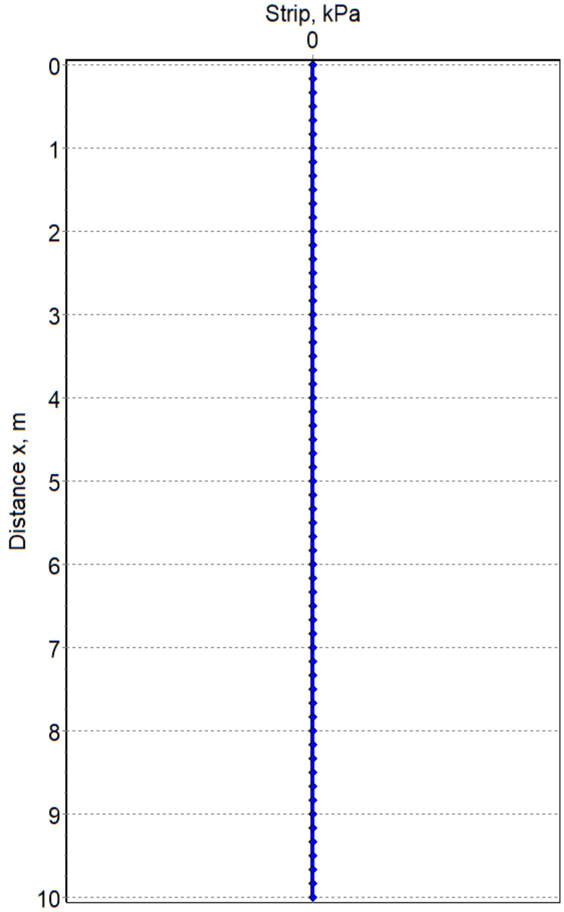
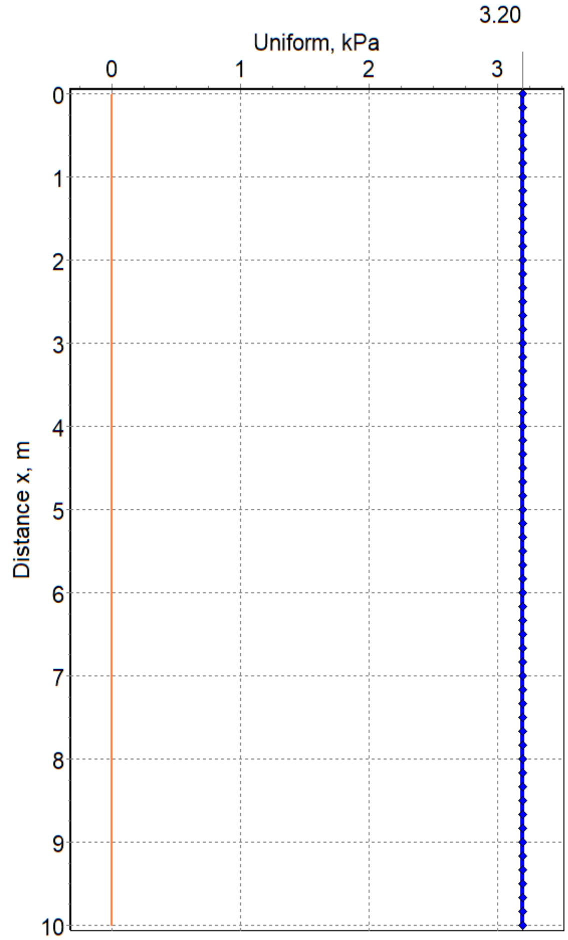


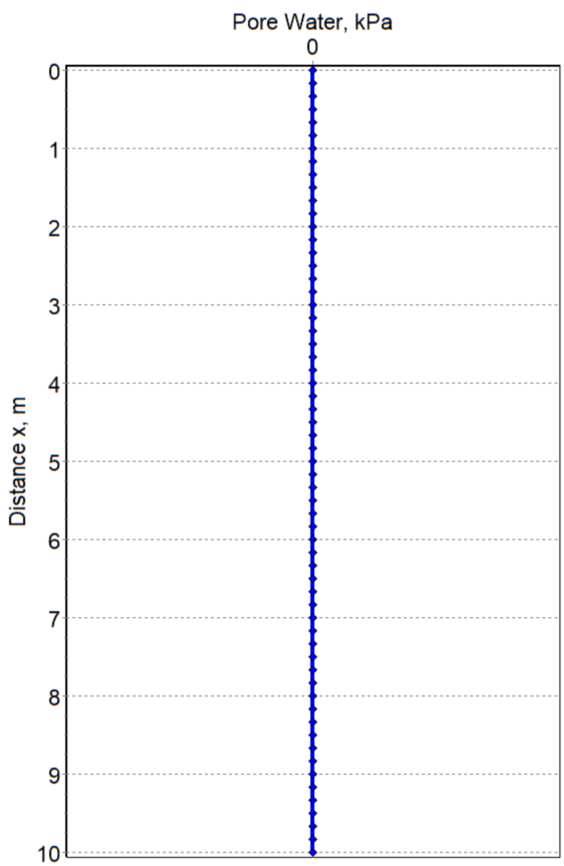
Table of Test Results

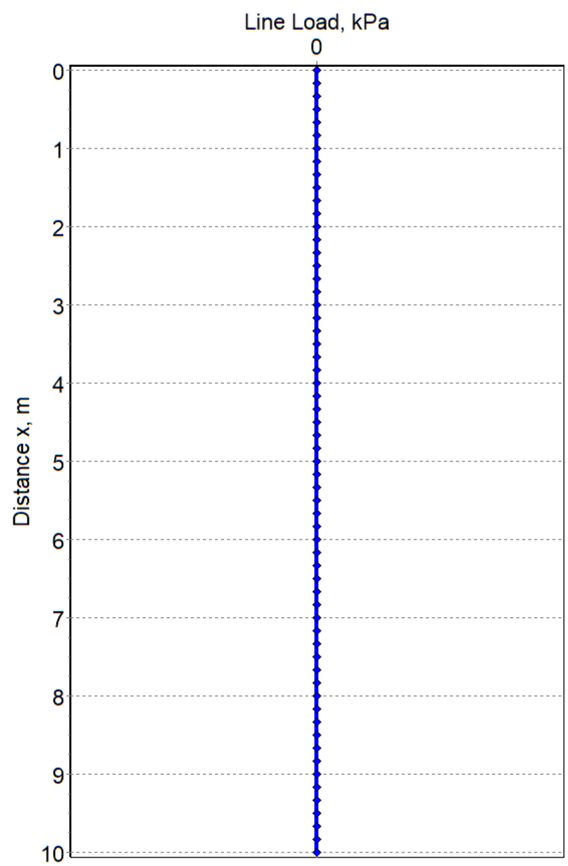
#	Depth, m	Strip Load, kPa	Unif. Surch, kPa	Pore Water, kPa	App. Earth, kPa	Line Load, kPa	Point Load, kPa	Seism. Load, kPa	Toal per m, kPa/m	Toal per Pile, kN/m
0	0.00	0.00	3.20	0.00	0.00	0.00	0.00	0.00	3.20	8.00
1	0.17	0.00	3.20	0.00	4.35	0.00	0.00	0.00	7.55	18.88
2	0.33	0.00	3.20	0.00	8.70	0.00	0.00	0.00	11.90	29.76
3	0.50	0.00	3.20	0.00	13.05	0.00	0.00	0.00	16.25	40.63
4	0.67	0.00	3.20	0.00	17.41	0.00	0.00	0.00	20.61	51.51
5	0.83	0.00	3.20	0.00	21.76	0.00	0.00	0.00	24.96	62.39
6	1.00	0.00	3.20	0.00	26.11	0.00	0.00	0.00	29.31	73.27
7	1.17	0.00	3.20	0.00	30.46	0.00	0.00	0.00	33.66	84.15
8	1.33	0.00	3.20	0.00	34.81	0.00	0.00	0.00	38.01	95.03
9	1.50	0.00	3.20	0.00	39.16	0.00	0.00	0.00	42.36	105.90
10	1.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
11	1.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
12	2.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
13	2.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
14	2.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
15	2.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
16	2.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
17	2.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
18	3.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
19	3.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
20	3.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
21	3.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
22	3.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
23	3.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
24	4.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
25	4.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
26	4.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
27	4.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
28	4.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
29	4.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
30	5.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
31	5.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
32	5.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
33	5.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
34	5.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
35	5.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
36	6.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
37	6.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
38	6.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
39	6.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
40	6.67	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
41	6.83	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
42	7.00	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
43	7.17	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
44	7.33	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
45	7.50	0.00	3.20	0.00	43.60	0.00	0.00	0.00	46.80	117.00
46	7.67	0.00	3.20	0.00	40.69	0.00	0.00	0.00	43.89	109.73
47	7.83	0.00	3.20	0.00	37.79	0.00	0.00	0.00	40.99	102.47
48	8.00	0.00	3.20	0.00	34.88	0.00	0.00	0.00	38.08	95.20
49	8.17	0.00	3.20	0.00	31.97	0.00	0.00	0.00	35.17	87.93
50	8.33	0.00	3.20	0.00	29.07	0.00	0.00	0.00	32.27	80.67
51	8.50	0.00	3.20	0.00	26.16	0.00	0.00	0.00	29.36	73.40
52	8.67	0.00	3.20	0.00	23.25	0.00	0.00	0.00	26.45	66.13
53	8.83	0.00	3.20	0.00	20.35	0.00	0.00	0.00	23.55	58.87
54	9.00	0.00	3.20	0.00	17.44	0.00	0.00	0.00	20.64	51.60
55	9.17	0.00	3.20	0.00	14.53	0.00	0.00	0.00	17.73	44.33
56	9.33	0.00	3.20	0.00	11.63	0.00	0.00	0.00	14.83	37.07
57	9.50	0.00	3.20	0.00	8.72	0.00	0.00	0.00	11.92	29.80
58	9.67	0.00	3.20	0.00	5.81	0.00	0.00	0.00	9.01	22.53
59	9.83	0.00	3.20	0.00	2.91	0.00	0.00	0.00	6.11	15.27
60	10.00	0.00	3.20	0.00	0.00	0.00	0.00	0.00	3.20	8.00

