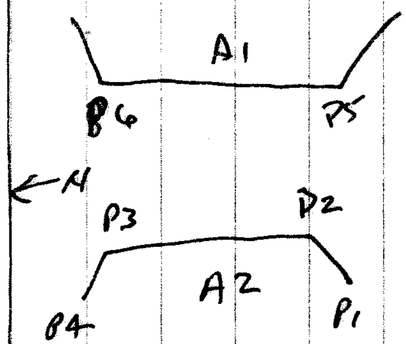
LOC	PLAN	* ↔ △	5/17/84	↔ △	5/22/84	↔ △	6/5/84	△ * 6/5 to PLAN	△ 5/17/84-6/5/84
A2 P1	—	—	135.10	—	13510	3/8	13507	—	3/8"
A2 P2	13491	1/4	134.89	—	13489	1/2	13485	3/4	1/2"
A2 P3	134.22	1 1/8	134.13	—	13413	3/4	13407	1 7/8	3/4"
A2 P4	—	—	133.14 ~~131.91~~	1/4	13312	3/4	13306	—	1"
A1 P5	131.56	5/8	131.91	—	131.91	1/2	13187	1 1/8	1/2"
A1 P6	131.98	1/4	131.46	1/4	131.44	3/4	131.38	1 1/4	1"

* assumes seats finished to grade 11/83

Contraction joint monitors

A2 N	5/19/84 5 7/8"	1/8	5/23/84 6"	1/8	6/6/84 6 1/8 1/4
A2 S	5/23/84 4 5/8	1/16	6/6/84 4 11/16		
A1 N	5/25 6 5/8	—	~~[illegible]~~ 6/6/84 6 5/8	1/16 5/23 to 5/18	
A1 S	OPEN				



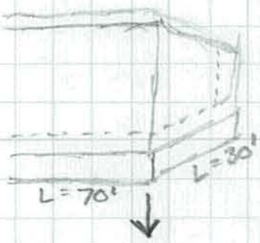
PRELIM. BY ______ DATE ______ PROJ. NO. ______ FILE NO. ____ OF ____
FINAL CHK. BY ______ DATE ______ LOCATION Brewer SH. NO. ____ OF ____
ITEM NO. ______ SUBJECT ~~I-3~~ Wilson ST over I-395

- Assume stiff mottled silty clay at El. $^{+}95 - ^{+}100$
- Thus, 14' of old fill above it. (to footing elev.)

Fin. gr ≈ $^{+}144$

Existing stress (before construction)

17' of old fill   17'×120 pcf = 1.02 TSF



Depth	
0	0
17	1.02 TSF
19.5	1.165
22	1.31

Clay (17 – 22)

Stress increase due to concrete dead load:

Depth	(L=30') m=L/z	(B=12.5') n=B/z	I	(p=1.4) Ip	(L=70') m=L/z	I	p=1.175 Ip	TOTAL
0	∞	∞	.25	.35	∞	.25	.29	.64
8	3.75	1.56	.231	.32	8.75	.231	.27	.59
13	2.3	.96	.198	.28	5.4	.201	.24	.52
15.5	1.94	.81	.182	.25	4.52	.186	.22	.47
18	1.67	.69	.164	.23	3.89	.17	.20	.43

Clay (13 – 18); Wing (first group of columns); Breastwall (second group of columns)

$$\Delta h = 5'\left(\frac{.02}{1+.845}\right) = 0.6''$$

Stress increase due to fully loaded abutment (no fill)

Depth	p=3.67 Ip	p=4.705 Ip	TOTAL
15.5	.67	.88	1.55

$$\Delta h = 5'\left(\frac{.05}{1+.8}\right) = 1.7''$$

Stress increase due to backfill (approach fill)   p = 26' (From 118 to 144) ×125 pcf = 1.625 TSF

Depth	(L=300') m=L/z	(B=80') n=B/z	I	p=1.625 Ip	(B=70') n=B/z	I	(p=1.625) Ip	
El. 118 – 0	∞	∞	.25	.41	∞	.25	.41	.82
18	16.7	4.4	.214	.35	3.9	.25	.41	.76
20.5	14.6	3.9	.21	.34	3.41	.25	.41	.75
23	13	3.48	.204	.33	3.0	.25	.41	.74

$$\Delta h = 5'\left(\frac{.03}{1+.845}\right) = 1''$$

TOTAL Δh = 2.7"

PRELIM. BY ____ DATE ____ PROJ. NO. ____ FILE NO. ____ OF ____
FINAL CHK. BY ____ DATE ____ LOCATION ____ SH. NO. 2 OF ____
ITEM NO. ____ SUBJECT ____

RATE OF CONSOLIDATION:

$$t = T\left(\frac{H^2}{C_v}\right)$$

For 95% consol., $T = 1.125$

$$t = 1.125\left(\frac{(5')^2}{100\,ft^2/yr}\right) = 3.4 \text{ months}$$

5# = 0.69 TSF
10# = 1.385 "
15# = 2.08 "

Abut. 2

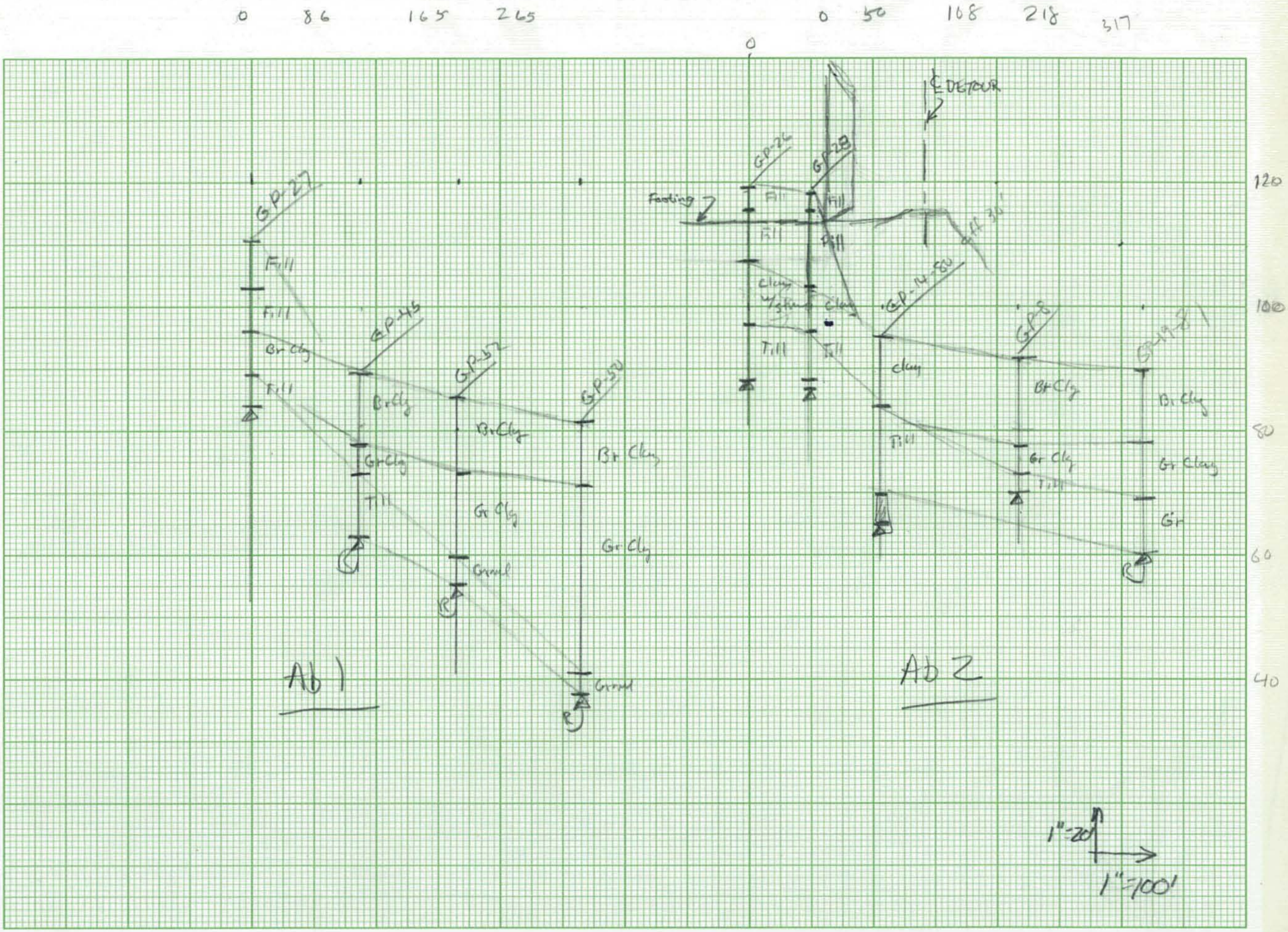
Foot. el. = 113.7 (4' high)
F.G. = 144 ±
Old ground = 118

App. slab elev. = 2'9" below F.G.

What is time span between deck const. and appr. slab const.?

Abut. 1

Foot. el. = 110 (4' high)
F.G. = 139 ±
Old grd. = 111


0
86
165
265
0
50
108
218
317
0
℄ DETOUR
GP-27
Fill
Fill
Br Clay
Till
GP-45
Br Clay
Gr Clay
Till
GP-52
Br Clay
Gr Clay
Gravel
GP-50
Br Clay
Gr Clay
Gravel
GP-26
GP-28
Footing
Fill
Fill
Till
Clay w/ sand
Clay
Till
Till
GP-14-80
Clay
Till
GP-8
Br Clay
Gr Clay
Till
GP-19-81
Br Clay
Gr Clay
Gr
120
100
80
60
40
Ab 1
Ab 2
1"=20'
1"=100'

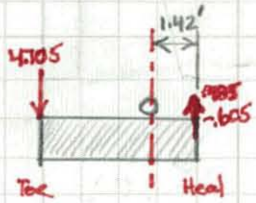
PRELIM. BY PMC DATE ____ PROJ. NO. ____ FILE NO. ____ OF ____
FINAL CHK. BY 5-31-88 DATE ____ LOCATION Brewer SH. NO. ____ OF ____
ITEM NO. ____ SUBJECT Rt 1A over I-395

134.2 10' 16'

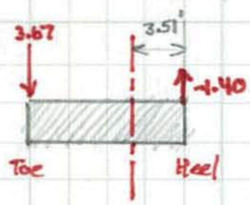
118

DEAD LOADS:

		concrete only	total bridge wt (steel, deck, etc)
Breastwall	Abut #2 (12.5' width) -	1.175 TSF (toe) 0.485 TSF (heel)	4.705 TSF (toe) -0.605 TSF (heel)
	Wing (12'9" wide) -	1.40 TSF (toe) 0 TSF (heel)	3.67 TSF (toe) -1.40 TSF (heel)



Breastwall



Wing

El. G9 Bridge Seat A2 w/ 395 134.22

Abut 2	footing	9/20/83	210 cy
"	Center Sec Breastwall	10/6/83	68 cy
"	Left Sec "	10/11/83	73 cy
"	Left wing	10/13/83	28 cy
"	breastwall Rt	10/14/83	81 cy
"	ftg Rt wing	10/17/83	26 cy
"	Left wing	10/21/83	43 cy
"	Rt. wing	10/28/83	36 cy
"	Backwall	11/18/83	18 cy
"	Backwall	11/28/83	11 cy

—TOTAL 594 cy—

ELEVATIONS: 5/22/84

Abut #2

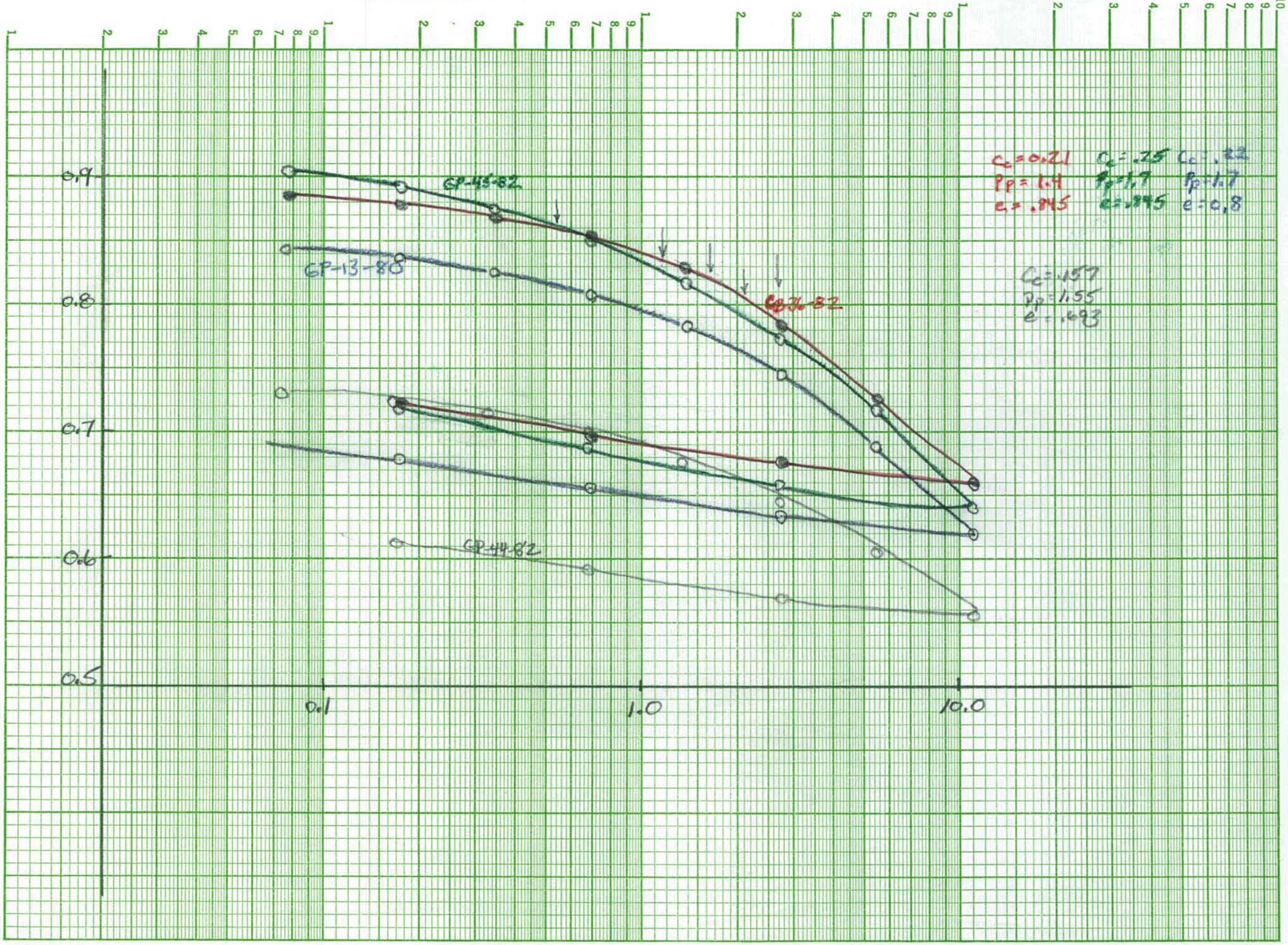
PAD #7 - 1/2" lower than Nov. 83
#8 - 3/4" "
#9 - 1 1/8" "

Abut #1

Pad #1 - 5/8" lower than Nov. 83
#2 - 1/2"
#3 - 1/8"
#4 - 3/8"
#5 - 1/2"
#6 - 3/8" lower than Nov. 83
#7 - 3/8"
#8 - 1/2"
#9 - 1/2"

K+E SEMI-LOGARITHMIC 46 6012
4 CYCLES X 70 DIVISIONS MADE IN U.S.A.
KEUFFEL & ESSER CO.

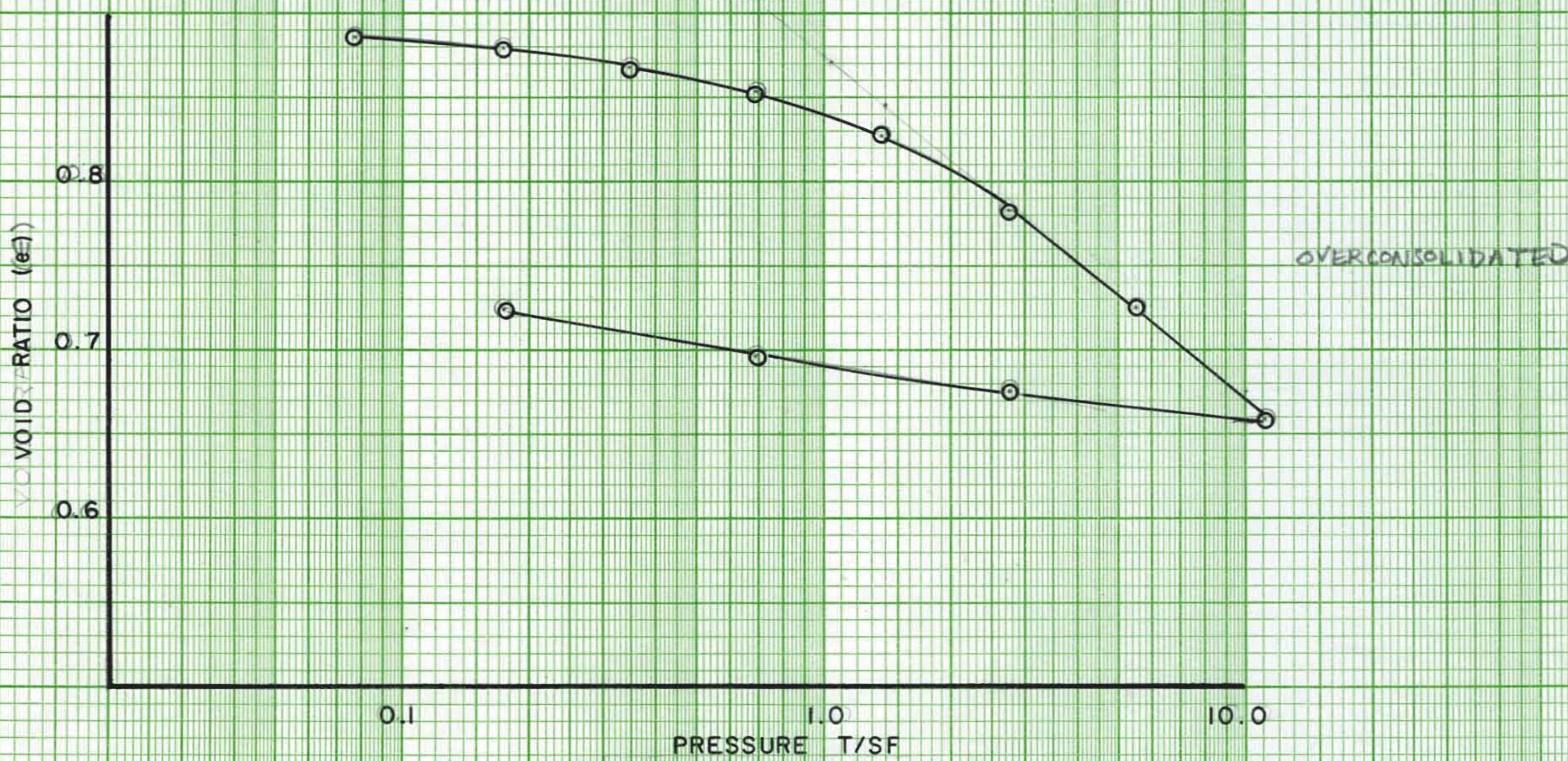
.05
.05



K+E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS KEUFFEL & ESSER CO. 46 6012 MADE IN U.S.A.