Charts

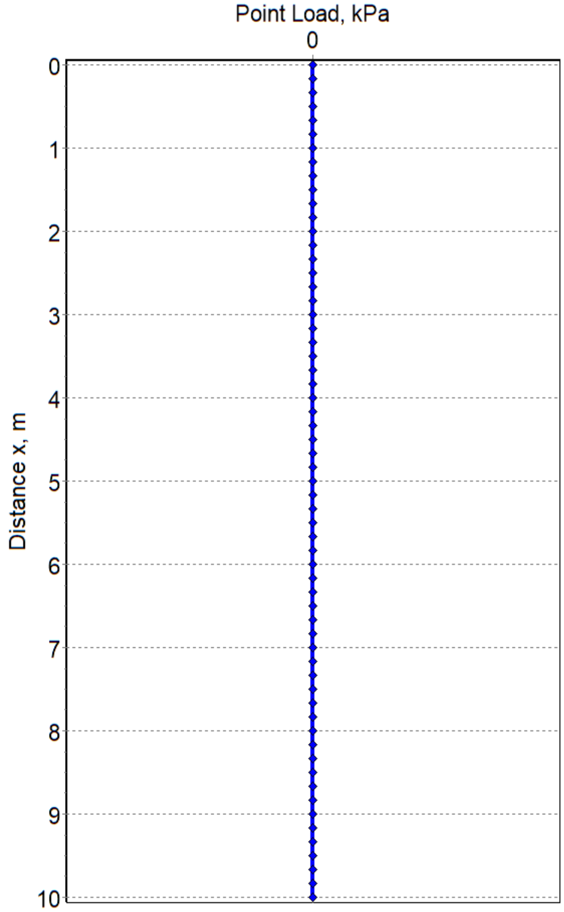

Strip, kPa
0
Distance x, m
0
1
2
3
4
5
6
7
8
9
10


3.20
Uniform, kPa
0
1
2
3
Distance x, m
0
1
2
3
4
5
6
7
8
9
10

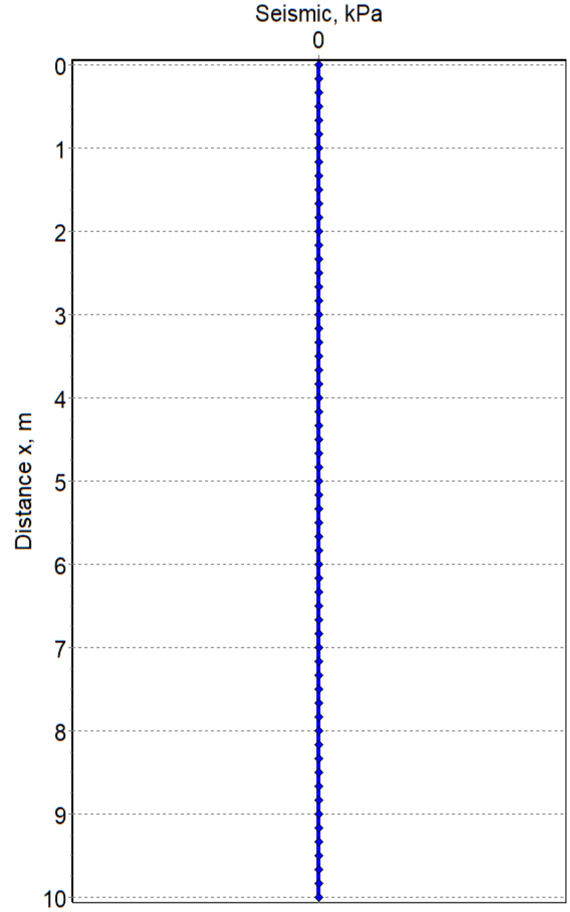

Pore Water, kPa
0
Distance x, m
0
1
2
3
4
5
6
7
8
9
10


Line Load, kPa
0
Distance x, m
0
1
2
3
4
5
6
7
8
9
10

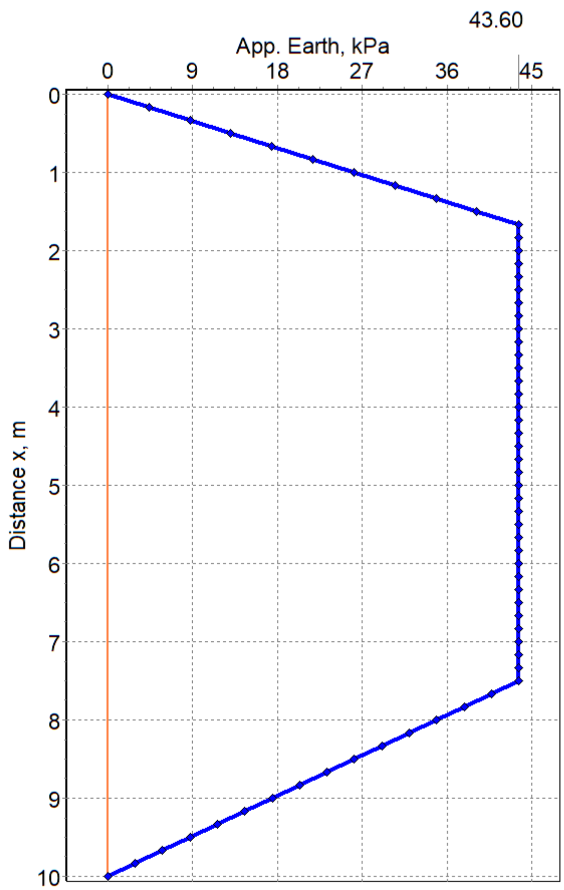
Charts



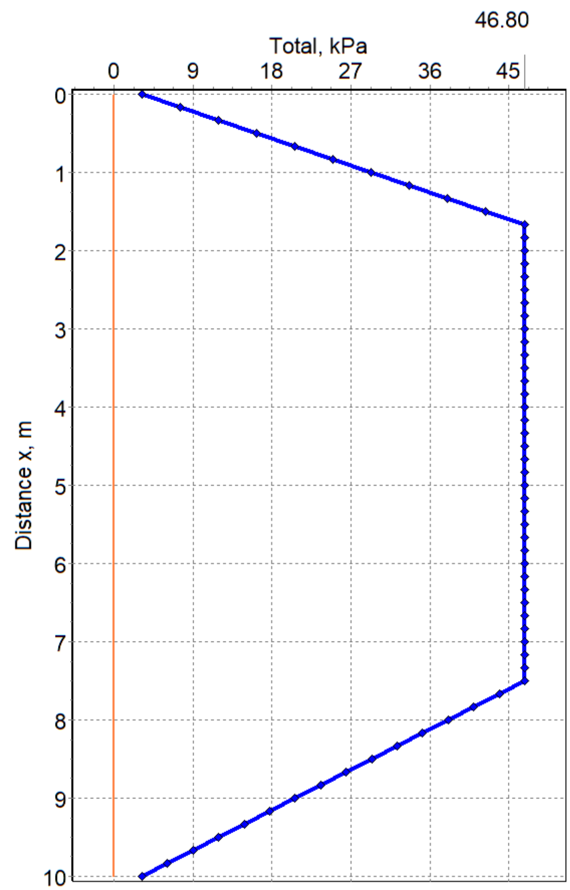
Point Load, kPa
0
Distance x, m
0
1
2
3
4
5
6
7
8
9
10



Seismic, kPa
0
Distance x, m
0
1
2
3
4
5
6
7
8
9
10

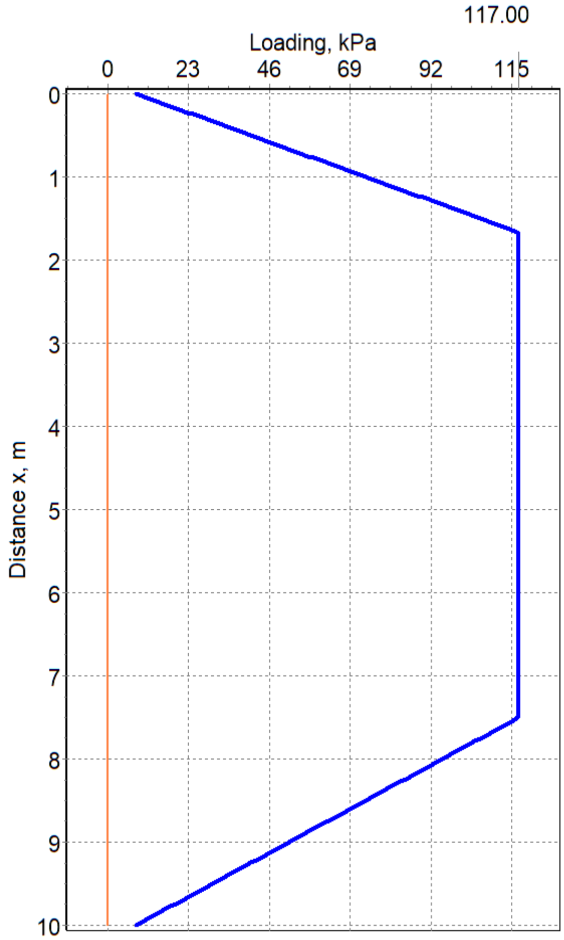


43.60
App. Earth, kPa
0
9
18
27
36
45
Distance x, m
0
1
2
3
4
5
6
7
8
9
10

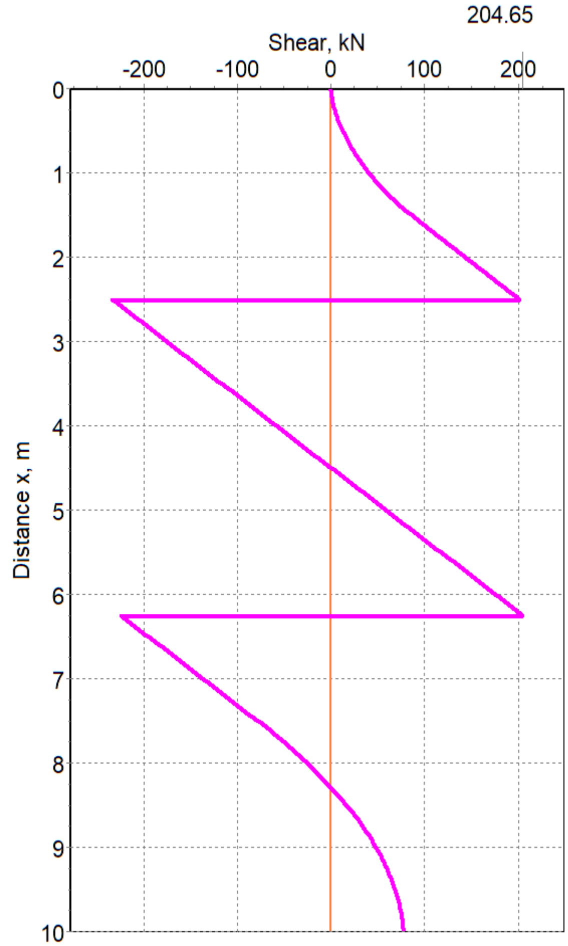


46.80
Total, kPa
0
9
18
27
36
45
Distance x, m
0
1
2
3
4
5
6
7
8
9
10

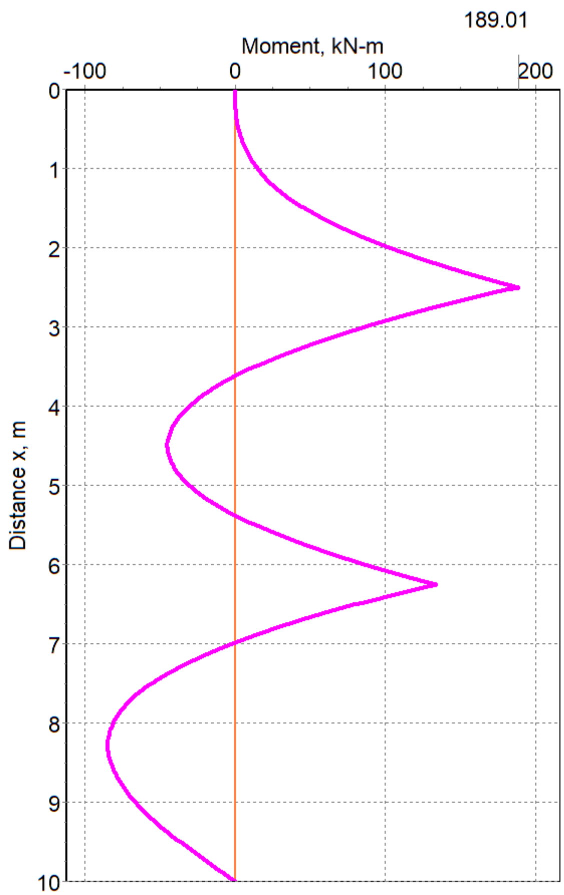
Charts



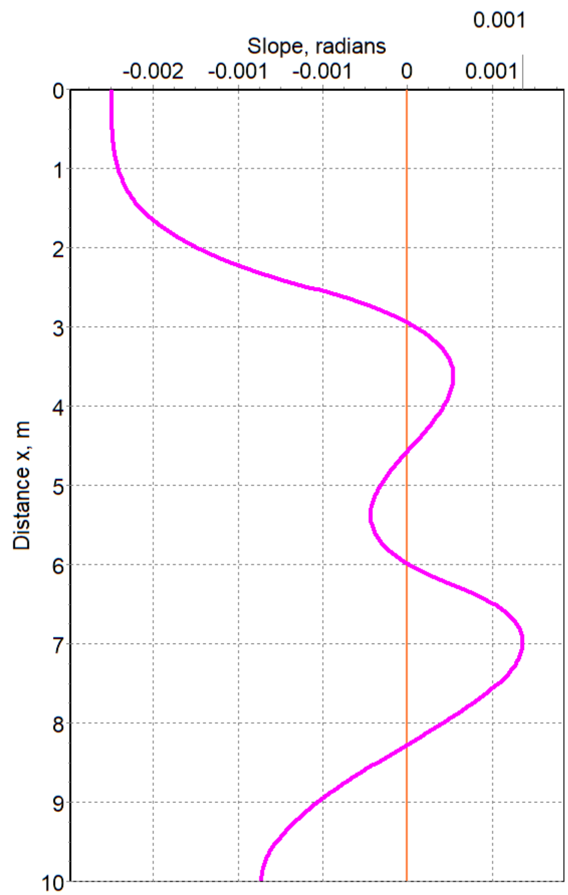
117.00
Loading, kPa
0
23
46
69
92
115
Distance x, m
0
1
2
3
4
5
6
7
8
9
10