VANE	.852	Pmin	1.05	Cv $20^{\#}$	-301
WC's	30-28	Pmax	2.5±	40	-216
G	2.76	Pp	1.4	80	-215
hs	.4659	e	.845		
		Cc	.21		

LC 19-82



PRESSURE - VOID RATIO DIAGRAM

BREWER

395-8 (79)

BORING CB 36-82 SAMPLE 1 U

JULY, 1982

83D

63 + 00 120' RT 5'-7'

K&E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS 46 6012 MADE IN U.S.A. KEUFFEL & ESSER CO.

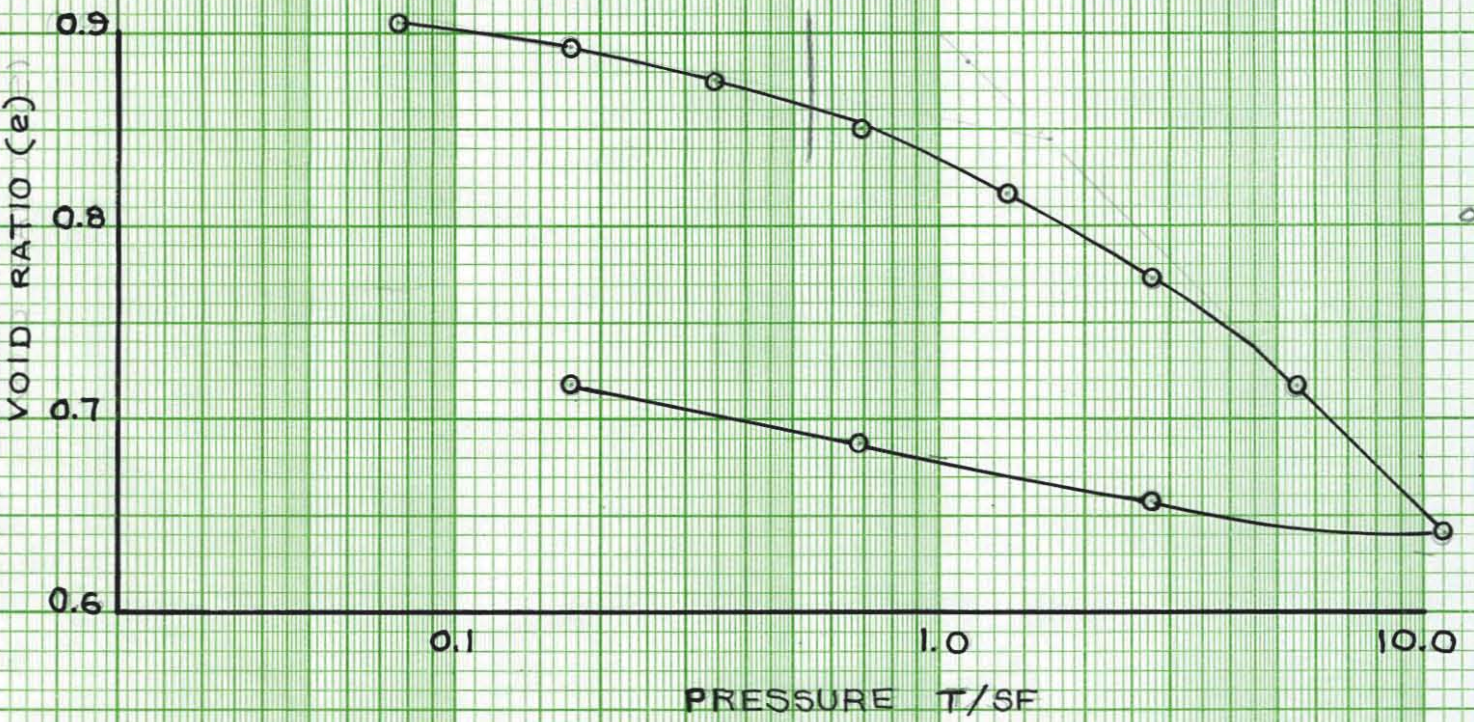
Med-stiff gray silty clay
w/
brown mottling

VANE .528
WC'S 33-27
G 2.78
h_S .4537

P_{MIN} 1.15
P_{MAX} 45
P_P 1.7
e .845
C_c .25

CV 10# 166
20 - 111
40 - 169
80 - 85

LC-24-82



PRESSURE-VOID RATIO DIAGRAM

BREWER

395-8 (79)

BORING GP-45-82 SAMPLE 2U

SEPTEMBER 1982

88C

66+00, 79' RT.

8'-10'

Vane .576
Wc's 32-26
G-2.74
hs-.474

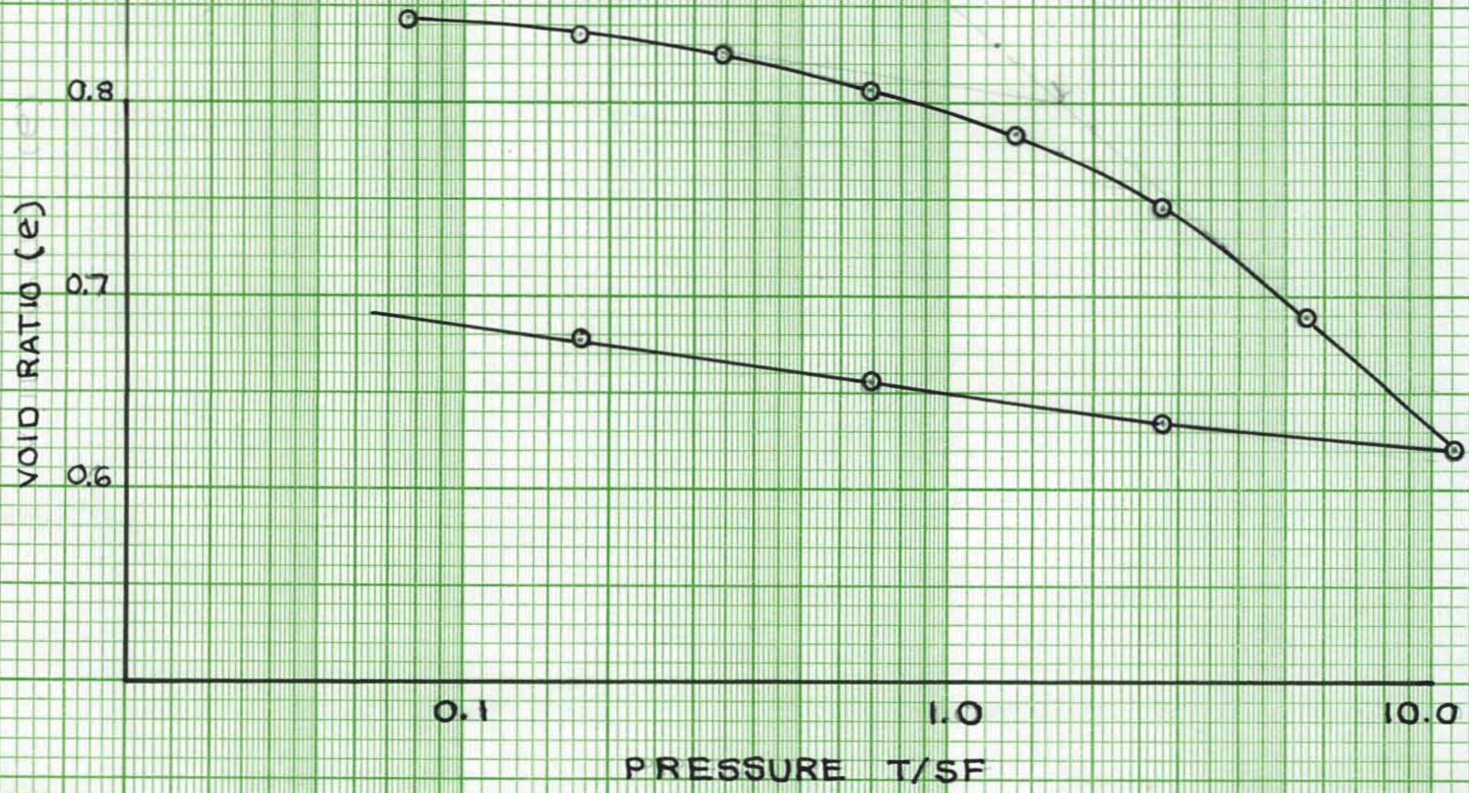
Pmin 1.25
Pmax 3.7
Pp 1.7
e .8
Cc .22

CV 5#-109
10 -128
20 - 91
40 - 83
80 - 87

LC-10

Stiff gray weathered silty clay

LC-10



OVERCONSOLIDATED

PRESSURE - VOID RATIO DIAGRAM
BREWER
395 - 8 (79)
BORING GP-13-80 SAMPLE 1U
APRIL 1980

86C

51+50, 5 RT Felts Brook (335+71, 83R)

10'-12'
el. 75.08

Sheet No.

BORING GP-43-84 STATION ____

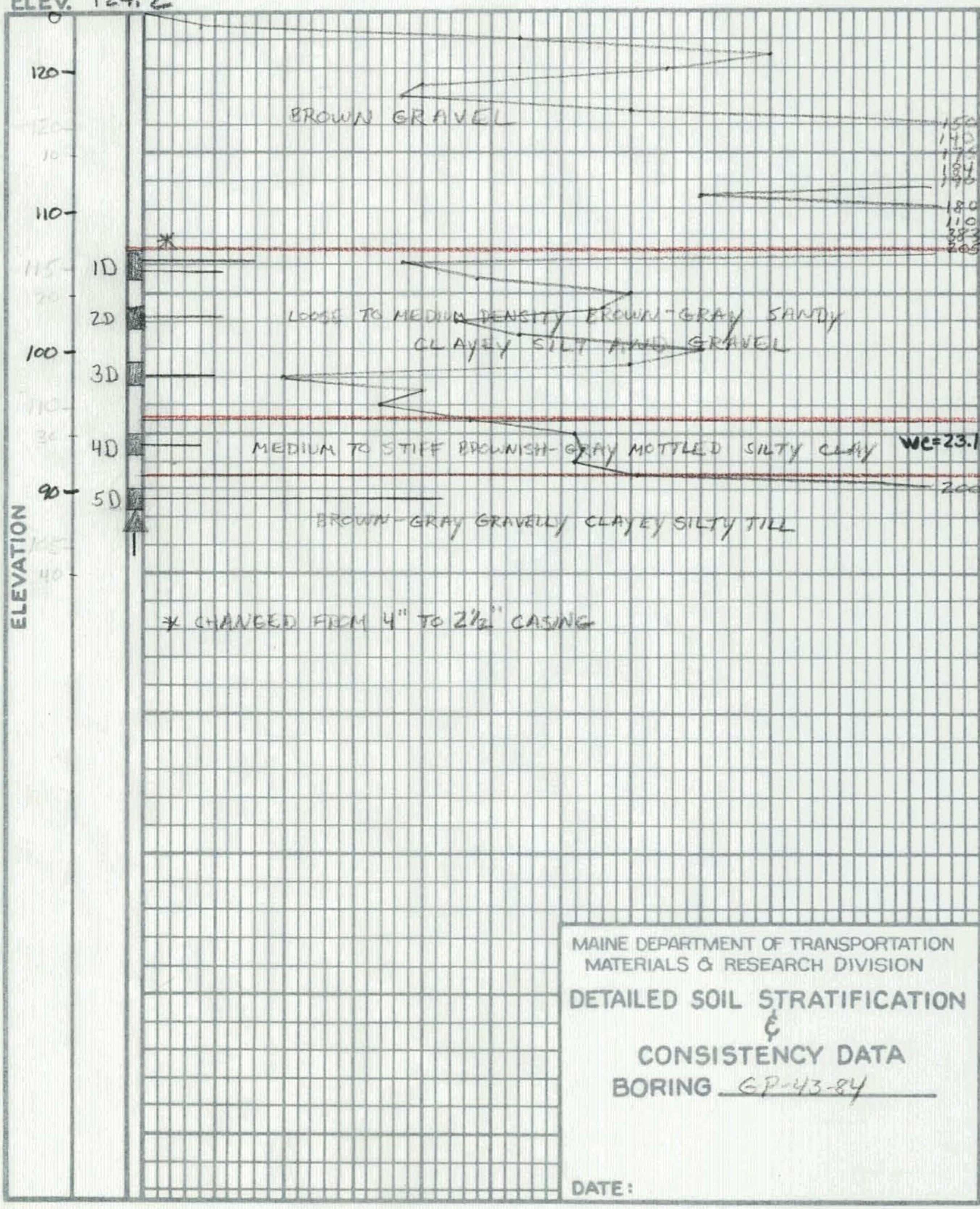
CASING SIZE 4" + 2½"

DRIVING RESISTANCE — Blows/Ft.

20 40 60 80 100

ELEV. 124.2

1" = 10'



MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING GP-43-84

DATE:

SML-202 (8-72)

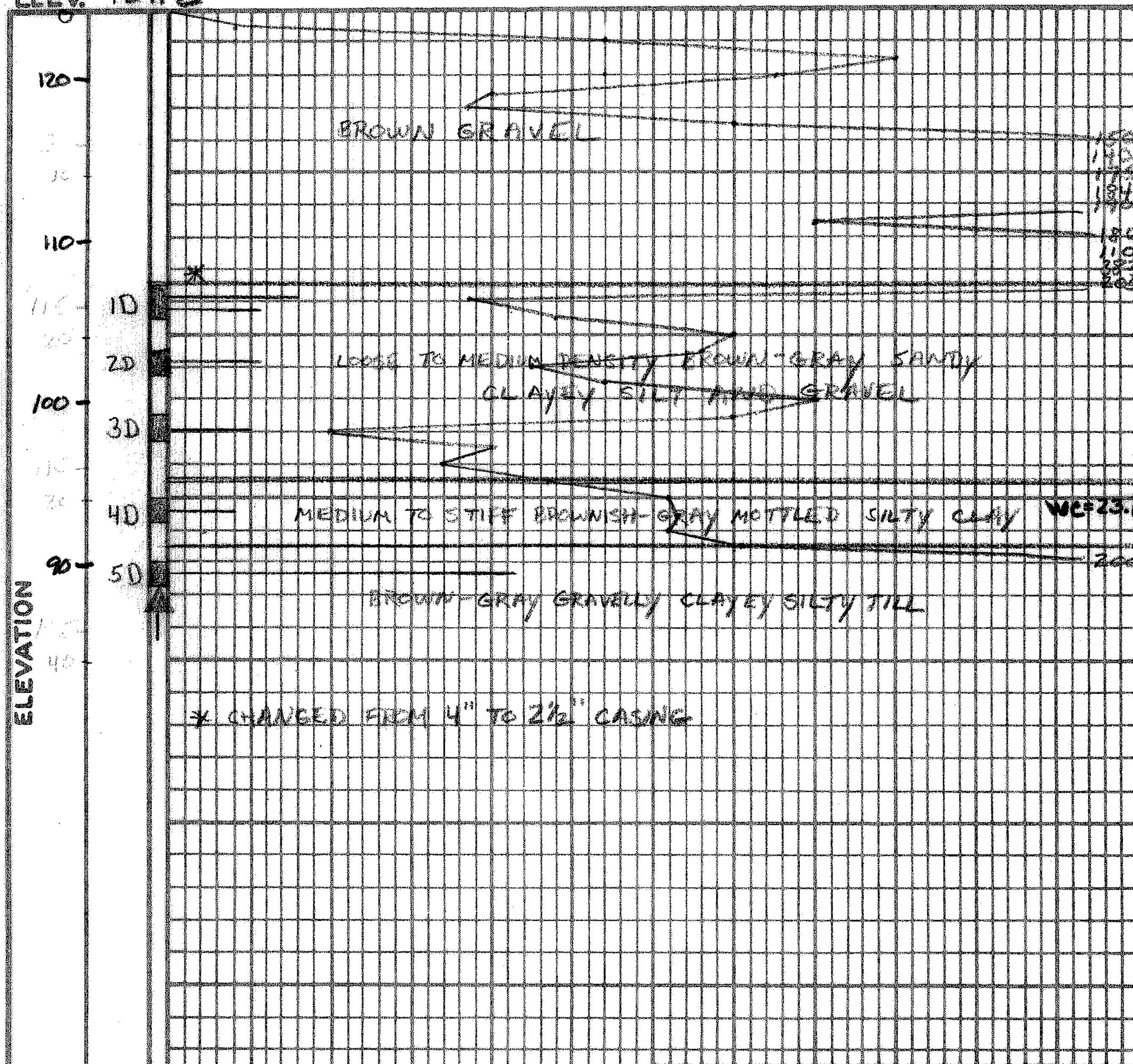
Sheet No.

BORING GP-43-84 STATION

CASING SIZE 4" + 2½"

DRIVING RESISTANCE — Blows/Ft.

ELEV. 124.2



1" = 10'

SML-202 (8-72)

MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING GP-43-84

DATE:

BORING NO. GP-43

Drilling Sequence Continued

Brewer

GROUND WATER OBSERVATIONS

Date	Hour	Water Elevation	Elev. Bottom Casing
1.			
2.			
3.			
4.			
5.			

Remarks

BORING NO. GP-43

CASING RECORD

LENGTH		DRIVING RESISTANCE						
		HAMMER		INT'V'L		INT'V'L		
PC.	TOT.	FALL	BLOWS	FR	TO	FR	TO	NUMBER
1	4^{6}	76"	BR 8	0	1			
4^{4}	8^{10}		54	1	2			
4^{2}	13^{0}		90	2	3			
			75	3	4			
4^{3}	17^{3}		40	4	5			
		4"	37	5	6			
			70	6	7			
			150	7	8			
			140	8	9			
			175	9	10			
			184	10	11			
			190	11	12			
			80	12	13			
			AR 180	13	14			
			110	14	15			
	4^{7}		383	15	16			
4^{3}	8^{10}		205	16	17			
4^{4}	13^{2}	2½"	AS 37	17	18	17	19	1D
4^{3}	17^{5}		48	18	19			
4^{3}	21^{8}		BR 70	19	20			
4^{3}	25^{11}		66	20	21			
4^{3}	30^{2}		AS 45	21	22	21	22^{6}	2^{D}
			54	22	23			
			BR 80	23	24			
			70	24	25			

BORING NO. GP-43

SAMPLES

HAMMER: 310 LBS.

FALL	BLOWS	DESCRIPTION OF MATERIALS
		Gravel
14"	10-6 5-6	Gray Brown Silt + Rocks
14"	6-7 4	

BORING NO. GP-43

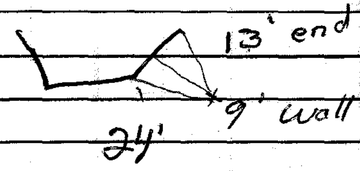
Project No.
Town Brewer
Station
Elev. Ground Surface:
Datum: Survey Plan
Size Casing: 4" + 2½"
Size Rods: AW

DATE	START	FINISH	HOURS	TOTAL DEPTH	WEATHER	TEMP.
5-22	3	5	2	8	Clear	65°
5-23	7	5	10	25'	Overcast	50°
5-24	7	11	4	35'6"	Clear	55°

Drilling Sequence Drove Cas to 8' w/o Drove Cas to 17' w/o Pulled Cas Drove 2½" Cas to 17' w/o Took Sample 1D From 17 to 19' Brown Gravel Changing to Gray Brown Silt + Rocks at 17'0" Drove Cas to 21' w/o Took Sample 2D From 21 to 22'6" Drove Cas to 25' w/o Took Sample 3D From 25 to 26'6" Drove Cas to 30' w/o Changed to Stiff Gray Brown Silty Clay at 29'0" Took Sample 4D From 30 to 31'6" Drove Cas to 34' w/o Changed to Brown Silty Till at 33' Took Sample 5D From 34 to 35'6" Pulled Casing No Refusal

BORING NO. GP-43

Drilling Sequence Continued



GROUND WATER OBSERVATIONS

Date	Hour	Water Elevation	Elev. Bottom Casing
1.			
2.			
3.			
4.			
5.			