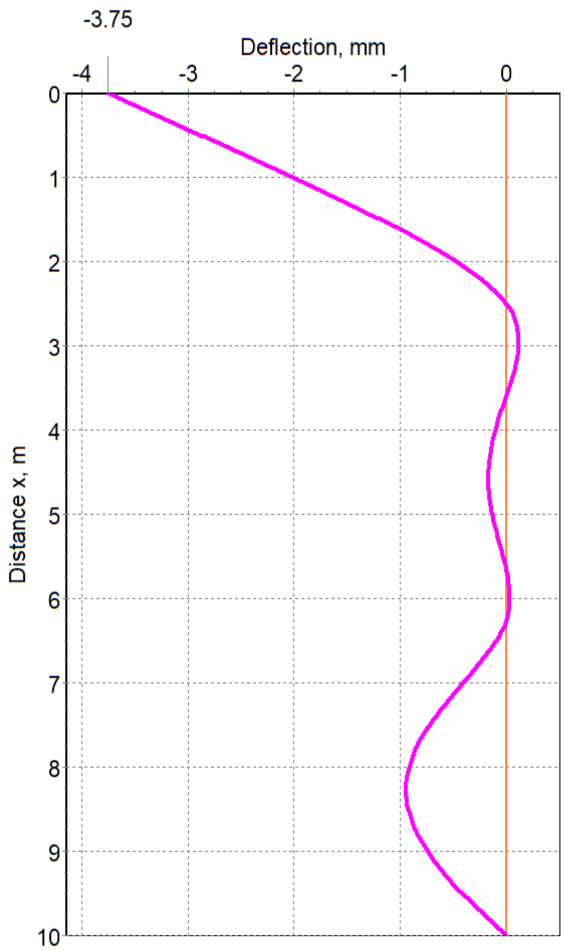
204.65
Shear, kN
-200
-100
0
100
200
Distance x, m
0
1
2
3
4
5
6
7
8
9
10



189.01
Moment, kN-m
-100
0
100
200
Distance x, m
0
1
2
3
4
5
6
7
8
9
10



0.001
Slope, radians
-0.002
-0.001
-0.001
0
0.001
Distance x, m
0
1
2
3
4
5
6
7
8
9
10



NOTES:
1. The Shoring Engineer shall check that the earth pressures entered in the software match the soils report & site soils parameters.
2. The Shoring Engineer shall check (a) Cantilever Shoring Height assuming a 2 ft over-excavation for top (A) anchor. You can use CANTILEVER SHORING Software by SoilStructure.com
3. The Shoring Engineer shall check all anchor stability before the final design. For example, a 3 level anchor should be checked for cantilever position of the first anchor (assuming 2 ft (0.6 m) over-excavation), the second anchor (also a 2 ft (0.6 m) over-excavation) and then the final third anchor installation.
4. It is advised that the Combined Forces Utilization be limited to 90% maximum due to the accuracy of the Apparent Earth Pressures assumed in this Software.
5. It is also advised that the vertical height between anchors or braces be limited to 16.4 ft (5 m) maximum. Also advised to limit height from lowest anchor to base of excavation to 16.4 ft (5 m) maximum.
6. We have tried our best to check the stability of excavations using generally accepted procedures. However, the final design of the shoring is the responsibility of the shoring engineer (licensed professional engineer).
7. The Apparent Earth Pressure Diagrams have been established on the basis of a rather limited number of cuts from about 26 ft to 62 ft (8 m to 19 m) in Height. Therefore, the user should use it with caution on much deeper cuts.
8. The Apparent Earth Pressure Diagrams do not bear any resemblance to the distribution of earth pressure against the shoring piles. It is merely an approximate method of calculating strut or tieback loads.
9. In no event will SOILSTRUCTURE SOFTWARE, INC., its employees, consultants or owners be liable to anyone for any unfavorable conditions occurring from the use of this Software. The Licensee acknowledges and accepts all of these statements when choosing to use this Software.

References:
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22. SoilStructure Software: Anchored or Braced Shoring v1.0.0

APPENDIX A

DESIGN EXAMPLE 1

ANCHORED WALL SUPPORTED EXCAVATION

WALL REQUIREMENTS

A 10-m high permanent anchored soldier beam and timber lagging wall is to be constructed as part of a depressed roadway project. When construction of the wall is completed, a 7.3-m wide entrance ramp will be constructed 3 m behind the wall. The wall is to be constructed in a medium dense silty sand profile as shown in figure A-1. No existing structures or underground utilities are located within 20 m of the top of the proposed wall location. A cast-in-place (CIP) concrete facing is to be used.

SUBSURFACE CHARACTERIZATION

Geotechnical borings drilled in front of, alongside, and behind the proposed wall alignment indicate that the subsurface stratigraphy is relatively uniform. The profile shown in figure A-1 is considered to be representative of the soil stratigraphy along the alignment of the wall. Soil properties for design are shown for individual layers in figure A-1. Groundwater was not encountered in any of the borings and it is concluded that groundwater levels at the site are below elevation 87 m MSL. Agressivity testing indicates that the site soils have a resistivity above 5,000 ohm-cm, a pH between 6.2 and 6.8, and no sulfides are present. The soils are therefore considered to be non-aggressive.

Written By : DGP Date: 99/01/28 Reviewed by: PJS Date: 99/01/30

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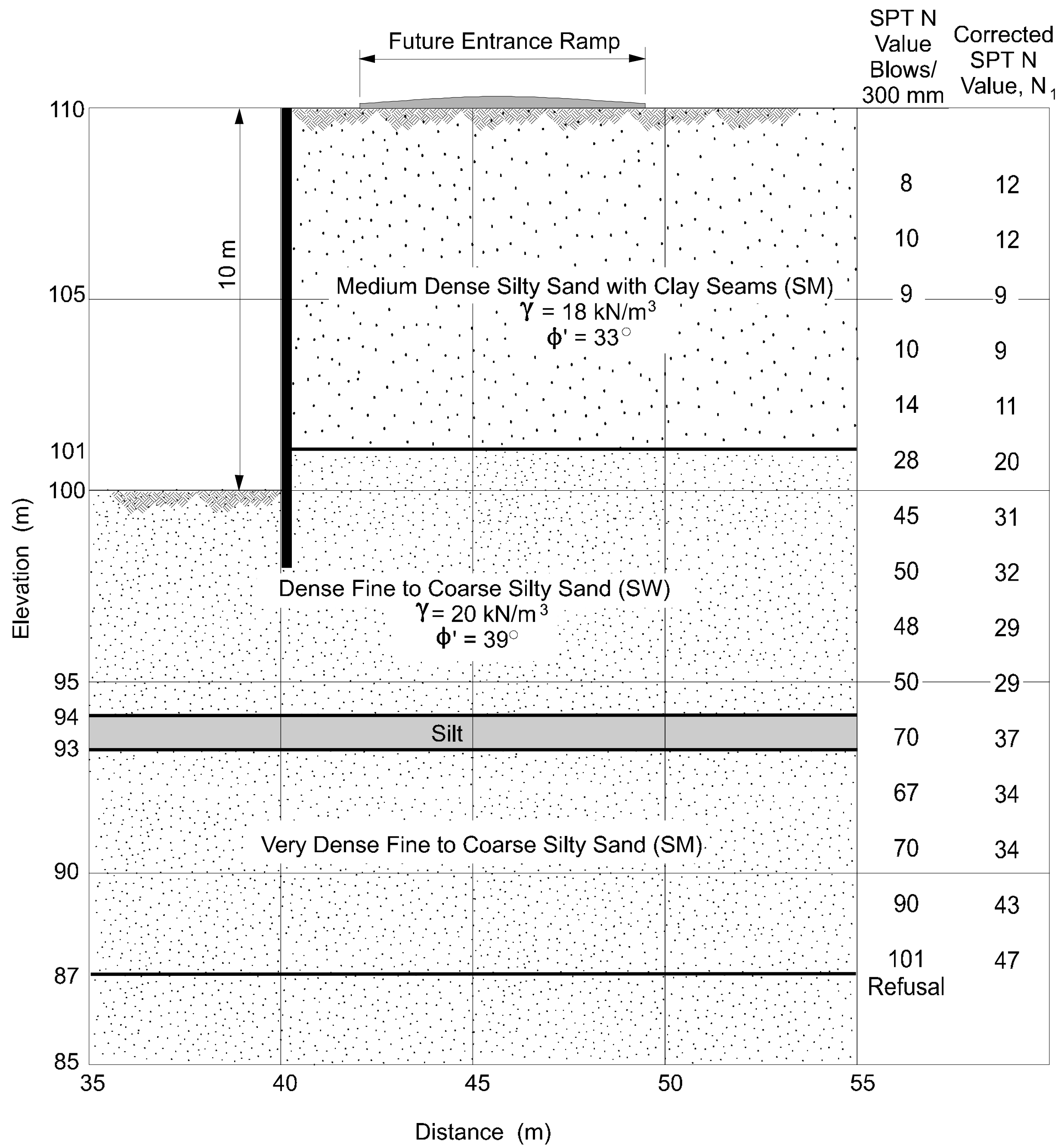

Figure A-1. Subsurface stratigraphy and design cross section.

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LOAD TYPES ACTING ON WALL

Wall loads are estimated based on AASHTO (1996) recommendations for a Group I Load Combination using the Service Load design method, as follows:

$$\text{Group I Load} = D \quad (L \quad I) \quad CF \quad E \quad B \quad SF$$

where D is dead load; L is live load; I is live impact load; CF is centrifugal force; E is lateral earth pressure; B is buoyancy; and SF is stream flow pressure.

Wall loads are approximated as follows:

1. D: The dead load acting at the base of each soldier beam was approximated as the sum of the weight of the soldier beam, concrete backfill (if used), timber lagging, and CIP concrete facing.

2. L and I: For conditions where traffic lanes are located within half the wall height behind the wall, AASHTO (1996) recommends that a surcharge pressure equivalent to 0.6 m of soil above the wall be included in the calculation of lateral earth pressure against the wall.

3. E: The lateral earth pressure was approximated using the trapezoidal apparent earth pressure diagram for sands as shown in figure 24.

4. B, CF, and SF: These load types are not expected to be present during the construction or service life of the wall.

LOCATION OF CRITICAL FAILURE SURFACE

The critical failure surface may be assumed to intersect the corner of the wall and exit at the ground surface and be sloped at 45°+ ’/2 from the horizontal where ’ is equal to the effective stress friction angle of the soil behind the wall. Alternatively, a slope stability analysis may be performed to evaluate the location of the critical potential failure surface. When using a slope stability analysis program, a uniform lateral surcharge load is applied to the wall face to model the restraint provided by the anchors. This load is increased until a factor of safety equal to one (FS = 1.0) is achieved. Input parameters for a slope stability analysis, including geometry of the wall, subsurface stratigraphy, and soil properties, are shown in figure A-1.

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APPARENT EARTH PRESSURE DIAGRAM

The apparent earth pressure diagram for a two-tier anchored wall constructed in predominately cohesionless soils is shown in figure A-2 where T_{H1} is the horizontal anchor load per meter of wall for the upper anchor; T_{H2} is the horizontal anchor load per meter of wall for the lower anchor; and p_e is the maximum ordinate of the apparent earth pressure diagram. It was assumed that the upper anchor is located 2.5 m below the top of the wall and the lower anchor is located 6.25 m below the top of the wall.

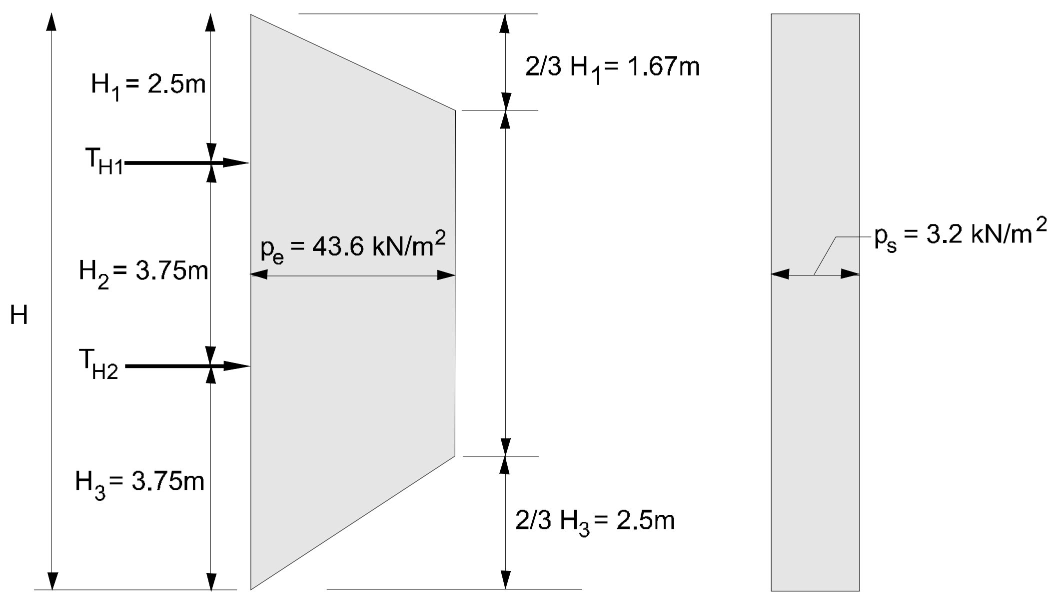

Figure A-2. Apparent earth pressure diagram and surcharge pressure diagram.

The majority of the excavation for the wall will penetrate through the upper soil layer, i.e., the medium dense silty sand layer. To develop the apparent earth pressure diagram, a unit weight of 18 kN/m^3 and effective stress friction angle of 33 degrees were used.

1. The value of p_e was calculated based on figure 24:

$$p_e \quad \frac{0.65\left(\tan^2\left(45 \quad \frac{\;}{2}\right)\right) \quad H^2}{H \quad \frac{H_1}{3} \quad \frac{H_3}{3}}$$

$$\frac{0.65\left(\tan^{2}\left(45 \quad \frac{33}{2}\right)\right) 18\ \text{kN/m}^{3} \quad 10\ \text{m}^{\ 2}}{10\ \text{m} \quad \frac{2.5\ \text{m}}{3} \quad \frac{3.75\ \text{m}}{3}} \quad 43.6\ \text{kN/m}^{2}$$

LATERAL EARTH PRESSURE DUE TO TRAFFIC SURCHARGE

The traffic surcharge pressure (q_s) applied at the ground surface is assumed to equal 0.6 m x 18 kN/m^3 = 11 kN/m^2. The corresponding lateral pressure (p_s) is assumed to act uniformly over the entire wall height and is calculated as follows:

$$p_s \quad K_A\ q_s$$

$$\tan^{2}\left(45 \quad \frac{33}{2}\right) 11\ \text{kN/m}^{2} \quad 3.2\ \text{kN/m}^{2}$$

The earth pressure diagram due to the traffic surcharge is shown in figure A-2.

HORIZONTAL ANCHOR LOADS, MAXIMUM WALL BENDING MOMENT, AND REACTION FORCE TO BE RESISTED BY THE SUBGRADE

The tributary area method (figure 34) was used to calculate the horizontal anchor loads, T_{H1} and T_{H2}, the maximum wall bending moment, M_{max}, and the reaction force to be resisted by the subgrade, R.