Remarks

BORING NO. GP-43

CASING RECORD

LENGTH		DRIVING RESISTANCE						
		HAMMER		INT'V'L		INT'V'L		
PC.	TOT.	FALL	BLOWS	FR	TO	FR	TO	NUMBER
1	30^{2}	16"	AS 20	25	26	25	26^{6}	3D
			41	26	27			
4^{4}	34^{6}		BR 34	27	28			
			47	28	29			
			62	29	30			
			AS 63	30	31	30	31^{6}	4D
			62	31	32			
			71	32	33			
			200	33	34			
				34	35	34	35^{6}	5D
				35	36			

BORING NO. GP-43

SAMPLES

HAMMER: 310 LBS.

FALL	BLOWS	DESCRIPTION OF MATERIALS
	6-5	
	4	
16	3-4	Gray Brown Silty Clay
	5	
16	25-21	
	18	Brown Silty Till

BORING NO. GP-43

Project No.

Town

Station

Elev. Ground Surface:

Datum:

Size Casing:

Size Rods:

DATE	START	FINISH	HOURS	TOTAL DEPTH	WEATHER	TEMP.

Drilling Sequence

setup 5/29

Maine State Highway Commission
BORING LOG

LOCATION: Brewer STRUCTURE: ______ SHEET No. 1 OF: 2

BORING No. 43 DATUM: Survey Plan BORING INSPECTOR: G. Paine DATE 5-29-84

ELEVATION	DEPTH	LEGEND	DESCRIPTION OF MATERIALS (TYPE, COLOR & CONSISTENCY)	CASING BLOWS	CASING PENETRATION	SAMPLER OR SPOON BLOWS *	SAMPLER OR SPOON PENETRATION	SAMPLE NO. **	REMARKS *** Rt 1A over I-395 Abut. #2 STA.
	0'0"		Ground Surface						ELEV. = 124.2
				BR 8	12"				
				54	"				
				90	"				
				75	"				
				40	"				
				37	"				
				20	"				
		17'0"	Brown Gravel	150	"				
				140	"				
				175	"				
				184	"				
				190	"				
				80	"				
				AC 180	"				
				110	"				
				383	"				
	17'0"			205	"				
				AS 37	"	10 6	12"	1D	
				48	"	5 6	"		
				BR 70	"				
		12'0"	Gray Brown Silt +	66	"				
			Rocks	AS 45	"	6 7	12"	2D	
				54	"	4	6"		
				BR 80	"				
				70	"				

* When sampler penetrates under weight of rods, rods plus hammer, or force of jack, record should be made of fact.

** Write sample No. at corresponding depth. Designate dry samples by D. Wash samples by W. 3 1/2" undisturbed tube samples by U. Rock cores by R. 2" tube samples by C.

*** When drilling cores in rock, record the percentage of recovery in each foot of penetration.

over

SML-8 (10/62)

LOCATION OF PROJECT: Brewer

LOCATION OF BORING:

ELEVATION OF GROUND SURFACE:

DRILL NO. 41-006 BORING FOREMAN: G. Paino

SIZE AND WEIGHT OF CASING: 4" Standard DEPTH: 17'0"

2½" Extra Heavy 34'0"

LENGTH OF HOLE; IN EARTH = 35'6" IN ROCK = ——

TYPE OF ROCK DRILL USED:

HAMMER: WEIGHT = 310 lbs. AVG. FALL (ON CASING = 16 ins.

(ON RODS = 14 spoon ins.

ELEVATION OF GROUND WATER SURFACE:

DATE	HOUR	WATER ELEVATION	EL. BOTTOM CASING

RECORD OF WORK

DATE	5-22-84	5-23-84	5-24-84				
START	3:00	7:00	7:00				
FINISH	5:00	5:00	11:00				
HOURS	2	10	4				
TOTAL DEPTH	8'0"	25'0"	35'6"				
WEATHER	Clear	Overcast	Clear				
TEMPERATURE	65°	55°	55°				

BORING INSPECTOR: Gary Paino

REMARKS

NOTE—Mark all sample containers with special gummed labels, completely filled out.

Maine State Highway Commission

BORING LOG

LOCATION: Brewer STRUCTURE: ______ SHEET No. 2 OF: 2

BORING No. 43 DATUM: Survey Plan BORING INSPECTOR: G. Paine DATE 5-29-84

STRATIFICATION			DESCRIPTION OF MATERIALS (TYPE, COLOR & CONSISTENCY)	CASING		SAMPLER OR SPOON		SAMPLE NO. **	REMARKS ***
ELEVATION	DEPTH	LEGEND		BLOWS	PENETRATION	BLOWS *	PENETRATION		
				20	12"	6 5	12"	3D	
				41	"	4	6"		
				34	"				
	29'0"			47	"				
		X		62	"				
		4'0"	Gray Brown Silty Clay	63	"	3 4	12"	4D	
				62	"	5	6"		
	33'0"			71	"				
		X		200	"				
		2'6"	Brown Silty Till			25 21	12"	5D	
	35'6"	—				18	6"		

* When sampler penetrates under weight of rods, rods plus hammer, or force of jack, record should be made of fact.

** Write sample No. at corresponding depth. Designate dry samples by D. Wash samples by W. 3½" undisturbed tube samples by U. Rock cores by R. 2" tube samples by C.

*** When drilling cores in rock, record the percentage of recovery in each foot of penetration.

over

SML-8 (10/62)

BORING TEST and SUMMARY SHEET

BORING NO. GP-43-84
PROJ. NO. 39.24 95 0 27 920
TOWN Brewer
BRIDGE Rt 1A over 395

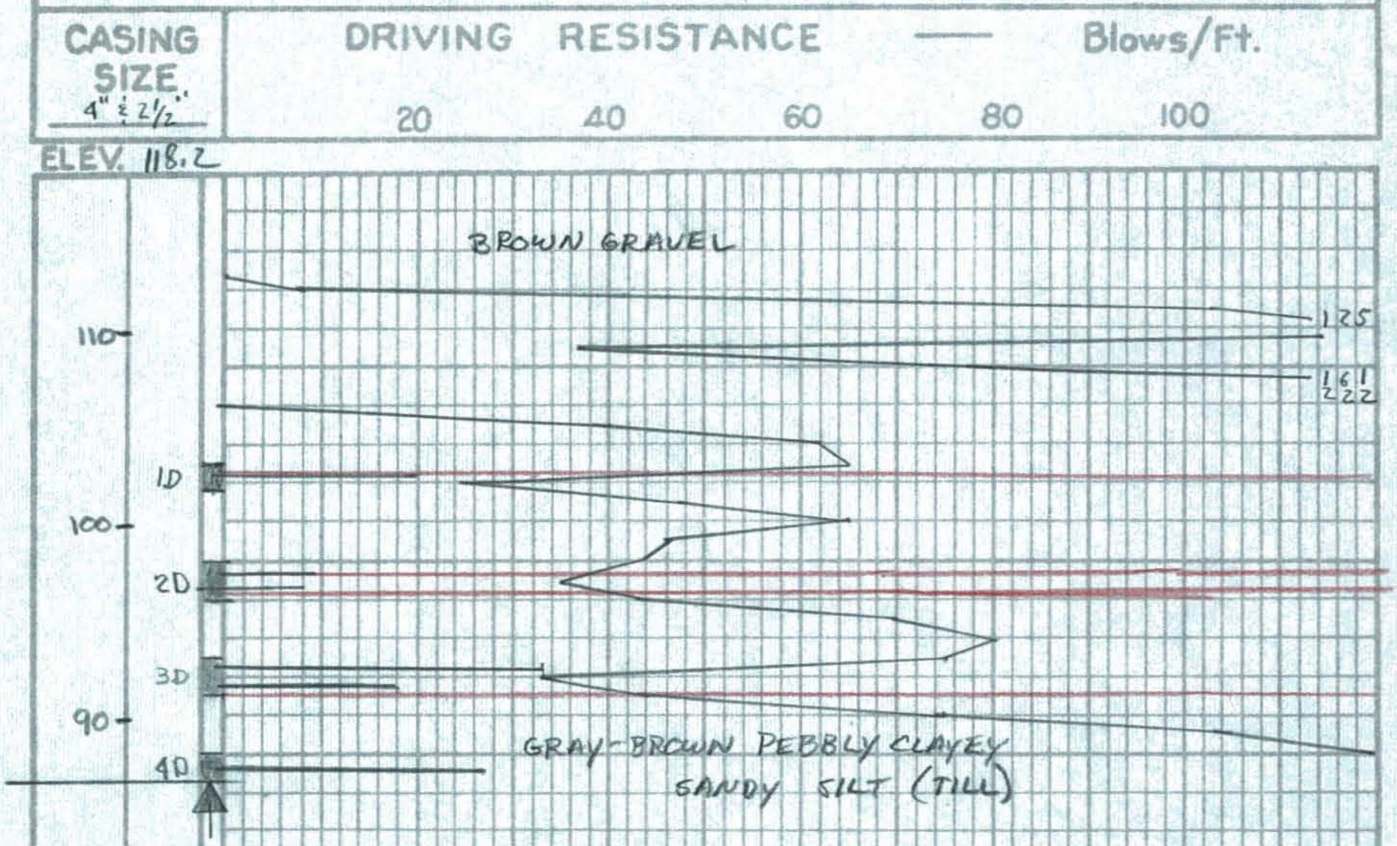
DATE SET UP 5-29-84

SAMPLE NO. & DEPTH	TESTS REQUESTED	TEST COMPLETE	DRAFTING COMPLETE	VISUAL DESCRIPTION OR REMARKS
1D 17-19				Brown sandy clayey silt ~~Clay~~ w/ fine pebbles.
2D 21-22^{6}				Brown-gray gravelly silty sand
3D 25-26^{6}				Brown-gray clay sandy silt w/ few small rocks
4D 30-31^{6}				Stiff brownish-grey lightly mottled silty clay
5D 34-35^{6}				Brown-gray gravelly clayey silty sand (till)

SML-210 (5/69)

Sheet No.

BORING GP-44-84 STATION



SML-202(8-72)

MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA

BORING GP-44-84

Brewer

DATE:

BORING NO. GP-44

Drilling Sequence Continued

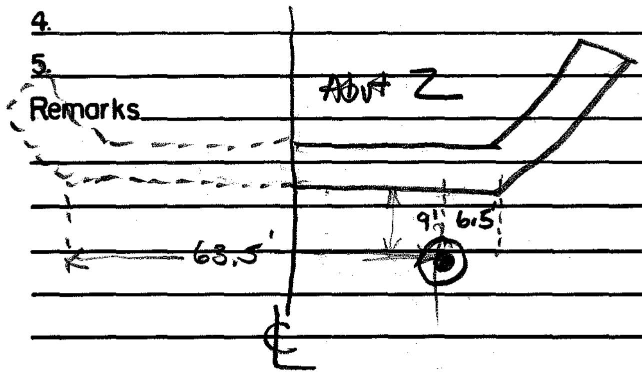
39.24

Brewer

GROUND WATER OBSERVATIONS

Date	Hour	Water Elevation	Elev. Bottom Casing
1.			
2.			
3.			
4.			
5.			

Remarks



SML-10 (10/72)

BORING NO. GP-44

CASING RECORD

LENGTH		DRIVING RESISTANCE						
		HAMMER		INT'V'L		INT'V'L		
PC.	TOT.	FALL	BLOWS	FR	TO	FR	TO	NUMBER
/	4^{9}	16"	BR Avr gerd	0	1			
4^{2}	8^{6}			1	2			
4^{3}	12^{9}		Ahead	2	3			
				3	4			
		4"		4	5			
			8	5	6			
			103	6	7			
			125	7	8			
/	4^{6}	2⅛"	AR 37	8	9			
4^{3}	8^{9}		80	9	10			
4^{3}	13^{0}		BR 161	10	11			
4^{3}	17^{3}		222	11	12			
4^{3}	21^{6}		40	12	13			
4^{3}	25^{9}		62	13	14			
			65	14	15			
			AS 25	15	16	15	16^{6}	1D
			48	16	17			
			BR 65	17	18			
			46	18	19			
			44	19	20			
			AS 35	20	21	20	22	2D
			44	21	22			
			BR 70	22	23			
			80	23	24			
			75	24	25			

BORING NO. GP-44

SAMPLES

HAMMER: 310 LBS.

FALL	BLOWS	DESCRIPTION OF MATERIALS
		Brown Gravel
14"	9-11 *	
	10	
		Gray Brown Silty Till
14"	5-4 *	Gray Pebbly Silt
	4-4 *	
		Brown Silty Pebbly Sand

SML-9(10/72)

BORING NO. GP-44

Project No. 39.24
Town Brewer
Station
Elev. Ground Surface:
Datum:
Size Casing: 4" Standard 2½" Extra Heavy
Size Rods: AW

DATE	START	FINISH	HOURS	TOTAL DEPTH	WEATHER	TEMP.
5-30	10	5	7	25	Showers	50
5-31	6	8	2	31'6"	Showers	45°

Drilling Sequence Drove Cas to 8' w/o To 10 Drove Cas to 12' w/o Put 2½" Cas in Drove to 15' w/o Took Sample 1D From 15 to 16⅙ Changed to Gray Brown Siltx Till at 15'6" Drove Cas to 20' w/o Took Sample 2D From 20 to 22 Changed to Gray Pebbly Silt at 20'6" Changed to Brown Siltx Pebbly Sand at 21'6" Drove Cas to 25' w/o Took Sample 3D From 25 to 27 Drove Cas to 30' w/o Changed to Brown Sandx Pebbly Till at 27' Took Sample 4D From 30 to 31'6" No Refusal Pulled Casing.

BORING NO. GP-44

Drilling Sequence Continued

GROUND WATER OBSERVATIONS

Date	Hour	Water Elevation	Elev. Bottom Casing
1.			
2.			
3.			
4.			
5.			

Remarks

BORING NO. GP-44

CASING RECORD

LENGTH		DRIVING RESISTANCE						
		HAMMER		INT'V'L		INT'V'L		
PC.	TOT.	FALL	BLOWS	FR	TO	FR	TO	NUMBER
/	25⁹	16"	AS 33	25	26	25	27	3D
			44	26	27			
4³	30⁰		BR 75	27	28			
			105	28	29			
			120	29	30			
				30	31	30	31½	4D
				31	32			

BORING NO. GP-44

SAMPLES

HAMMER: 310 LBS.

FALL	BLOWS	DESCRIPTION OF MATERIALS
14"	12-11	
	11-7 X	
		Brown Sandy Pebbly
		Till
14"	10-10	
	20 /	

BORING NO. GP-44

Project No.

Town

Station

Elev. Ground Surface:

Datum:

Size Casing:

Size Rods:

DATE	START	FINISH	HOURS	TOTAL DEPTH	WEATHER	TEMP.

Drilling Sequence

39.24 set -p 6-1-84

Maine State Highway Commission
BORING LOG

LOCATION: Brewer STRUCTURE: SHEET No. 1 OF: 2

BORING No. 44 DATUM: Survey Plan BORING INSPECTOR: G. Paine DATE 5-31-84

STRATIFICATION: ELEVATION	DEPTH	LEGEND	DESCRIPTION OF MATERIALS (TYPE, COLOR & CONSISTENCY)	CASING: BLOWS	CASING: PENETRATION	SAMPLER OR SPOON: BLOWS *	SAMPLER OR SPOON: PENETRATION	SAMPLE NO. **	REMARKS ***
	0'0"		Ground Surface						ELEV. = 118.2
				BR	12"				20.6
					"				97.6
					"				
					"				
					"				
					6"				
				8	6"				
				103	12"				
		15'6"	Brown Gravel	125	"				
				37	"				
				80	"				
				BR 161	"				
				222	"				
				40	"				
				62	"				
	15'6"			65	"				
				25	"	9 11	12"	1D	
				48	"	10	6"		
		5'0"	Gray Brown Silty	BR 65	"				
			Till	46	"				
	20'6"			44	"				
		1'0"	Gray Pebbly Silt	35	"	5 4	12"	2D	
	22'6"			44	"	4 4	"		
				BR 70	"				
				80	"				
		5'6"	Brown Silty Pebbly Sand	75	"				

* When sampler penetrates under weight of rods, rods plus hammer, or force of jack, record should be made of fact.

** Write sample No. at corresponding depth. Designate dry samples by D. Wash samples by W. 3½" undisturbed tube samples by U. Rock cores by R. 2" tube samples by C.

*** When drilling cores in rock, record the percentage of recovery in each foot of penetration.

over

SML-8 (10/62)

LOCATION OF PROJECT: Brewer

LOCATION OF BORING:

ELEVATION OF GROUND SURFACE:

DRILL NO. 41-006 BORING FOREMAN: G. Paine

SIZE AND WEIGHT OF CASING: 4" Standard DEPTH: 12'0"
2½" Extra Heavy 30'0"

LENGTH OF HOLE; IN EARTH = 31'6" IN ROCK = —

TYPE OF ROCK DRILL USED: —

HAMMER: WEIGHT = 310 lbs. AVG. FALL (ON CASING = 16 ins.
(ON RODS = 14 spoon ins.

ELEVATION OF GROUND WATER SURFACE:

DATE	HOUR	WATER ELEVATION	EL. BOTTOM CASING

RECORD OF WORK

DATE	5-30-84	5-31-84					
START	10:00	6:00					
FINISH	5:00	8:00					
HOURS	7	2					
TOTAL DEPTH	25'0"	31'6"					
WEATHER	Showers	Showers					
TEMPERATURE	50°	45°					

BORING INSPECTOR: Gary Paine

REMARKS

NOTE—Mark all sample containers with special gummed labels, completely filled out.

39.24

Maine State Highway Commission
BORING LOG

LOCATION: Brewer STRUCTURE: SHEET No. 2 OF 2

BORING No. 44 DATUM: Survey Plan BORING INSPECTOR: G. Paine DATE 5-31-84

STRATIFICATION: ELEVATION	DEPTH	LEGEND	DESCRIPTION OF MATERIALS (TYPE, COLOR & CONSISTENCY)	CASING: BLOWS	CASING: PENETRATION	SAMPLER OR SPOON: BLOWS *	SAMPLER OR SPOON: PENETRATION	SAMPLE NO. **	REMARKS ***
	25'0"								
				15 33	12"	12 11	12"	3D	
	27'0"			44	"	11 7	"		
		X							
				BR 75	"				
		4'6"	Brown Sand, Pebbly	105	"				
			Till	120	"				
						10 10	12"	4D	
	31'6"	Y				20	6"		

* When sampler penetrates under weight of rods, rods plus hammer, or force of jack, record should be made of fact.

** Write sample No. at corresponding depth. Designate dry samples by D. Wash samples by W. 3½" undisturbed tube samples by U. Rock cores by R. 2" tube samples by C.

*** When drilling cores in rock, record the percentage of recovery in each foot of penetration.

SML-8 (10/62) over

BORING TEST and SUMMARY SHEET

BORING NO. GP-44-84

PROJ. NO. 39.24

TOWN Brewer

BRIDGE

DATE SET UP 6-1-84

SAMPLE NO. & DEPTH	TESTS REQUESTED	TEST COMPLETE	DRAFTING COMPLETE	VISUAL DESCRIPTION OR REMARKS
1D 15-16⁶				Gray pebbly sandy clay-silt
2D 20-22				Stiff gray-brown lightly mottled sandy clay-silt w/ a few small stones + pebbles
3D 25-27				Brown sandy silt w/ a few small stones and pebbles
4D 30-31⁶				Gray-brown pebbly clayey sandy silt w/ occas. large stone.

SML-210 (5/69)

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BUREAU OF HIGHWAYS
MATERIALS & RESEARCH DIVISION

WATER CONTENT DETERMINATION

PROJECT NO. 395-8(79) TOWN BREWER

BORING NO. GP44-84 DATE

SAMPLE NO. LL= LI= LC= LG= LD= LU= LT=

LENGTH	1D	2D	3D	4D.		
TARE NO.	140	35	15	20		
T+ WET SOIL	256.2	187.1	212.1	222.9		
T+ DRY SOIL	234.3	161.8	191.7	203.7		
WT. WATER	21.9	25.3	20.4	19.2		
TARE WEIGHT	62.7	64.7	61.9	52.8		
WEIGHT SOIL	171.6	97.1	129.8	150.9		
WATER CONT.	12.8	26.1	15.7	12.7		
VANE (IN. LBS)						
VANE (T/SQ.FT.)						

SAMPLE NO. LL= LI= LC= LG= LD= LU= LT=

LENGTH						
TARE NO.						
T+ WET SOIL						
T+ DRY SOIL						
WT. WATER						
TARE WEIGHT						
WEIGHT SOIL						
WATER CONT.						
VANE (IN. LBS.)						
VANE (T/SQ.FT.)						

SAMPLE NO. LL= LI= LC= LG= LD= LU= LT=

LENGTH						
TARE NO.						
T+ WET SOIL						
T+ DRY SOIL						
WT. WATER						
TARE WEIGHT						
WEIGHT SOIL						
WATER CONT.						
VANE (IN. LBS.)						
VANE (T/SQ.FT.)						

SAMPLE NO. LL= LI= LC= LG= LD= LU= LT=

LENGTH						
TARE NO.						
T+ WET SOIL						
T+ DRY SOIL						
WT. WATER						
TARE WEIGHT						
WEIGHT SOIL						
WATER CONT.						
VANE (IN. LBS.)						
VANE (T/SQ.FT.)						

TESTED BY

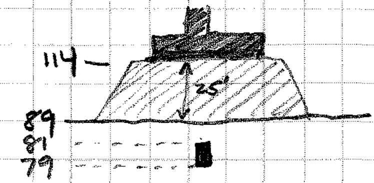
CHECKED BY

SML-103 (3/67)

PRELIM. BY ______ DATE ______ PROJ. NO. ______ FILE NO. ____ OF ____
FINAL CHK. BY ______ DATE ______ LOCATION Brewer SH. NO. ____ OF ____
ITEM NO. ______ SUBJECT Settlement Abut 2 - Rt 1A over I-395

GP-45-82, ZU

Depth = 8'-10'
Elev. = 81.1-79.1



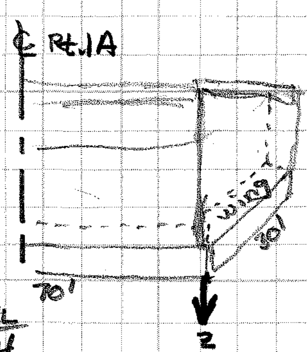
25 x 120 pcf = 1.5 TSF

Existing stress:

9' x 120 pcf = 0.54 TSF

Stress increase due to 25' fill:

Depth	a/z (a=60)	b/z (b=85)	I	2Ip (p=1.5)
0	∞	∞	.50	1.5
9'	6.7	9.4	.50	1.5



Stress increase due to concrete dead load:

Depth	m = a/z (L=30')	n = b/z (B=12.5')	I	Ip (p=1.4)	m = a/z (L=70')	I	Ip (p=1.175)	TOTAL
0	∞	∞	.25	.35	∞	.25	.29	.64
8	3.75	1.56	.231	.32	8.75	.231	.27	.59
17	1.76	.73	.17	.24	4.11	.176	.21	.45
25	1.20	.50	.127	.18	2.8	.137	.16	.34
clay → 34	.88	.37	.091	.13	2.06	.106	.12	.25
	Wing				Breastwall			

TOTAL INCREASE = 1.5 + .25 = 1.75 TSF

$$\Delta h = 12'\left(\frac{.86 - .785}{1 + .845}\right) = 6''$$

APPENDIX E

Final Design Calculations

Seismic Site Class and Design Parameters

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values and assumed S_u values from test borings drilled approximately near the proposed substructures.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the six test borings near the proposed substructures (BB-BWS-102, BB-BWS-103, BB-BWS-104, BB-BWS-202, BB-BWS-203, and BB-BWS-206), recommend a **Seismic Site Class C**.

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
3. International Building Code 2009.
4. ASCE/SEI 7-05 Minimum Design Loads For Buildings and Other Structures
5. International Building Code 2012.
6. ASCE/SEI 7-10 Minimum Design Loads For Buildings and Other Structures

AVAILABLE INFORMATION

1. Test boring logs.
2. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. WOH/WOR = SPT N-value of 1.
3. For test borings BB-BWS-103, BB-BWS-104, BB-BWS-202, BB-BWS-203, and BB-BWS-206, used Method C and assumed S_u values per MaineDOT Field Indentification Card.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A
Average shear wave velocity for the upper 100 ft of the soil profile:

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

PROCEDURE

Method B

Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th soil layer not to exceed 100 ft (blows/ft).

d_i = thickness of *i* th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n.

Method C

Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th cohesionless soil layer (blows/ft).

d_i = thickness of *i* th cohesionless soil layer (ft).

m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and m.

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where

s_{ui} = undrained shear strength of *i* th cohesive soil layer (psf), not to exceed 5000 psf

d_i = thickness of *i* th cohesive soil layer (ft).

k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and k.

Based on the available information, Method A/B/C will be used for the seismic Site Class evaluation.

SITE CLASS DEFINITIONS

(Table from AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (with 2012 Interim Revisions)).

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s$ < 5000 ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N}$ > 50 blows/ft or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40%, and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific ground motion response evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

HALEY ALDRICH	CALCULATIONS	File No.	132076-005
		Sheet	4 of 9
Client	Maine Department of Transportation	Date	16-Mar-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by	NAS
Subject	Seismic Site Class Evaluation	Checked by	KAR

CALCULATIONS - METHOD B

Exploration ID: BB-BWS-102
Ground Surface El.: 112.2

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1.0	111.2	SILT (Fill)	2.0	5	0.400
2D	3.0	109.2	SILT (Fill)	2.0	10	0.200
3D	5.0	107.2	SILT (Fill)	2.0	34	0.059
4D	7.0	105.2	SILT (Fill)	2.0	22	0.091
5D	9.0	103.2	SILT (Fill)	2.0	32	0.063
6D	11.0	101.2	SILT (Glacial Till)	3.5	26	0.135
7D	16.0	96.2	SILT (Glacial Till)	5.0	54	0.093
8D	20.7	91.5	SILT (Glacial Till)	4.7	50	0.094
9D	25.9	86.3	SILT (Glacial Till)	5.2	62	0.084
10D	30.9	81.3	SILT (Glacial Till)	5.0	58	0.086
11D	35.6	76.6	SILT (Glacial Till)	6.4	71	0.090
R1-R4	39.8	72.4	BEDROCK	60.2	100	0.602
			Totals =	100.0		1.996

N-bar (blows/ft) = 50.1

Site Class = C

HALEY ALDRICH	CALCULATIONS	File No.	132076-005
		Sheet	5 of 9
Client	Maine Department of Transportation	Date	16-Mar-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by	NAS
Subject	Seismic Site Class Evaluation	Checked by	KAR

CALCULATIONS - METHOD C

Exploration ID: BB-BWS-103
Ground Surface El.: 107.1

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless SPT N (blows/ft)	Cohesionless d/N	Cohesive Su (psf)	Cohesive d/Su
1D	1.0	106.1	SAND (Fill)	2.5	7	0.357		
2D/2A	4.0	103.1	SAND (Fill)	2.5	13	0.192		
3D	6.0	101.1	SILT (Marine Deposit)	3.5			2000	0.002
4D	11.0	96.1	SILT (Marine Deposit)	5.0			2000	0.003
5D	16.0	91.1	SILT (Marine Deposit)	5.0			4000	0.001
6D	20.7	86.4	SILT (Glacial Till)	4.7	69	0.068		
7D	25.7	81.4	SILT (Glacial Till)	5.0	77	0.065		
8D	30.4	76.7	SILT (Glacial Till)	6.0	50	0.120		
R1-R10	34.2	72.9	BEDROCK	65.8	100	0.658		
					Σd/N =	1.461	Σd/Su =	0.006

Total Thickness of Cohesionless (ft) =	86.5	$N_{ch\text{-}bar}$ (blows/ft) =	59.2	$S_{u\text{-}bar}$ (psf) =	2455
Total Thickness of Cohesive (ft) =	13.5	Site $Class_N$ =	C	Site $Class_{Su}$ =	C
Total Thickness (ft) =	100.0	**Site Class =**	**C**		

HALEY ALDRICH	CALCULATIONS	File No.	132076-005
		Sheet	6 of 9
Client	Maine Department of Transportation	Date	16-Mar-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by	NAS
Subject	Seismic Site Class Evaluation	Checked by	KAR

CALCULATIONS - METHOD C

Exploration ID: BB-BWS-104
Ground Surface El.: 100.9

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless SPT N (blows/ft)	Cohesionless d/N	Cohesive Su (psf)	Cohesive d/Su
1D	1.0	99.9	SILT (Fill)	2.0	3	0.667		
2D	3.0	97.9	SILT (Fill)	2.0	11	0.182		
3D	5.0	95.9	GRAVEL (Fill)	2.0	28	0.071		
4D	7.0	93.9	SILT (Marine Deposit)	3.0			2000	0.002
5D	11.0	89.9	SILT (Marine Deposit)	4.5			3000	0.002
6D	16.0	84.9	SILT (Marine Deposit)	5.0			3000	0.002
7D	21.0	79.9	SILT (Marine Deposit)	5.0			3000	0.002
8MD	25.7	75.2	SILT (Glacial Till)	3.2	78	0.041		
9D	27.4	73.5	SILT (Glacial Till)	2.2	50	0.044		
10D	31.0	69.9	SILT (Glacial Till)	4.6	42	0.110		
11D	35.5	65.4	SILT (Glacial Till)	3.3	50	0.066		
R1-R3	36.8	64.1	BEDROCK	63.2	100	0.632		
					Σd/N =	1.812	Σd/Su =	0.006

Total Thickness of Cohesionless (ft) =	82.5	$N_{ch\text{-}bar}$ (blows/ft) =	45.5	$S_{u\text{-}bar}$ (psf) =	2763
Total Thickness of Cohesive (ft) =	17.5	Site $Class_N$ =	D	Site $Class_{Su}$ =	C
Total Thickness (ft) =	100.0	**Site Class =**	**D**		

	CALCULATIONS	File No.	132076-005
		Sheet	7 of 9
Client	Maine Department of Transportation	Date	15-Mar-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by	NAS
Subject	Seismic Site Class Evaluation	Checked by	BCS

CALCULATIONS - METHOD C

Exploration ID: BB-BWS-202
Ground Surface El.: 143.8

					Cohesionless		Cohesive	
Sample Number	**Depth (ft)**	**Elevation (ft)**	**Description**	**d (ft)**	**SPT N (blows/ft)**	**d/N**	**Su (psf)**	**d/Su**
1D	1.0	142.8	SAND (Fill)	3.0	40	0.075		
2D	3.0	140.8	SAND (Fill)	2.0	38	0.053		
3D	5.0	138.8	SAND (Fill)	2.0	70	0.029		
4D	7.0	136.8	SAND (Fill)	3.0	70	0.043		
5D	10.0	133.8	SAND (Fill)	5.0	103	0.049		
6D	15.0	128.8	SAND (Fill)	5.0	43	0.116		
7D	20.0	123.8	SAND (Fill)	5.0	80	0.063		
8D	25.0	118.8	SAND (Fill)	5.0	54	0.093		
9D	30.0	113.8	GRAVEL (Fill)	5.0	15	0.333		
10D	35.0	108.8	SAND (Fill)	5.0	84	0.060		
11D	40.0	103.8	SAND (Fill)	5.0	36	0.139		
12D	45.0	98.8	SILT/CLAY (Marine Deposit)	5.0			4000	0.001
13D	50.0	93.8	SILT/CLAY (Marine Deposit)	5.0			4000	0.001
14D	55.0	88.8	SILT/CLAY (Marine Deposit)	5.0			4000	0.001
15D	60.0	83.8	SILT/CLAY (Marine Deposit)	5.0			4000	0.001
16D/R1	65.0	78.8	SILT (Glacial Till)	1.6			4000	0.000
17D/R2	71.7	72.1	SILT/SAND (Glacial Till)	5.9	100	0.059		
R2-R3	72.5	71.3	BEDROCK	27.5	100	0.275		
					Σd/N =	1.385	Σd/Su =	0.005

Total Thickness of Cohesionless (ft) =	78.4	$N_{ch\text{-}bar}$ (blows/ft) =	56.6	$S_{u\text{-}bar}$ (psf) =	4000
Total Thickness of Cohesive (ft) =	21.6	Site $Class_N$ =	C	Site $Class_{Su}$ =	C
Total Thickness (ft) =	100.0	**Site Class =**	**C**		

HALEY ALDRICH	CALCULATIONS	File No.	132076-005
		Sheet	8 of 9
Client	Maine Department of Transportation	Date	15-Mar-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by	NAS
Subject	Seismic Site Class Evaluation	Checked by	BCS

CALCULATIONS - METHOD C

Exploration ID: BB-BWS-203
Ground Surface El.: 111.2

					Cohesionless		Cohesive	
Sample Number	**Depth (ft)**	**Elevation (ft)**	**Description**	**d (ft)**	**SPT N (blows/ft)**	**d/N**	**Su (psf)**	**d/Su**
1D	1.0	110.2	GRAVEL (Fill)	2.0	12	0.167		
2D	2.0	109.2	GRAVEL (Fill)	1.5	61	0.025		
3D	4.0	107.2	SILT (Fill)	2.5			3000	0.001
4D	6.0	105.2	SILT (Fill)	2.0			3000	0.001
5D	8.0	103.2	CLAY (Fill)	1.1			2000	0.001
5D	9.1	102.1	CLAY (Marine Deposit)	5.9			2000	0.003
6D	15.0	96.2	SILT (Marine Deposit)	5.0			4000	0.001
7D	20.0	91.2	SILT (Marine Deposit)	5.0			4000	0.001
8D	25.0	86.2	SILT (Marine Deposit)	5.0			4000	0.001
9D	30.0	81.2	SILT (Marine Deposit)	5.0			4000	0.001
10D	35.0	76.2	SILT (Marine Deposit)	1.6			4000	0.000
11D	40.0	71.2	GRAVEL (Glacial Till)	7.9	100	0.079		
R1-R3	44.5	66.7	BEDROCK	55.5	100	0.555		
					Σd/N =	0.825	Σd/Su =	0.010
Total Thickness of Cohesionless (ft) =			66.9		$N_{ch\text{-}bar}$ (blows/ft) =	81.1	$S_{u\text{-}bar}$ (psf) =	3183
Total Thickness of Cohesive (ft) =			33.1		Site Class$_N$ =	C	Site Class$_{Su}$ =	C
Total Thickness (ft) =			100.0		**Site Class =**	**C**		

CALCULATIONS - METHOD C

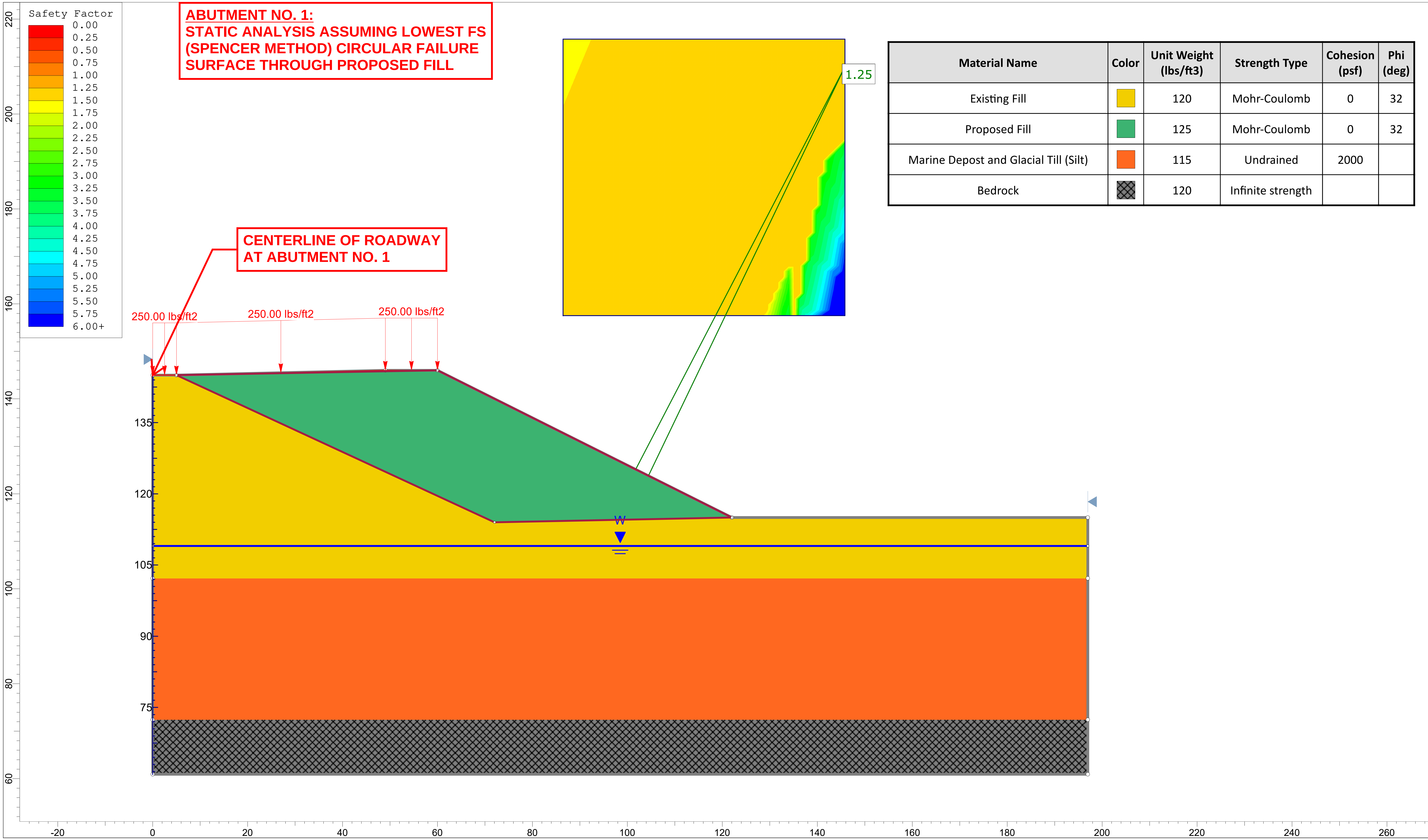
Exploration ID: BB-BWS-206

Ground Surface El.: 134.6

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless SPT N (blows/ft)	Cohesionless d/N	Cohesive Su (psf)	Cohesive d/Su
1D	0.0	134.6	SAND (Fill)	2.0	112	0.018		
2D	2.0	132.6	SILT (Fill)	2.0			4000	0.001
3D	4.0	130.6	SILT (Fill)	5.0			4000	0.001
4D	9.0	125.6	SILT (Fill)	5.0			4000	0.001
5D	14.0	120.6	SILT (Fill)	5.0			4000	0.001
6D	19.0	115.6	SILT (Fill)	5.0			4000	0.001
7D	24.0	110.6	SILT (Fill)	4.7			4000	0.001
8D	28.7	105.9	GRAVEL (Fill)	5.3	100	0.053		
9D	34.0	100.6	SILT (Fill)	5.0			2000	0.003
10D	39.0	95.6	GRAVEL (Fill)	5.0	63	0.079		
11D	44.0	90.6	GRAVEL (Fill)	5.2	49	0.106		
12D	49.2	85.4	CLAY (Marine Deposit)	4.8			2000	0.002
13D	54.0	80.6	SILT (Glacial Till)	5.0			4000	0.001
14D	59.0	75.6	CLAY (Glacial Till)	5.0			4000	0.001
15D	64.0	70.6	SILT (Glacial Till)	5.0			4000	0.001
16D	69.0	65.6	SILT (Glacial Till)	5.0			4000	0.001
17D	74.0	60.6	SILT (Glacial Till)	5.0			4000	0.001
R1-R3	79.0	55.6	BEDROCK	21.0	100	0.210		
					Σd/N =	0.466	Σd/Su =	0.018