1. The horizontal anchor loads were calculated using the tributary area method, as follows:

$$T_{H1} \quad \left(\frac{2}{3} H_1 \quad \frac{H_2}{2}\right) p_e \quad \left(H_1 \quad \frac{H_2}{2}\right) p_s$$

$$\left(\frac{2}{3} 2.5\ \text{m} \quad \frac{3.75\ \text{m}}{2}\right) 43.6\ \text{kN/m}^{2} \quad \left(2.5\ \text{m} \quad \frac{3.75\ \text{m}}{2}\right) 3.2\ \text{kN/m}^{2}$$

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$$168 \text{ kN/m (upper anchor)}$$ 180.4 kN/m

$$T_{H2} \quad \left(\frac{H_2}{2} \quad \frac{23}{48} H_3\right) p_e \quad \left(\frac{H_2}{2} \quad \frac{H_3}{2}\right) p_s$$

$$\left(\frac{3.75 \text{ m}}{2} \quad \frac{23}{48} 3.75 \text{ m}\right) 43.6 \text{ kN/m}^2 \quad \left(\frac{3.75 \text{ m}}{2} \quad \frac{3.75 \text{ m}}{2}\right) 3.2 \text{ kN/m}^2$$

$$172 \text{ kN/m (lower anchor)}$$ 177.8 kN/m

2. Wall bending moments were calculated for the upper anchor level (M_1), between the upper and lower anchor level (M_2), and between the lower anchor level and the base of the excavation (M_3) using the tributary area method. The wall bending moment used for design, M_{max}, is the largest of M_1, M_2, and M_3.

   The value of M_1 was calculated as follows:

$$M_1 \quad \frac{13}{54} H_1^{\ 2} \ p_e \quad p_s \ H_1 \ \frac{H_1}{2}$$

$$\frac{13}{54} \ 2.5 \text{ m}^{\ 2} \ 43.6 \text{ kN/m}^2 \quad 3.2 \text{ kN/m}^2 \ (2.5 \text{ m}) \left(\frac{2.5 \text{ m}}{2}\right)$$

$$76 \text{ kN - m/m}$$

   The maximum bending moment below the upper anchor was calculated assuming $H_2 = H_3 = 3.75$ m:

$$M_{2,3} \quad \frac{1}{10} \ H_{2,3}^{\ \ 2} \ p_e \quad p_s$$

$$\frac{1}{10} \ 3.75 \text{ m}^{\ 2} \ 43.6 \text{ kN/m}^2 \quad 3.2 \text{ kN/m}^2$$

$$66 \text{ kN} \quad \text{m/m}$$

   The wall bending moment used for design is M_{max}=76 kN-m/m. 75.6 kN/m

3. The reaction force to be resisted by the subgrade is assumed to act at the base of the excavation and was calculated using the tributary area method as follows:

$$R \quad \left(\frac{3H_3}{16}\right)p_e \quad \left(\frac{H_3}{2}\right)p_s$$

$$\left(\frac{3(3.75\,\mathrm{m})}{16}\right)43.6\,\mathrm{kN/m^2} \quad \left(\frac{3.75\,\mathrm{m}}{2}\right)3.2\,\mathrm{kN/m^2} \quad 37\,\mathrm{kN/m}$$

INITIAL TRIAL DESIGN ASSUMPTIONS

Initial designs were developed for a soldier beam and lagging wall with bar anchors and for a soldier beam and lagging wall with strand anchors. The inclination of all anchors was assumed to be 15º and the soldier beam center-to-center spacing was assumed to be 2.5 m. A cross section view of the initial design for the wall including the bar anchors is shown in figure A-3. The wall design including the strand anchors is the same as that shown in figure A-3, except that the minimum unbonded length of the lowermost anchor is greater than that for the bar anchor configuration. A discussion of the unbonded and bond lengths for the strand and bar designs is provided subsequently.

ANCHOR DESIGN LOAD

1. Upper anchor: The anchor design load (DL_1) was calculated as follows:

$$DL_1 \quad \frac{T_{H1} \quad 2.5\ \mathrm{m}}{\cos 15} \quad 168\ \mathrm{kN/m} \quad \frac{2.5\ \mathrm{m}}{\cos 15} \quad 435\ \mathrm{kN}$$

451.1 kN/m

2. Lower anchor: The anchor design load (DL_2) was calculated as follows:

$$DL_2 \quad \frac{T_{H2} \quad 2.5\ \mathrm{m}}{\cos 15} \quad 172\ \mathrm{kN/m} \quad \frac{2.5\ \mathrm{m}}{\cos 15} \quad 445\ \mathrm{kN}$$

444.6 kN/m

The maximum calculated anchor design load is 445 kN.

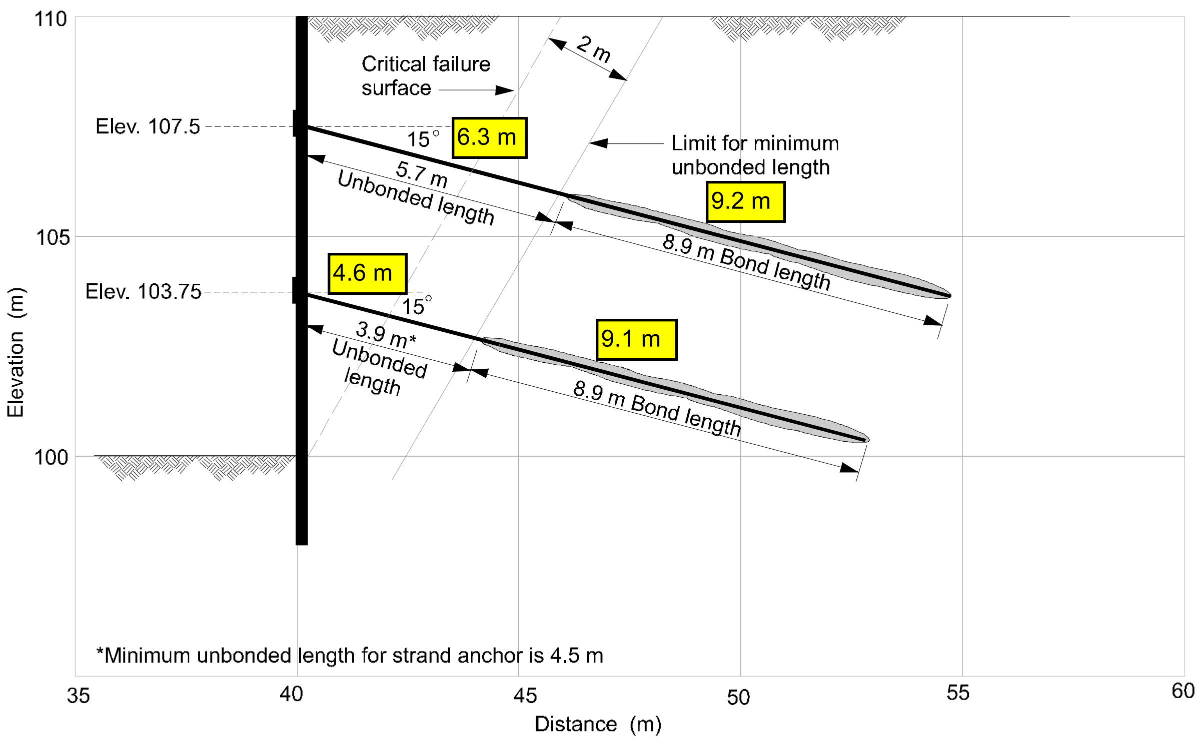

Figure A-3. Location of unbonded and bond lengths for ground anchors.

DESIGN OF THE UNBONDED LENGTH

For the design that includes bar anchors, the minimum unbonded length was selected to be the greater of either 3 m or the distance from the wall to a location 2 m beyond the critical failure surface. For the design that includes strand anchors, the minimum unbonded length was selected to be the greater of either 4.5 m or the distance from the wall to a location 2 m beyond the critical failure surface. These minimum values for the unbonded length are discussed in section 5.3.7.

ANCHOR CAPACITY

The anchor bond zones will be formed in the medium dense silty sand layer (Elevation 101 to 110 m MSL) and the dense silty sand layer (Elevation 94 to 101 m MSL). Assuming that the load transfer rate is controlled by the medium dense silty sand layer, a load transfer rate of 100 kN/m was selected (see table 6).

The design load with a factor of safety of 2.0 should be able to be achieved with a typical soil anchor bond length of 12 m, assuming a small diameter low pressure grouted anchor. For a length of 12 m the bond strength is [(100kN/m)/2.0]x12 m = 600 kN. The allowable anchor capacity of 600 kN is larger than the maximum design load of 445 kN. This implies that the design load can be attained at this site for the assumed anchor spacings and inclination. Right of way estimates can be made based on the bond length required for mobilization of the design load, as follows:

$$\text{Maximum Bond Length} \quad \frac{445\,\text{kN} \quad 2.0}{100\,\text{kN/m}} \quad 8.9\,\text{m}$$

EXTERNAL STABILITY

The external stability of the anchored wall was evaluated using a slope stability analysis program. A target factor of safety of 1.3 was selected. Wall and subsurface input parameter values used are the same as those used for the stability analysis to evaluate the anchor unbonded lengths. The location of the end of each anchor bond zone is shown in figure A-3. The analysis was performed for the anchored wall including the bar anchors. The minimum calculated factors of safety for potential failure slip surfaces located behind the upper and lower anchors were calculated to be 2.5 and 2.6, respectively. Based on these calculations the anchored wall is considered stable with respect to external stability.

SELECTION OF TENDON

Although the site soils are classified as nonaggresive, the consequence of failure and subsequent closure of the roadway is considered serious. Therefore, a Class I (double protection) encapsulated tendon is selected. Dimensions are calculated for both strand and bar tendons assuming a maximum test load of 1.33 DL.

A 32-mm diameter, Grade 150 prestressing bar may be selected, based on an allowable tensile capacity of 60 percent of the specified minimum tensile strength (SMTS). The allowable tensile capacity is 501 kN (see table 9) which exceeds the calculated maximum design load of 445 kN. The minimum estimated trumpet opening is 95 mm for a Class I corrosion protection system (see table 11).

A 3 strand, Grade 270 strand anchor may also be selected. The allowable tensile capacity of the tendon is 469 kN (see table 10). The minimum estimated trumpet opening is 150 mm for a Class I corrosion protection system.

SOLDIER BEAM SELECTION

The required section modulus, S_{req}, of each soldier beam is calculated as follows:

$$S_{req} \quad \frac{M_{max}}{F_b}$$

where F_b is the allowable bending stress of the steel, which is equal to 55 percent of the yield stress for permanent applications. Yield stresses for Grade 36 and Grade 50 steels are 248 MPa (36 ksi) and 345 MPa (50 ksi), respectively. Using M_{max} from previous calculations, the maximum soldier beam moment is equal to (76 kN-m/m x 2.5 m) = 190 kN-m.

1. Grade 36 steel: $S_{req} \quad \frac{190\ \text{kN - m}}{0.55 \quad 248\ \text{MPa}} \quad 0.001393\ \text{m}^3$

   Two C15 x 40 channel sections provides a section modulus of 0.001524 m^3.

2. Grade 50 steel: $S_{req} \quad \frac{190\ \text{kN} \quad \text{m}}{0.55 \quad 345\ \text{MPa}} \quad 0.001001\ \text{m}^3$

   Two MC12 x 31 channel sections provide a section modulus of 0.001109 m^3.

It was assumed that a pair of MC12 x 31 Grade 50 channel sections would be used for each soldier beam. It was also assumed that each hole would be backfilled from the bottom of the hole to the elevation of the excavation base with structural concrete such that the full diameter of the shaft may be considered for axial and lateral load capacity evaluations. The minimum required diameter of the shaft was calculated based on the diagonal distance between the tips of the flanges. For a MC 12x31 section, the flange width and beam depth are 93 mm and 305 mm, respectively. Assuming a 150 mm open space between channels, b_{os}, for the tendon, the minimum required diameter is:

$$\text{minimum required diameter} \quad \sqrt{(2 \text{ x flange width} \quad b_{os})^2 \quad (\text{beam depth})^2}$$

$$\text{minimum required diameter} \quad \sqrt{(2 \text{ x } 93\,\text{mm} \quad 150\,\text{mm})^2 \quad (305\,\text{mm})^2}$$

$$\text{minimum required diameter} \quad 454\,\text{mm}$$

A shaft diameter of 610 mm will be used.

DESIGN OF TIMBER LAGGING

For a soldier beam consisting of two channel sections, the required length of timber lagging may be calculated as the center-to-center spacing of the soldier beams minus the space between the channel sections of a soldier beam. This can be written as follows:

$$\text{required length of timber lagging} = s - b_{os}$$

$$\text{required length of timber lagging} = 2.5\ \text{m} - 0.15\ \text{m}$$

$$\text{required length of timber lagging} = 2.35\ \text{m}$$

A timber lagging thickness of 75 mm was selected based on table 12.

LATERAL CAPACITY OF SOLDIER BEAM TOE

The soldier beam must be sufficiently embedded to develop passive resistance to carry the lateral load resulting from the reaction force to be resisted by the subgrade, R, and the active pressure acting over the soldier beam width, b, (i.e., 0.6 m) along the embedded soldier beam length. A factor of safety of 1.5 is required. The lateral load, R_{Load}, is calculated as follows:

$$R_{Load} \quad Rs \quad \frac{1}{2} DK_{A} \quad 2H \quad D \quad b$$

$$R_{Load} \quad 37\ \text{kN/m}(2.5\ \text{m}) \quad \frac{1}{2} D \tan^{2}\left(45 \quad \frac{39}{2}\right) 2\ (10\ \text{m}) \quad D \quad 0.6\ \text{m}$$

The ultimate passive resistance is assumed to be the minimum ultimate passive resistance calculated from equations B-2, B-4, B-5, and B-6 (see appendix B). The factor of safety was calculated as the ratio of the ultimate passive force, F_p to R_{Load}. Calculations were performed using the spreadsheet presented in figure A-4. Based on these calculations, a soldier beam embedment depth of 2.0 m is required to achieve a factor of safety that exceeds the target value of 1.5.

Unit weight of soil ( )	18	kN/m3
Height of soldier beam above final excavation level (H)	10	m
Drilled shaft diameter (b)	0.6	m
Soldier beam center to center spacing (s)	2.5	m
Clear spacing between drilled shafts (s_c)	1.9	m
soil friction angle ( )	39	degrees
=45+ /2	64.5	degrees
= (for dense sands)	39	degrees
Subgrade reaction force (R)	37	kN/m
at-rest earth pressure coefficient (K_o) = 1-sin	0.37	
active earth pressure coefficient (K_a) = $\tan^2$(45- /2)	0.23	
passive earth pressure coefficient (K_p) = $\tan^2$(45+ /2)	4.40	

Toe Depth (m)	Wedge Resistance (single pile) (Eq. B-2) (kN/m)	Wedge Resistance (intersecting wedges) (kN/m)	Flow Resistance (Eq. B-5) (kN/m)	Rankine Continuous (Eq. B-6) (kN/m)	Minimum Wang-Reese Passive Resistance (kN/m)	Total Passive Force (kN)	Total Active Force (kN)	Total Subgrade Reaction Force (kN)	Factor of Safety
0.0	0	0	0	0	0	0	0.0	92.5	0.0
0.5	60	60	490	99	60	15	12.6	92.5	0.1
1.0	194	169	980	198	169	73	25.8	92.5	0.6
1.5	401	289	1,470	297	289	187	39.6	92.5	1.4
2.0	**680**	**419**	**1,960**	**396**	**396**	**358**	**54.1**	**92.5**	**2.4**
2.5	1,034	559	2,449	494	494	581	69.1	92.5	3.6
3.0	1,460	710	2,939	593	593	853	84.8	92.5	4.8
3.5	1,959	870	3,429	692	692	1,174	101.0	92.5	6.1
4.0	2,532	1,041	3,919	791	791	1,545	117.9	92.5	7.3
4.5	3,178	1,222	4,409	890	890	1,965	135.4	92.5	8.6
5.0	3,897	1,414	4,899	989	989	2,435	153.6	92.5	9.9

Figure A-4. Embedment depth calculations (Wang-Reese method).