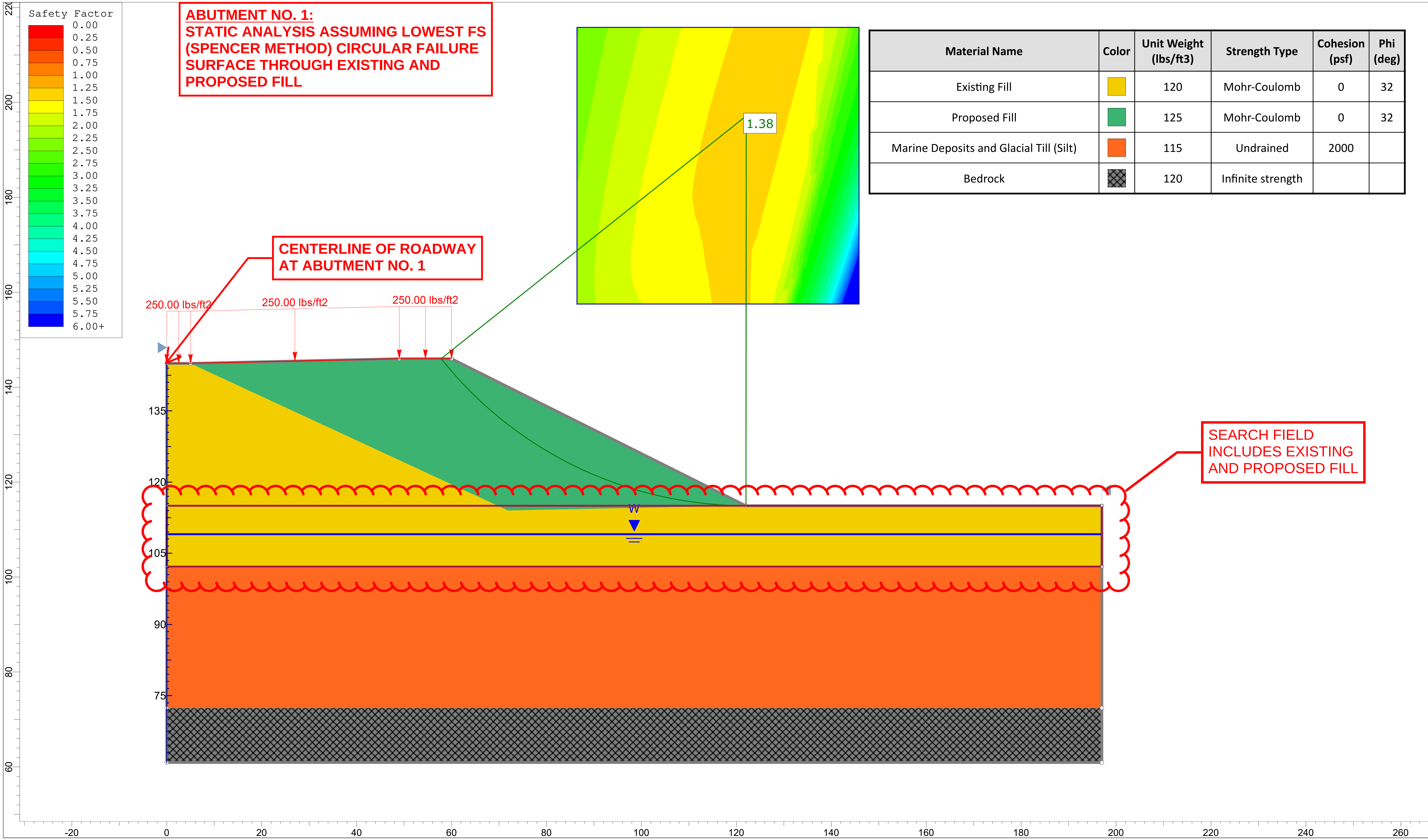
Total Thickness of Cohesionless (ft) =	38.5	$N_{ch\text{-}bar}$ (blows/ft) =	82.6	$S_{u\text{-}bar}$ (psf) =	3450
Total Thickness of Cohesive (ft) =	61.5	Site $Class_N$ =	C	Site $Class_{Su}$ =	C
Total Thickness (ft) =	100.0	**Site Class =**	**C**		

Global Embankment Stability



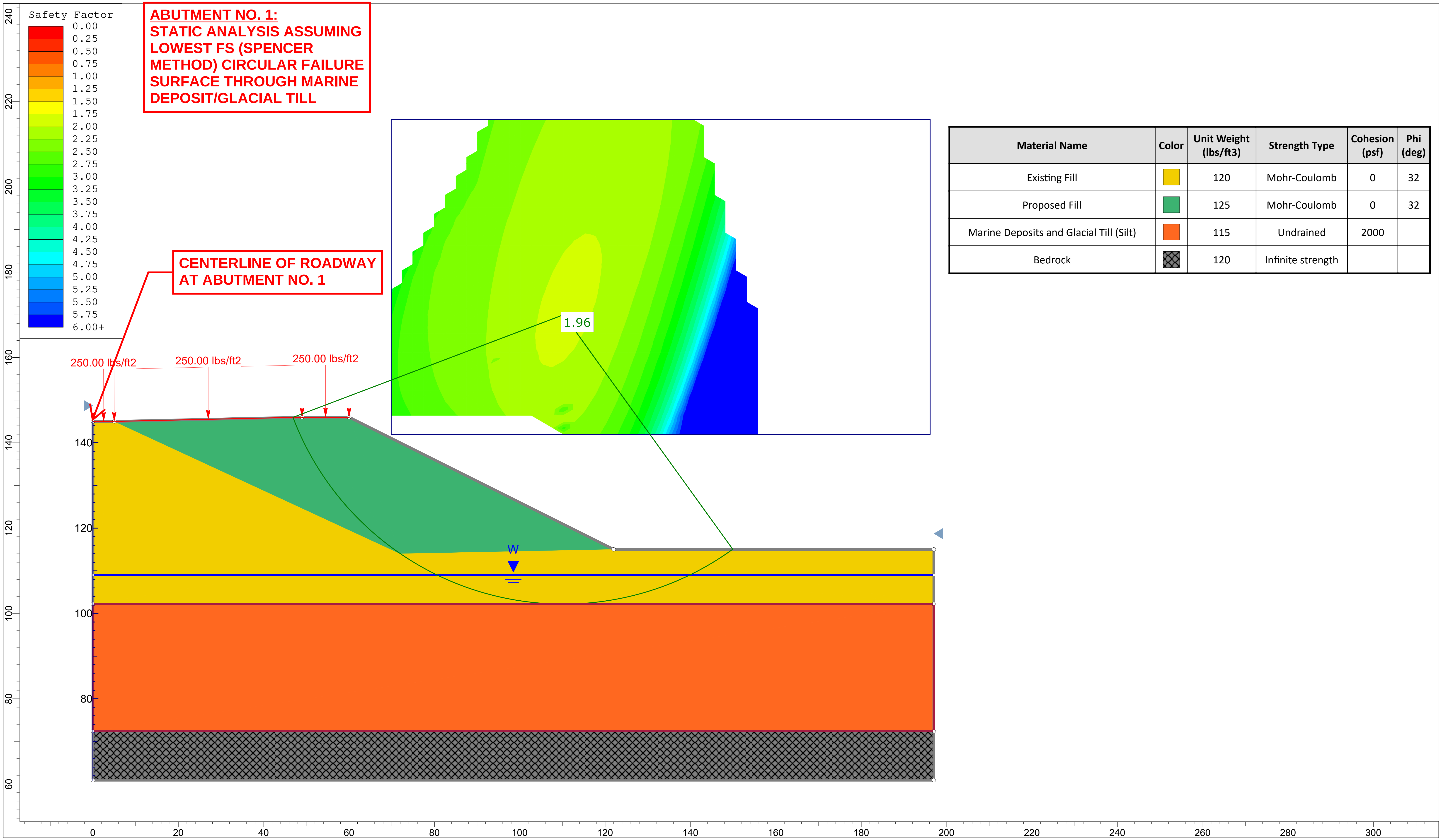
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Depost and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		

2020-0312-HAI-Wilson Street-Revised Static-Abut 1-Prop Fill-D1.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Deposits and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		

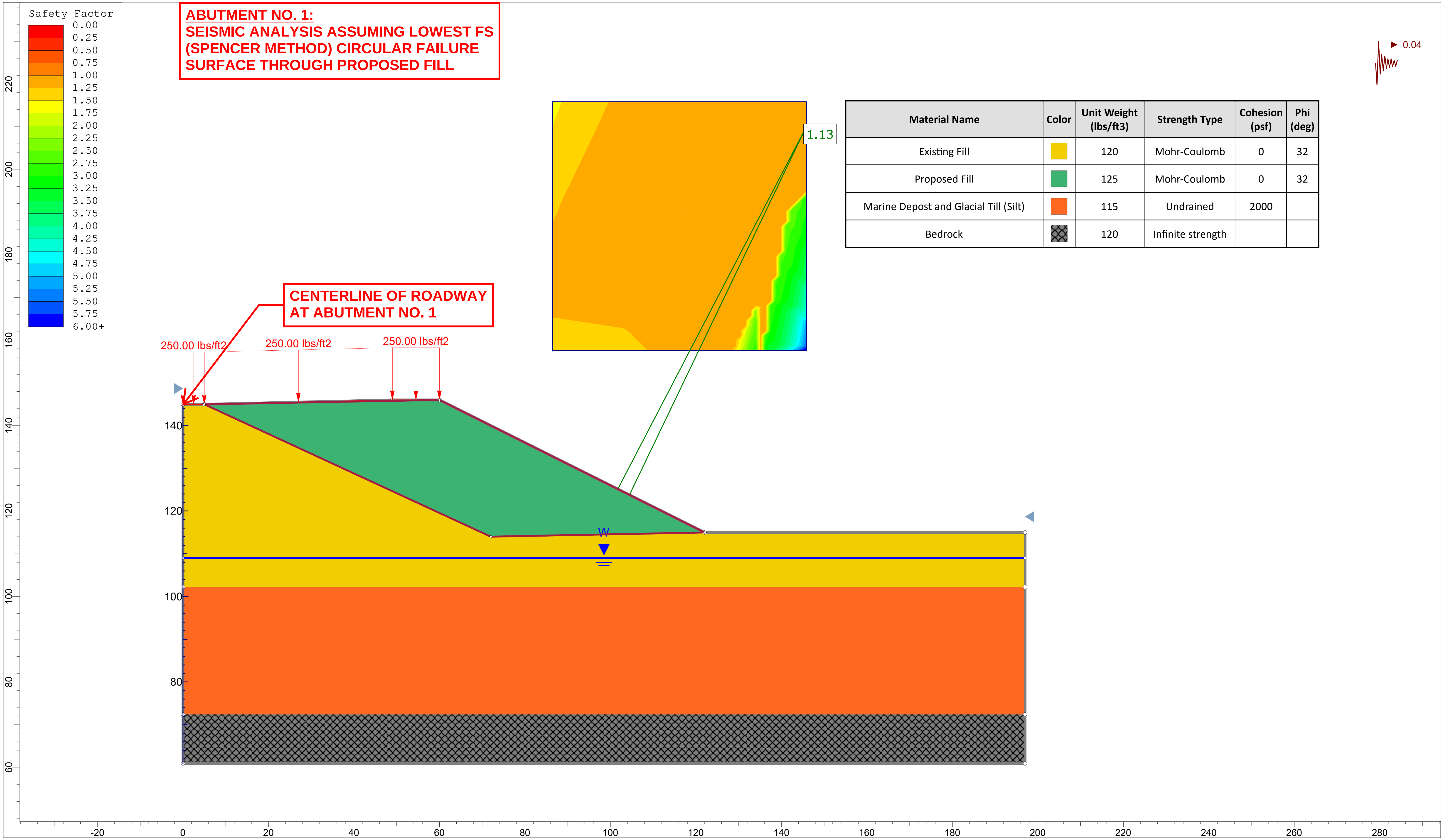
2020-0312-HAI-Wilson Street-Revised Static-Abut 1-Ex Fill-D1.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Deposits and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		

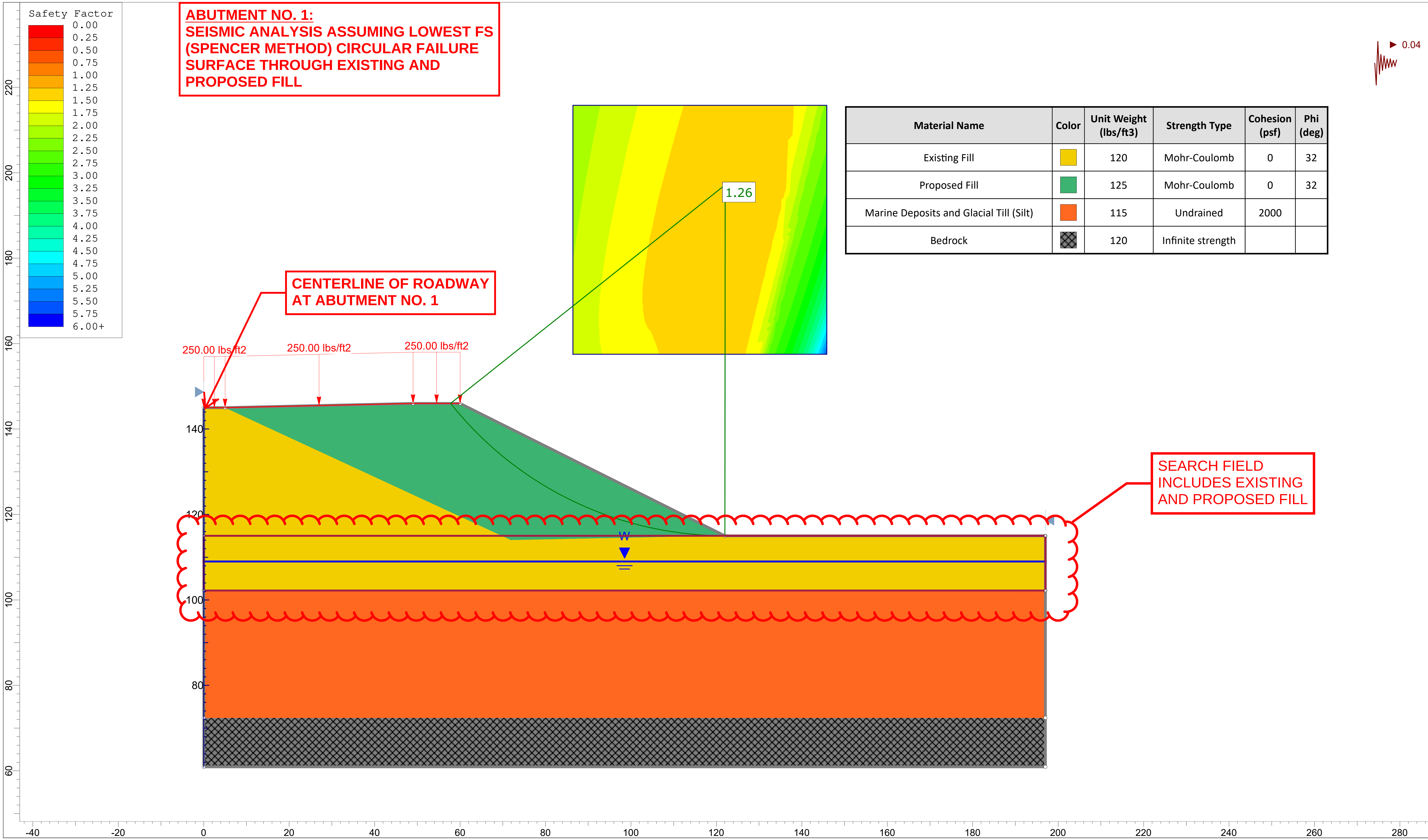
2020-0312-HAI-Wilson Street-Revised Static-Abut 1-MarineTill-D1.slim

Haley & Aldrich, Inc. 10/30/19

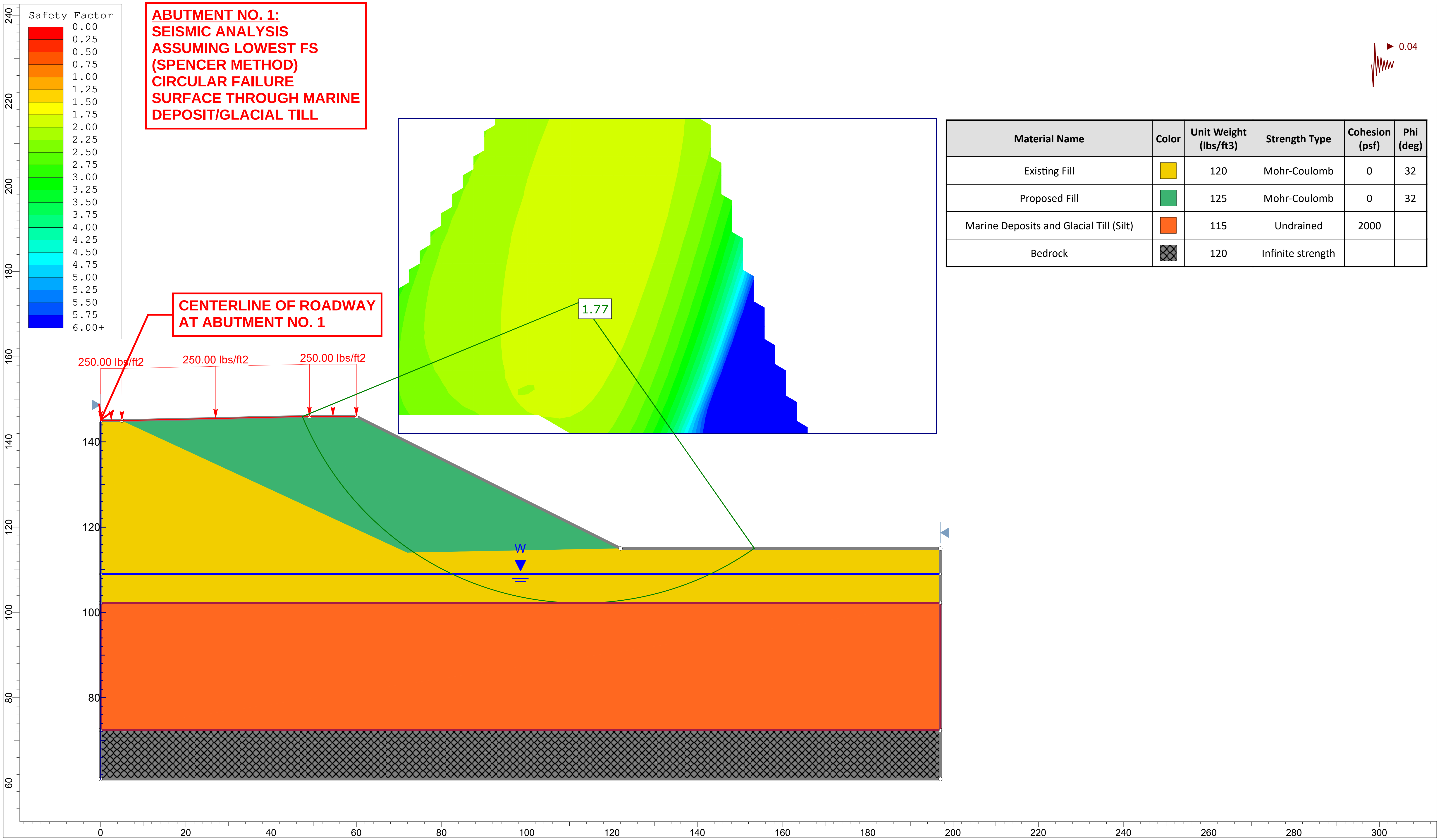


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Depost and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		

2020-0312-HAI-Wilson Street-Revised Seismic-Abut 1-Prop Fill-D1.slim

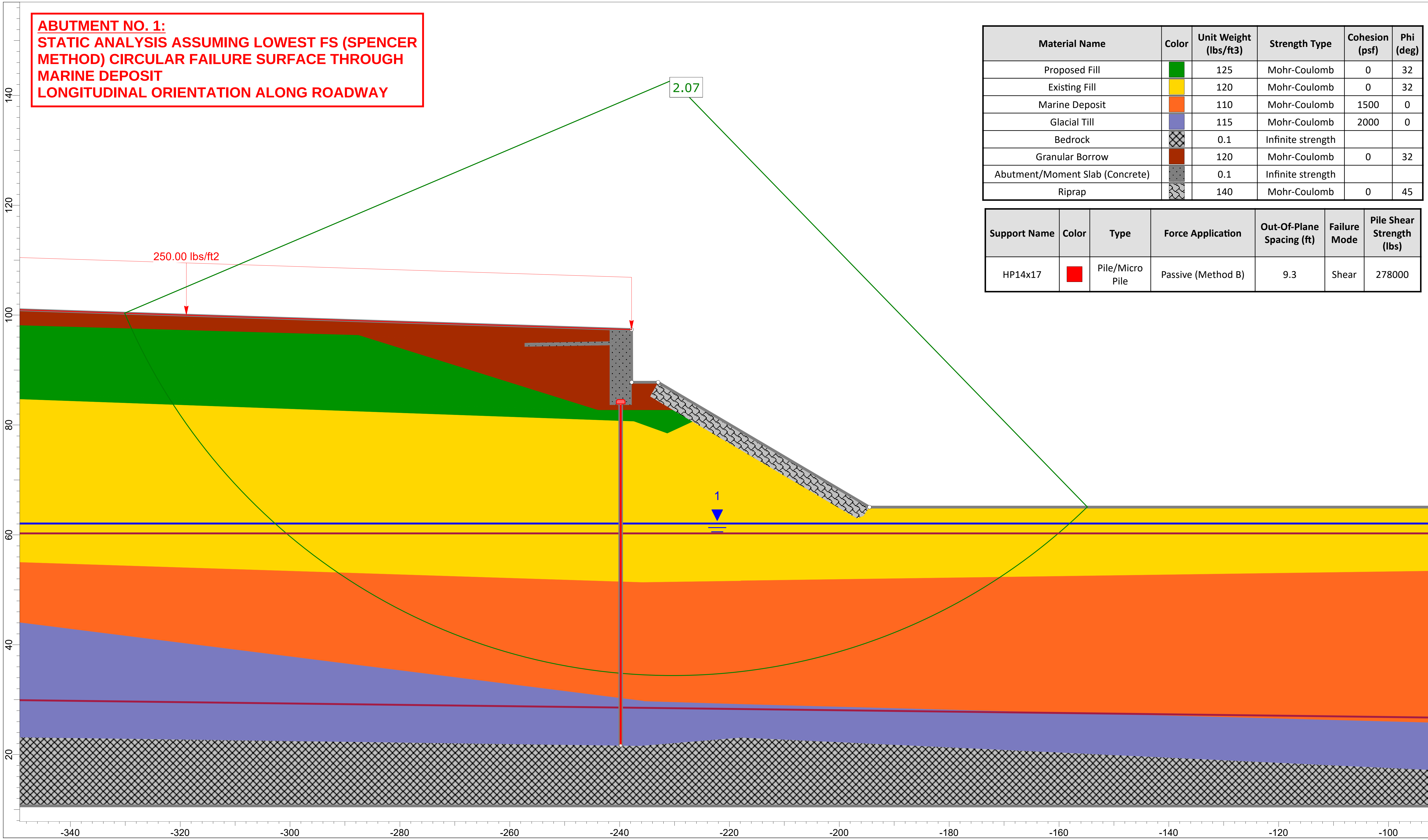


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Deposits and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	32
Proposed Fill		125	Mohr-Coulomb	0	32
Marine Deposits and Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		

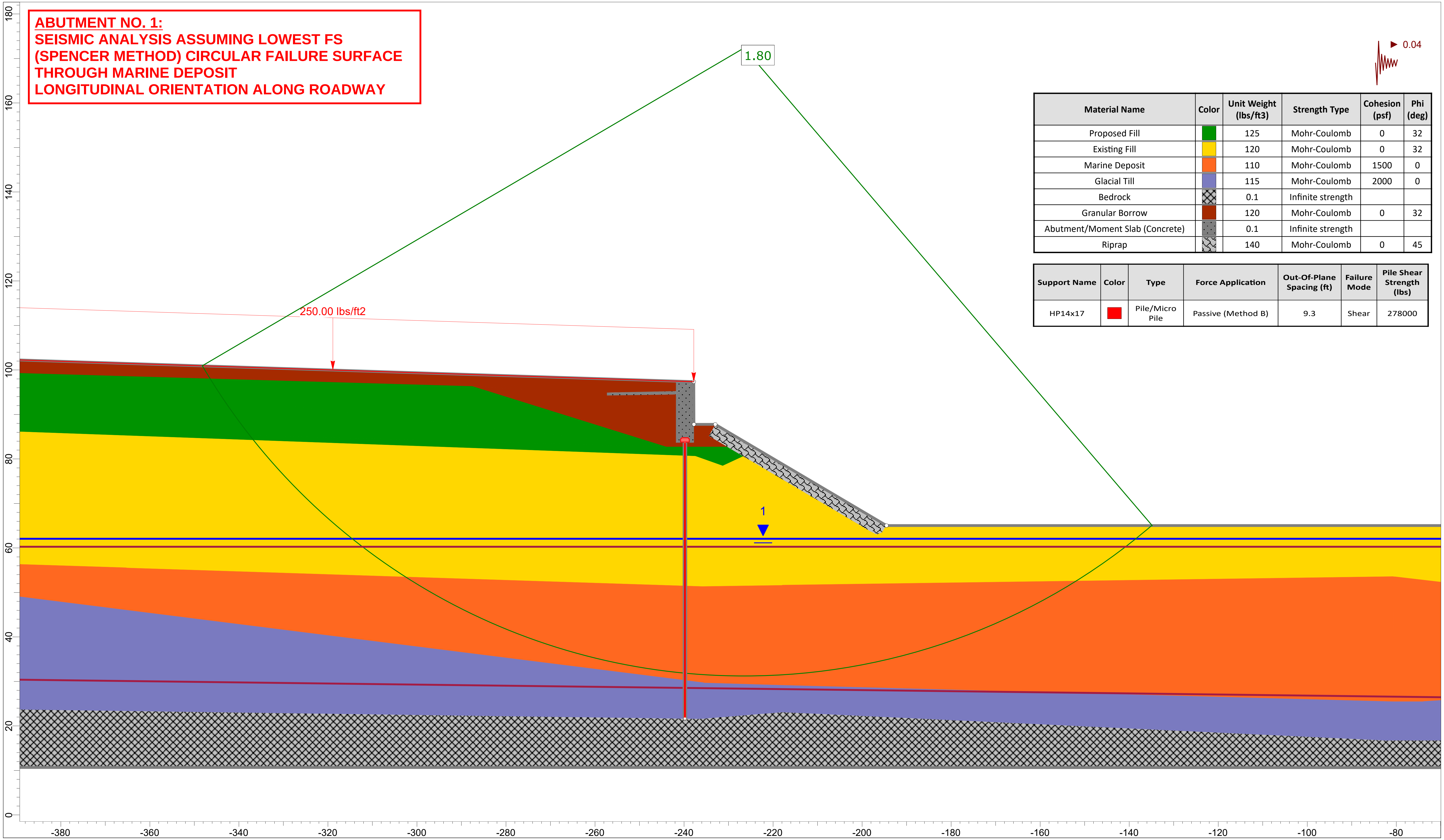
ABUTMENT NO. 1:
STATIC ANALYSIS ASSUMING LOWEST FS (SPENCER METHOD) CIRCULAR FAILURE SURFACE THROUGH MARINE DEPOSIT
LONGITUDINAL ORIENTATION ALONG ROADWAY



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Fill		125	Mohr-Coulomb	0	32
Existing Fill		120	Mohr-Coulomb	0	32
Marine Deposit		110	Mohr-Coulomb	1500	0
Glacial Till		115	Mohr-Coulomb	2000	0
Bedrock		0.1	Infinite strength		
Granular Borrow		120	Mohr-Coulomb	0	32
Abutment/Moment Slab (Concrete)		0.1	Infinite strength		
Riprap		140	Mohr-Coulomb	0	45

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)
HP14x17		Pile/Micro Pile	Passive (Method B)	9.3	Shear	278000

2020-0410-HAI-Wilson Street-Longitudinal Static-Abut 1-D4.slim 4/9/2020, 5:05:31 PM



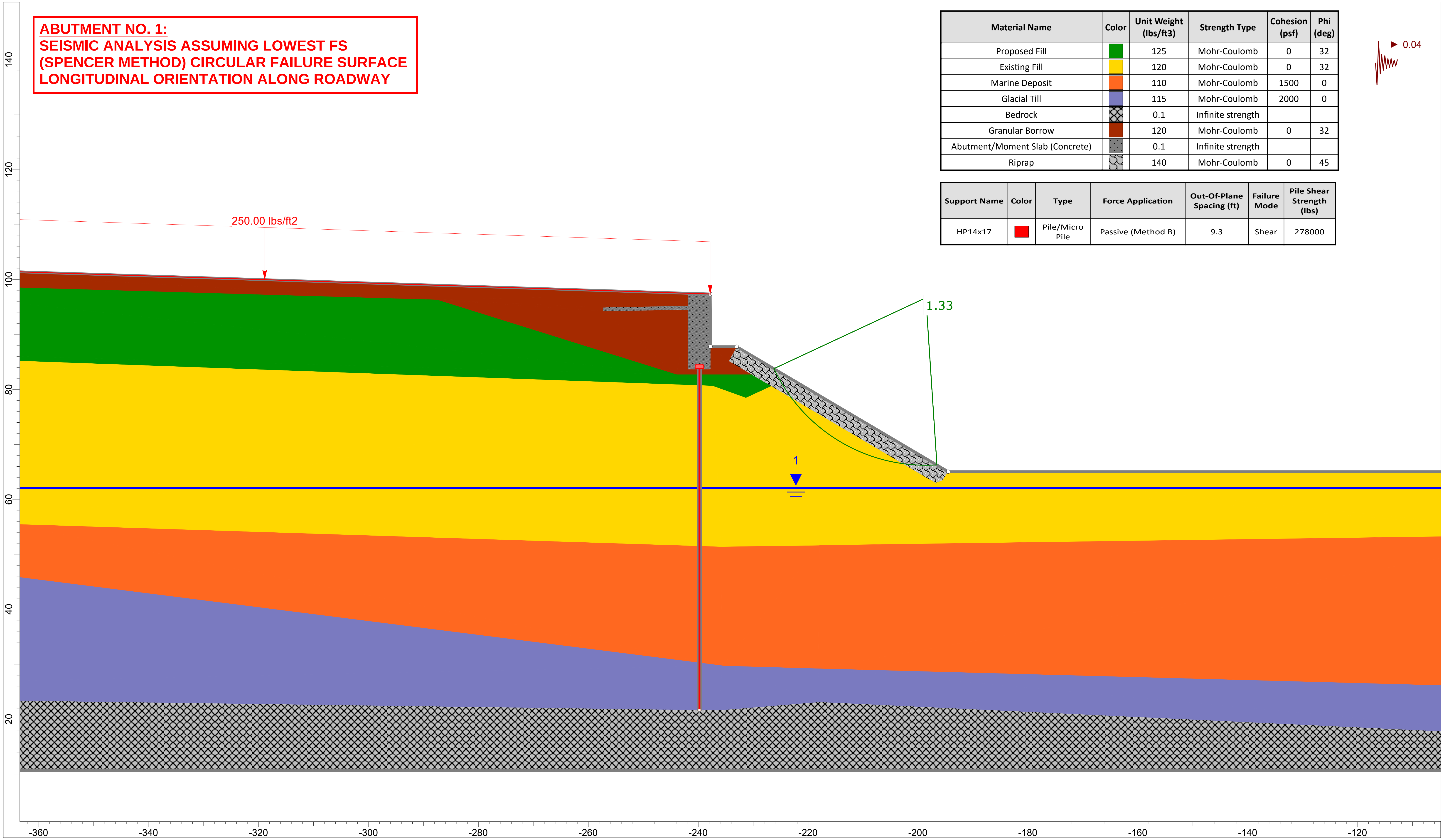
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Fill		125	Mohr-Coulomb	0	32
Existing Fill		120	Mohr-Coulomb	0	32
Marine Deposit		110	Mohr-Coulomb	1500	0
Glacial Till		115	Mohr-Coulomb	2000	0
Bedrock		0.1	Infinite strength		
Granular Borrow		120	Mohr-Coulomb	0	32
Abutment/Moment Slab (Concrete)		0.1	Infinite strength		
Riprap		140	Mohr-Coulomb	0	45

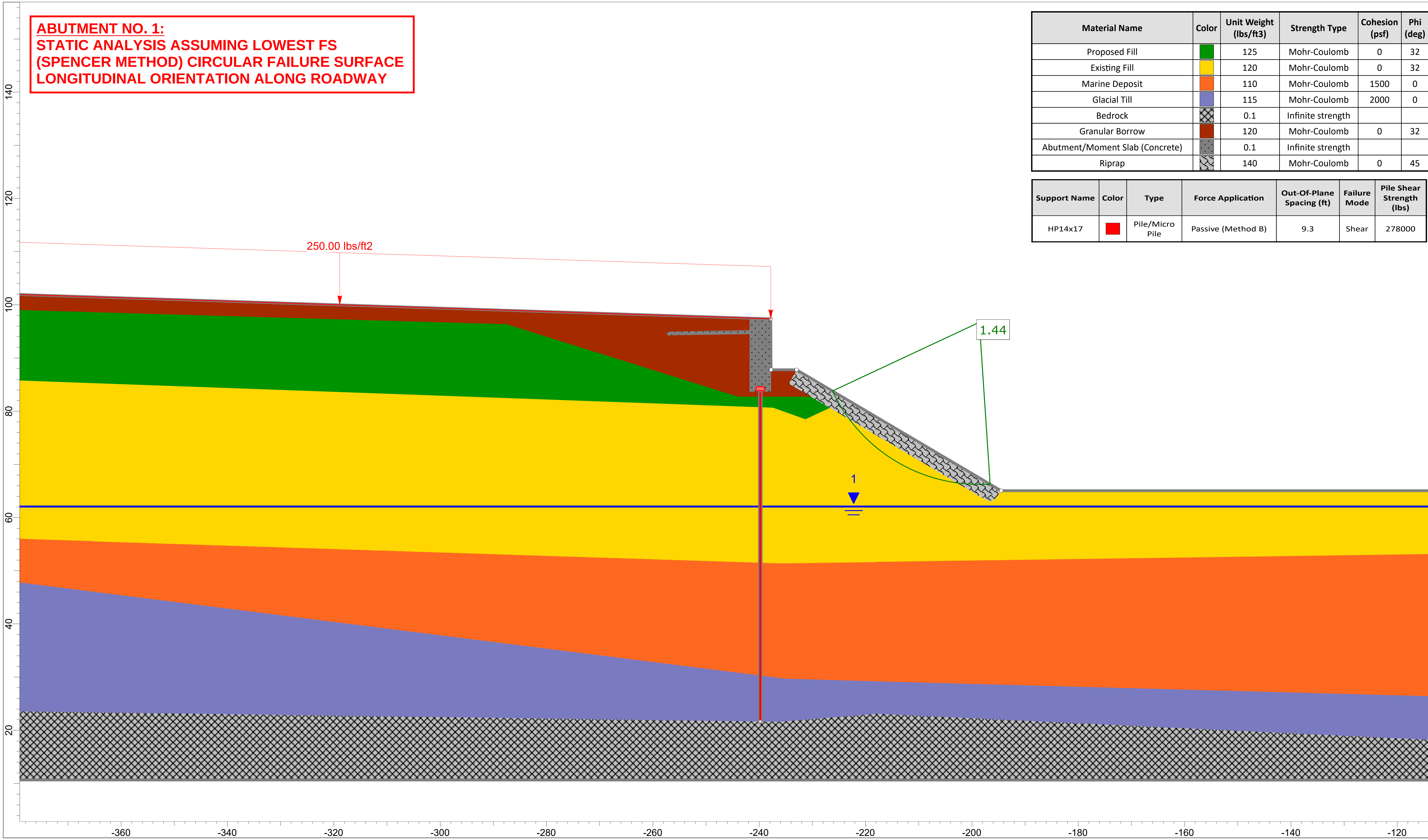
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)
HP14x17		Pile/Micro Pile	Passive (Method B)	9.3	Shear	278000

ABUTMENT NO. 1:
SEISMIC ANALYSIS ASSUMING LOWEST FS (SPENCER METHOD) CIRCULAR FAILURE SURFACE LONGITUDINAL ORIENTATION ALONG ROADWAY

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Fill		125	Mohr-Coulomb	0	32
Existing Fill		120	Mohr-Coulomb	0	32
Marine Deposit		110	Mohr-Coulomb	1500	0
Glacial Till		115	Mohr-Coulomb	2000	0
Bedrock		0.1	Infinite strength		
Granular Borrow		120	Mohr-Coulomb	0	32
Abutment/Moment Slab (Concrete)		0.1	Infinite strength		
Riprap		140	Mohr-Coulomb	0	45