AXIAL CAPACITY OF SOLDIER BEAM

1. Calculate total axial load

The total axial load was calculated as the sum of the vertical anchor forces and weights of the soldier beam, concrete backfill, timber lagging, and CIP concrete facing. For the calculations, it was assumed that the embedment depth of the soldier beam was 2.5 m.

The vertical anchor forces were calculated using anchor design loads and inclinations as follows:

Vertical force of upper anchor: 435 kN x sin 15 = 113 kN
Vertical force of lower anchor: 445 kN x sin 15 = 115 kN

The weight of two MC12 x 31 Grade 50 channel sections with an assumed embedment depth of 2.5 m and a unit weight of 0.452 kN/m is calculated as follows:

$$\text{Weight of soldier beam} \quad 2 \text{ x } 0.452 \text{ kN/m x } 12.5 \text{ m} \quad 11 \text{ kN}$$

The drill hole size selected for a soldier beam fabricated from a pair of MC12 x 31 shapes is 0.6 m. The weight of concrete backfill for a drilled-in soldier beam for a 0.6 m diameter concrete section and a unit weight of 22.6 kN/m^3 was calculated as follows:

$$\text{Weight of concrete backfill} \quad 22.6 \text{ kN/m}^3 \quad \frac{0.6 \text{ m}^{\;2}}{4} \quad 12.5 \text{ m} \quad 80 \text{ kN}$$

This weight was reduced to account for the removal of the lean-mix concrete backfill during lagging installation. The area of concrete to be removed down to the front flange of the channel beams was calculated to be 0.055 m^2.

$$\text{Weight of removed concrete} = \quad 22.6 \text{ kN/m}^3 \quad 0.055 \text{ m}^2 \quad 10 \text{ m} \quad 12 \text{ kN}$$

The weight of timber lagging was calculated for 75-mm thick boards. The unit weight of timber lagging was assumed to be 8 kN/m^3.

$$\text{Weight of timber lagging} \quad 8 \text{ kN/m}^3 \quad 10 \text{ m} \quad 2.35 \text{ m} \quad 0.075 \text{ m} \quad 14 \text{ kN}$$

The weight of the CIP concrete facing is calculated for a 254-mm thick facing. The unit weight of reinforced concrete was assumed to be 23.6 kN/m^3.

$$\text{Weight of concrete facing} \quad 23.6 \text{ kN/m}^3 \quad 10 \text{ m} \quad 2.5 \text{ m} \quad 0.254 \text{ m} \quad 150 \text{ kN}$$

The total axial load was calculated as the sum of the above loads and is equal to 471 kN.

2. Calculate the required axial capacity

The required axial capacity of a drilled-in soldier beam was calculated using procedures described in section 5.6 for drilled-in soldier beams in cohesionless soil. The required axial capacity (Q_a) is calculated by applying a safety factor of 2.0 to the ultimate skin friction and a factor of safety of 2.5 to the ultimate end bearing such that:

$$Q_a \quad \frac{f_s A_s}{2.0} \quad \frac{q_t A_t}{2.5}$$

<u>End Bearing</u>

Using the SPT blowcount value at the approximate location of the bottom of the soldier beam (use N=45) and equation 30 results in:

$$Q_a \text{ (end bearing)} \quad \frac{q_t A_t}{2.5} \quad \left[57.5(45)\frac{}{4} \quad 0.6\text{m}^{\;2}\right]/2.5 \quad 293\text{kN}$$

<u>Side Resistance</u>

Using an assumed embedment depth, D, of 2.5 m and equation 29 results in:

$$Q_a \text{ (skin friction)} \quad \frac{f_s A_s}{2.0} \quad \frac{p_o A_s}{2.0}$$

$$1.5 \quad 0.42z^{0.34}; z \quad \frac{1}{2} \text{ H} \quad \text{D}$$

$$1.5 \quad 0.42\left(\frac{10\,\text{m} \quad 2.5\,\text{m}}{2}\right)^{0.34} \quad 0.72$$

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$$p_o \quad \frac{\;}{2} \; H \quad D$$

$$\frac{18\ \text{kN/m}^3}{2}(10\ \text{m} \quad 2.5\ \text{m}) \quad 112\ \text{kN/m}^2$$

$$Q_a\ (\text{skin friction}) \quad \left(0.72\ \underbrace{112\ \text{kN/m}^2}_{p_o} \quad \underbrace{2.5\ \text{m}\ \ 0.6\ \text{m}}_{A_s} \right) / 2.0 \quad 190\ \text{kN}$$

Total Axial Capacity

480.0 kN

for D = 2.5 m, Q_a = 293 kN + 190 kN = 483 kN > 471 kN (OK)

RESISTING THE UPPER ANCHOR TEST LOAD

The factor of safety against passive failure of the retained soil above the upper anchor level at the anchor test load is calculated as the ratio of the maximum passive resistance of the retained soil and the test load (see section 5.11.4). The test load is equal to 1.33 times the horizontal component of the design anchor load, i.e., (1.33*435 kN cos 15^o = 559 kN). The maximum passive resistance of the retained soil was calculated using the following equation:

$$F_p \quad 1.125 K_p \quad H_1{}^2 s$$

where K_p = 6.0 based on an effective stress friction angle of 33 for the upper sand layer and an assumed wall/soil interface friction angle equal to 0.5 ’ (Figure 17).

$$F_p \quad 1.125 \;\; 6.0 \quad 18\ \text{kN}/\text{m}^3 \quad 2.5\ \text{m}^{\ 2} \;\; 2.5\ \text{m} \quad 1{,}898\ \text{kN}$$

The factor of safety against passive failure is 1,898 kN/559 kN = 3.4 > 1.5 (OK).

PERMANENT FACING DESIGN

The 254-mm thick permanent CIP concrete facing is designed as a one-way concrete slab with supports at the soldier beam locations. The permanent facing is designed to resist apparent earth pressures and it is assumed that the timber lagging is ineffective in carrying earth pressure loadings for long-term permanent conditions. Using table 13, the maximum bending moment was estimated using a moment coefficient of 1/10. This results in:

$$M_{max} \quad \frac{1}{10}(p_e \quad p_s)s^2$$

$$M_{max} \quad \frac{1}{10}(43.6\,kN/m \quad 3.2\,kN/m)(2.5\,m)^2 \quad 29.3\,kN \quad m/m$$

The structural design of the permanent facing should consider this maximum moment and the connection between the anchor system and the permanent facing should be performed in accordance with the latest AASHTO specifications.

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SUMMARY OF INITIAL DESIGN

Soldier Beams

Initial Design Results

Spacing	2.5 m
Diameter	0.6 m
Embedment	2.5 m
Size	Two MC12 x 31 Grade 50 Channel Sections

Design Analysis Information	Required Properties	Initial Design Results
Section Modulus	0.001001 m^3	0.001109 m^3
Vertical Capacity	471 kN	491 kN

Anchors

Initial Design Results

Rows	2
Size	32-mm diameter Grade 150 bar or 3@15-mm diameter Grade 270 strand
Depth	2.5 m (upper) and 6.25 m (lower)
Inclination	15 for both rows

Design Analysis Information	Required Properties	Initial Design Results
Row 1, Allowable bond capacity	435 kN	600 kN
Row 2, Allowable bond capacity	445 kN	600 kN
32-mm Bar		
Allowable Capacity	445 kN	501 kN
Trumpet Diameter	95 mm	150 mm
3@15-mm Strand		
Allowable Capacity	445 kN	469 kN
Trumpet Diameter	150 mm	150 mm

CONCLUSIONS

The initial design is feasible. A review of the results indicates that sufficient bond capacity is available to permit a wider spacing of the soldier beams. A second iteration of the design should be performed with a wider soldier beam spacing and flatter anchor inclinations to determine the optimum design.