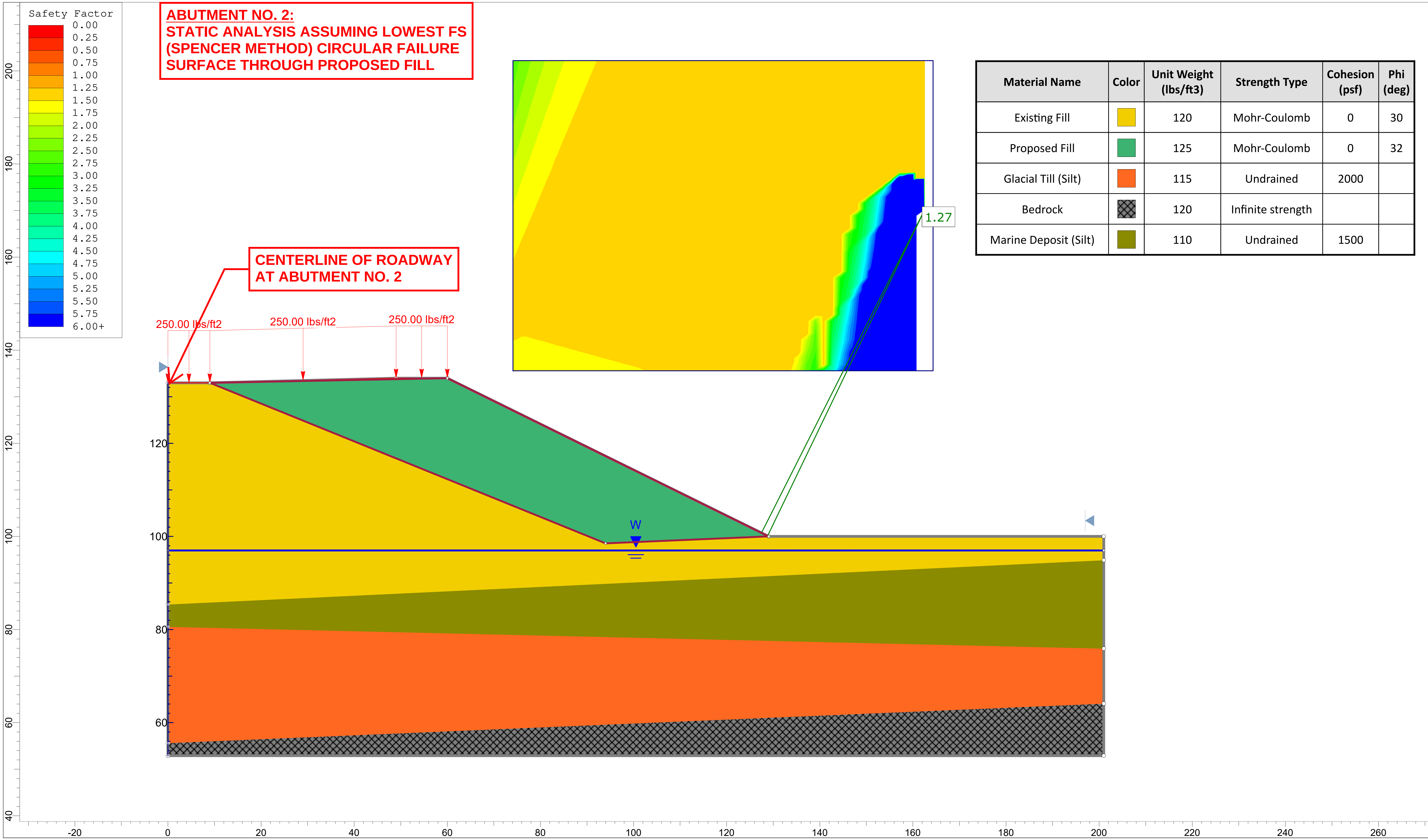
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)
HP14x17		Pile/Micro Pile	Passive (Method B)	9.3	Shear	278000





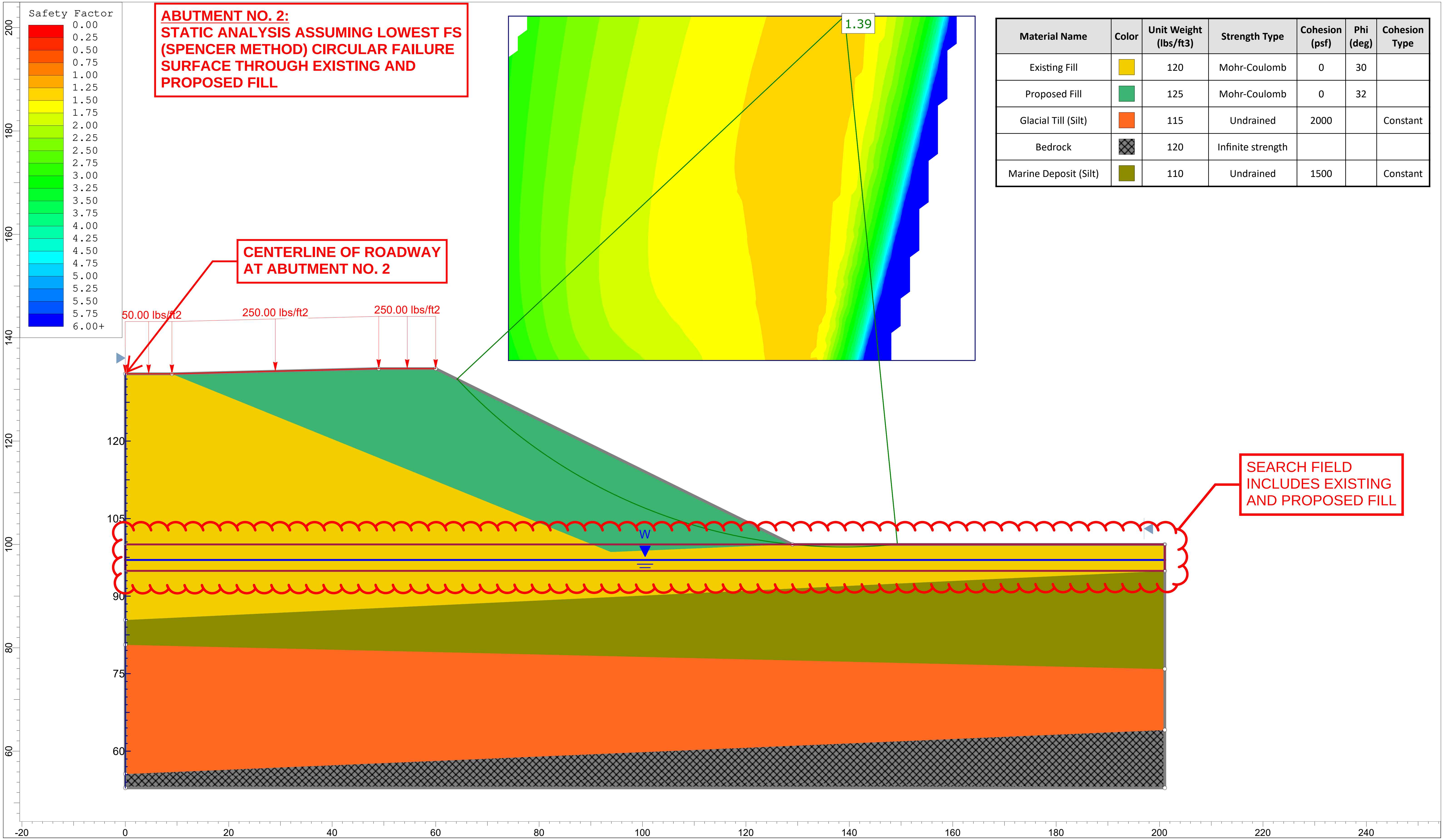
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Fill		125	Mohr-Coulomb	0	32
Existing Fill		120	Mohr-Coulomb	0	32
Marine Deposit		110	Mohr-Coulomb	1500	0
Glacial Till		115	Mohr-Coulomb	2000	0
Bedrock		0.1	Infinite strength		
Granular Borrow		120	Mohr-Coulomb	0	32
Abutment/Moment Slab (Concrete)		0.1	Infinite strength		
Riprap		140	Mohr-Coulomb	0	45

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)
HP14x17		Pile/Micro Pile	Passive (Method B)	9.3	Shear	278000

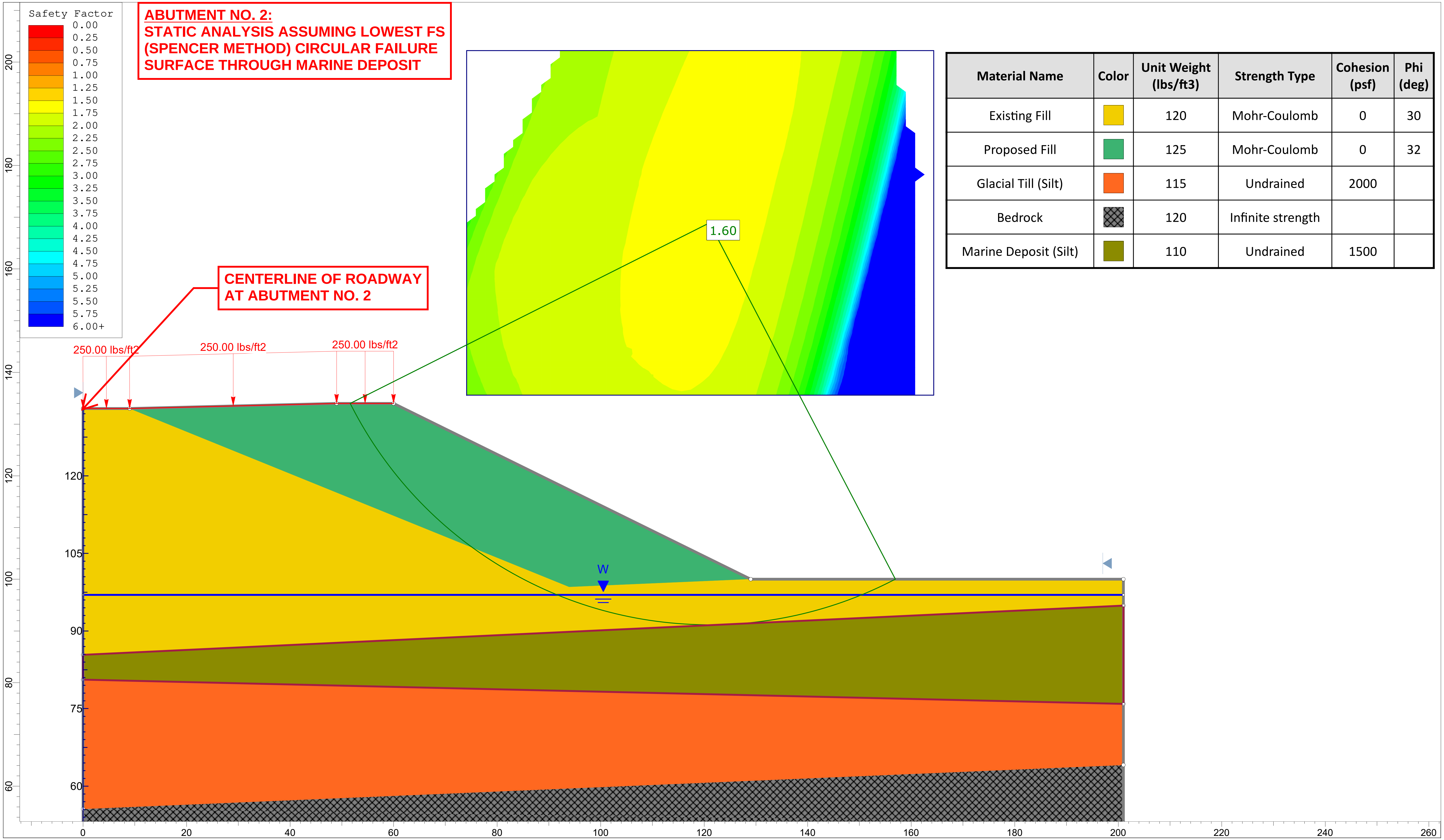


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	30
Proposed Fill		125	Mohr-Coulomb	0	32
Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		
Marine Deposit (Silt)		110	Undrained	1500	

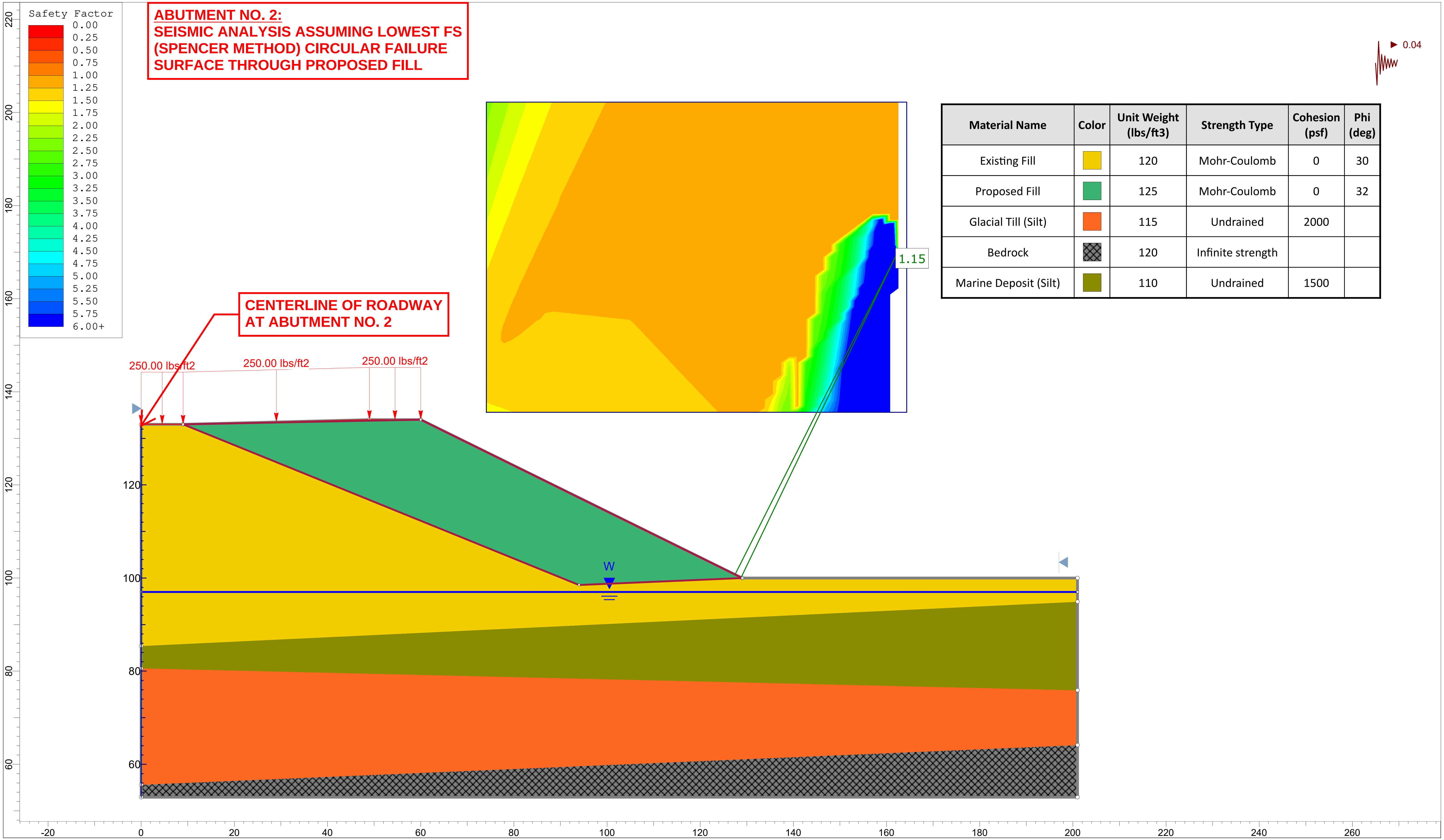
2020-0312-HAI-Wilson Street-Rev Static-Abut 2-Prop Fill-D1.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type
Existing Fill		120	Mohr-Coulomb	0	30	
Proposed Fill		125	Mohr-Coulomb	0	32	
Glacial Till (Silt)		115	Undrained	2000		Constant
Bedrock		120	Infinite strength			
Marine Deposit (Silt)		110	Undrained	1500		Constant



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	30
Proposed Fill		125	Mohr-Coulomb	0	32
Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		
Marine Deposit (Silt)		110	Undrained	1500	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	30
Proposed Fill		125	Mohr-Coulomb	0	32
Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		
Marine Deposit (Silt)		110	Undrained	1500	

2020-0312-HAI-Wilson Street-Rev Seismic-Abut 2-Prop Fill-D1.slim

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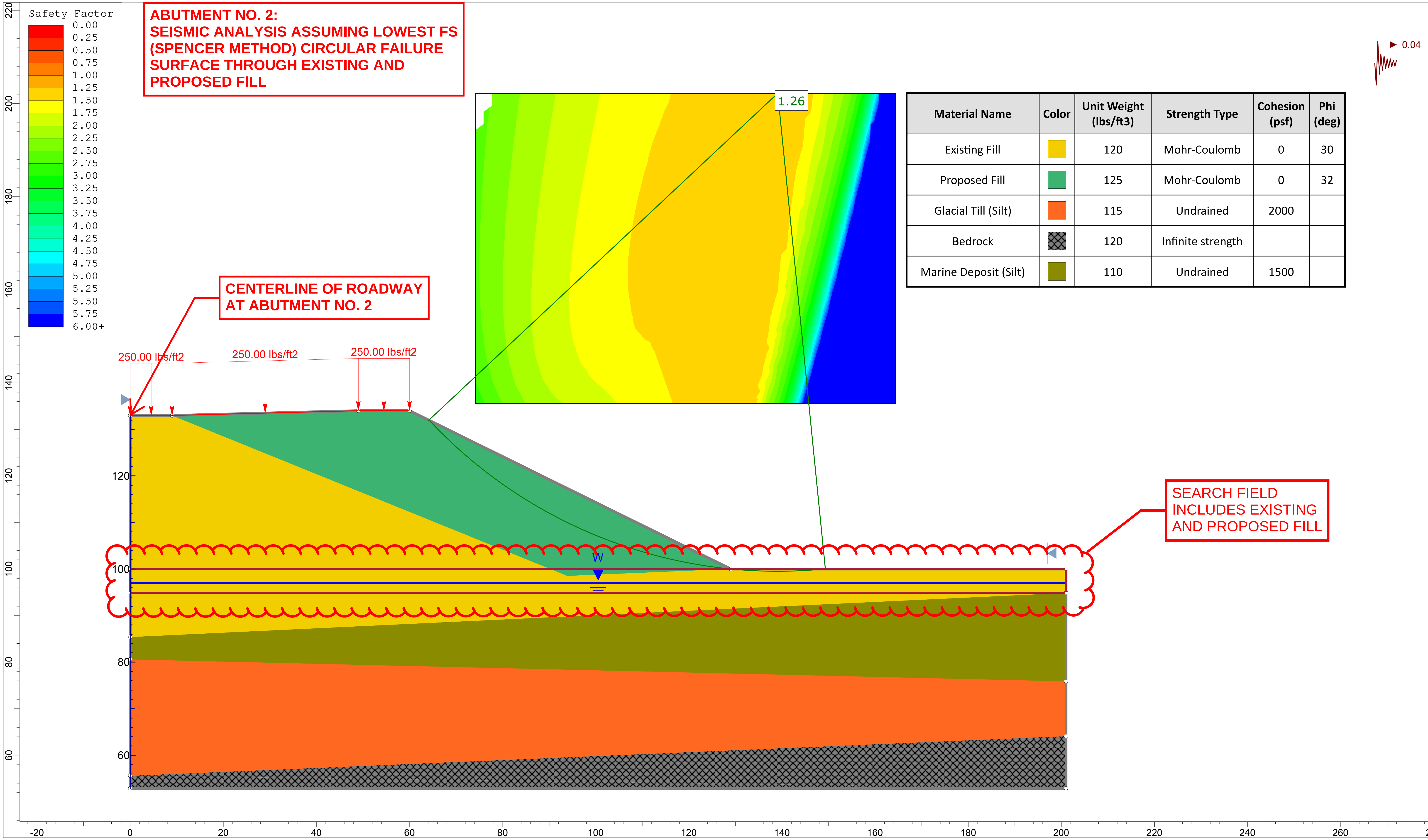
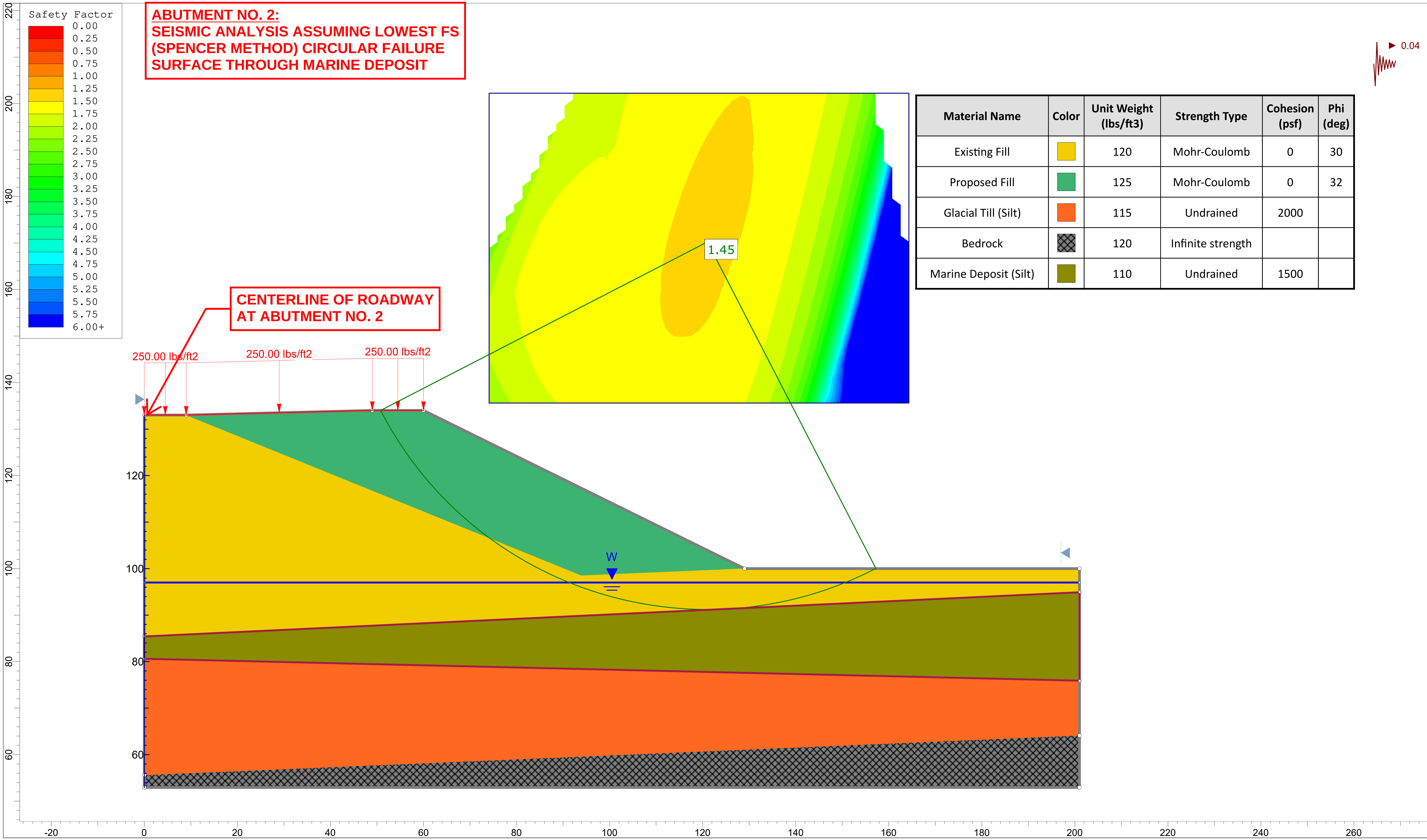


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	30
Proposed Fill		125	Mohr-Coulomb	0	32
Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		
Marine Deposit (Silt)		110	Undrained	1500	

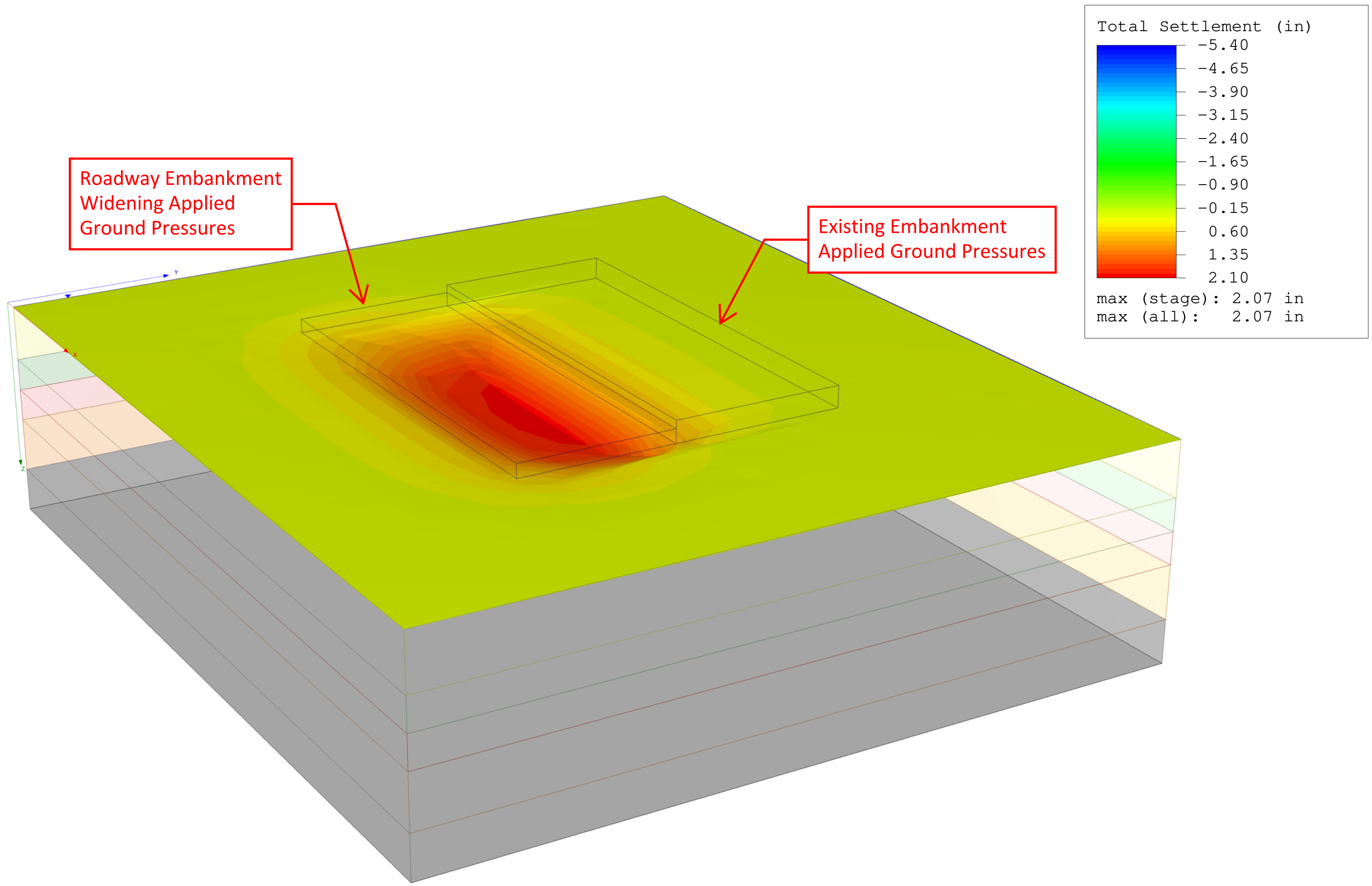
2020-0312-HAI-Wilson Street-Rev Seismic-Abut 2-Ex Fill-D1.slim

Haley & Aldrich, Inc. 10/30/19



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing Fill		120	Mohr-Coulomb	0	30
Proposed Fill		125	Mohr-Coulomb	0	32
Glacial Till (Silt)		115	Undrained	2000	
Bedrock		120	Infinite strength		
Marine Deposit (Silt)		110	Undrained	1500	

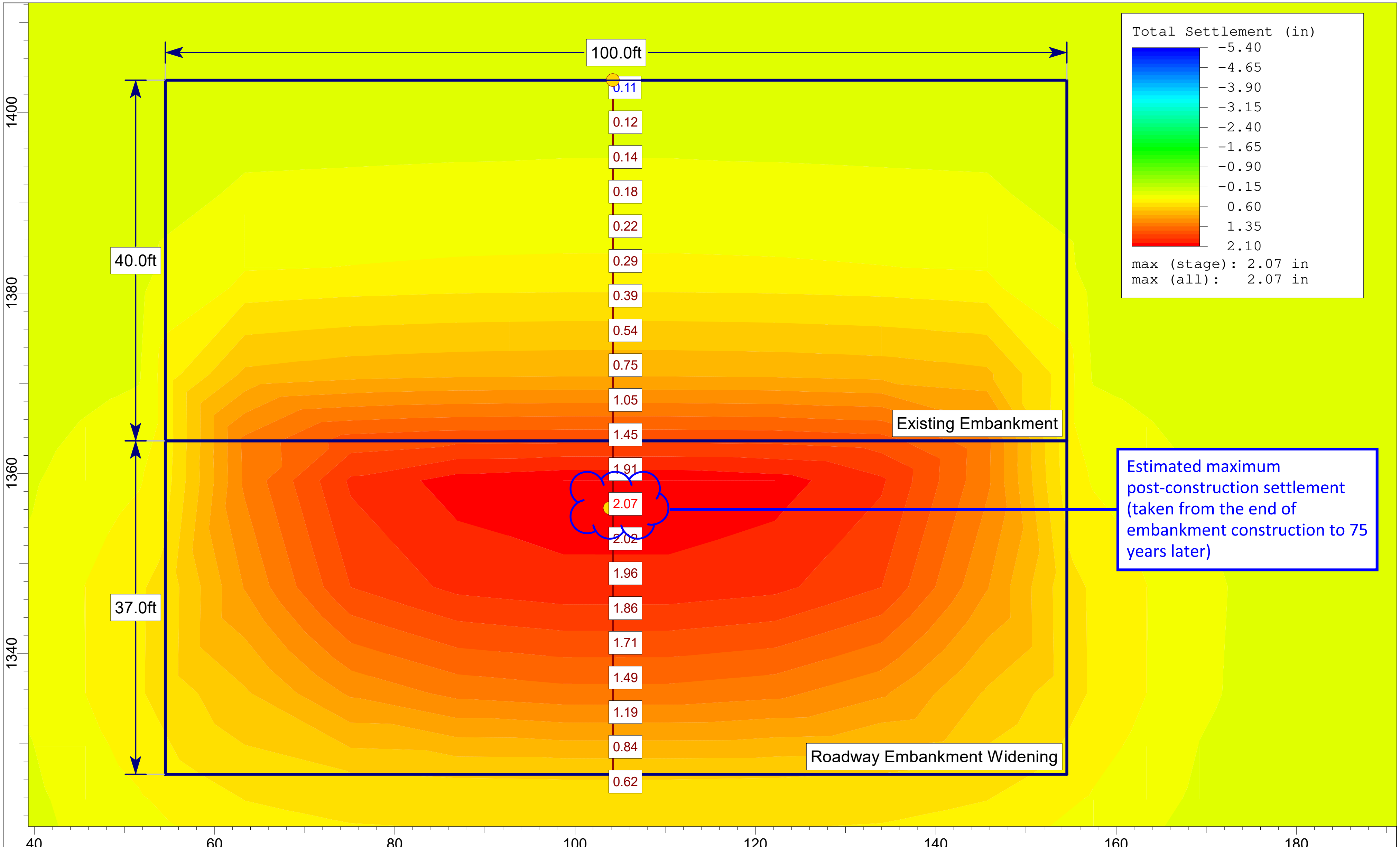
Roadway Embankment Settlement



rocscience

SETTLE3D 4.023

Project	Wilson Street Bridge Replacement - WIN No. 018915.00
Analysis Description	Settlement Due to Embankment Widening
Drawn By	Sherwood
Company	Haley & Aldrich
Date	4/10/20
File Name	2020-0410-HAI-Wilson Street-Embankment Settlement-D3.s3z

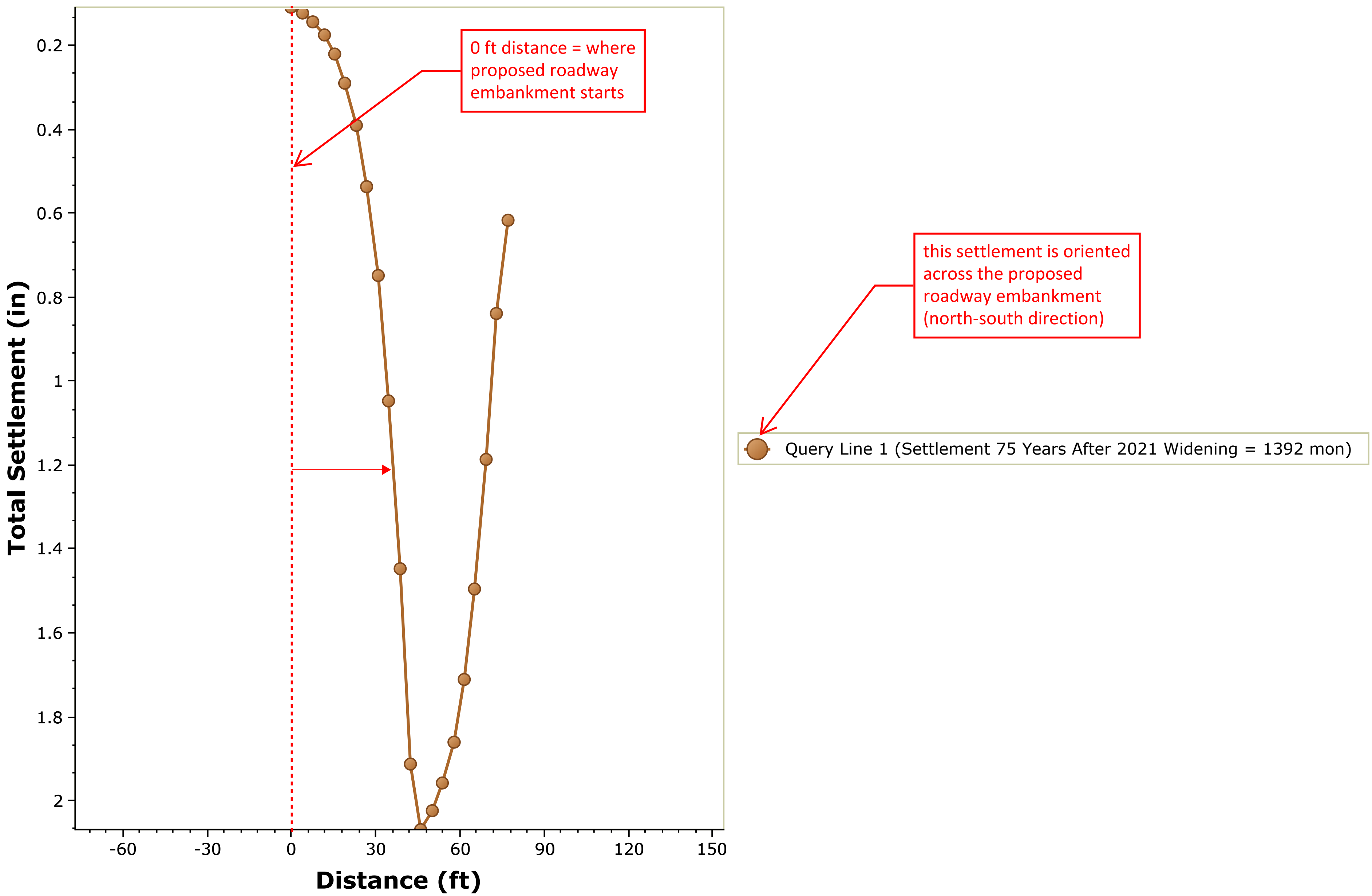


rocscience

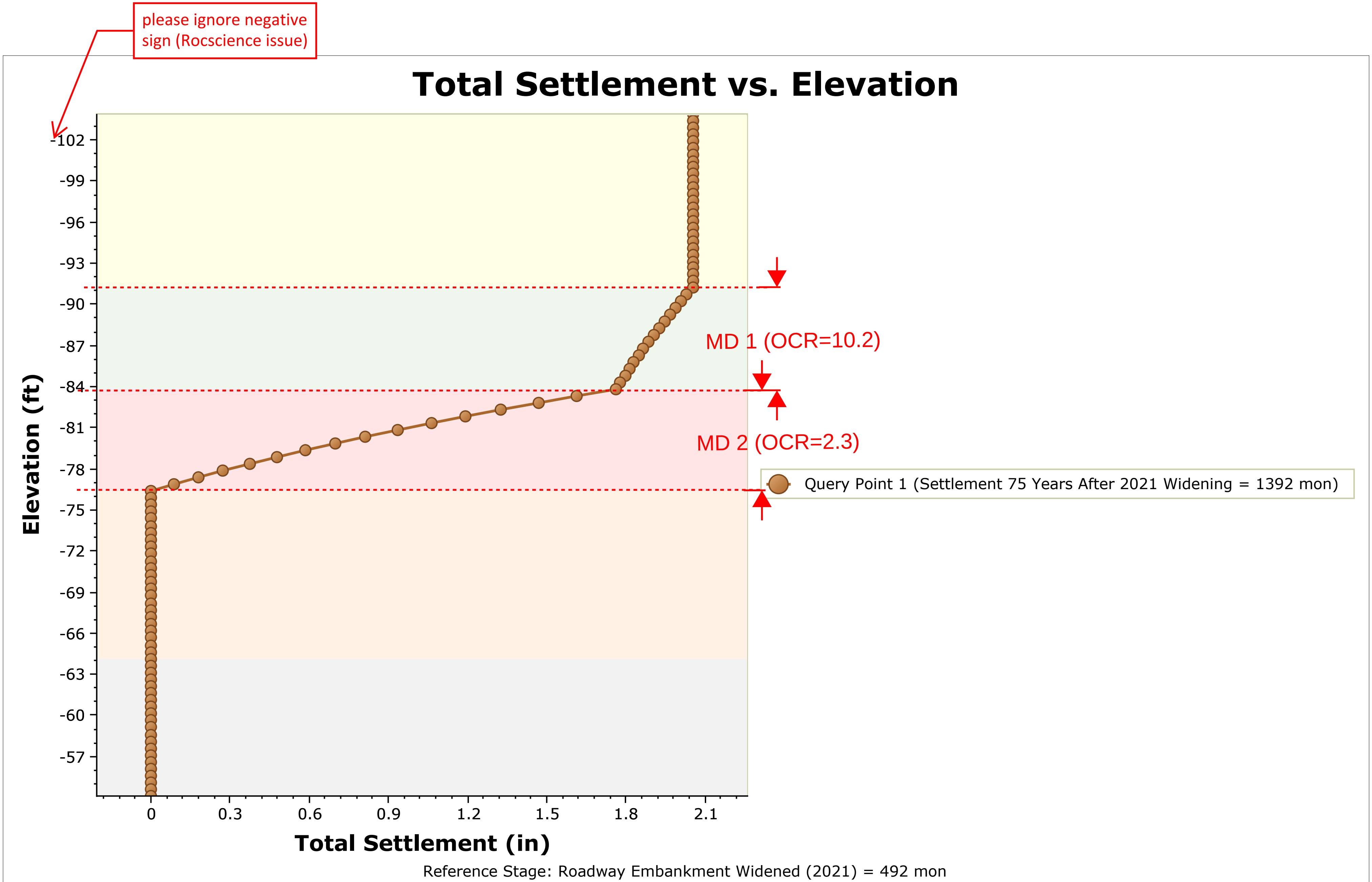
SETTLE3D 4.023

Project	Wilson Street Bridge Replacement - WIN No. 018915.00		
Analysis Description	Settlement Due to Embankment Widening		
Drawn By	Sherwood	*Company*	Haley & Aldrich
Date	4/10/20	*File Name*	2020-0410-HAI-Wilson Street-Embankment Settlement-D3.s3z

Distance vs. Total Settlement



Reference Stage: Roadway Embankment Widened (2021) = 492 mon
Total Settlement at Depth = -103.9 ft



rocscience	Project	Wilson Street Bridge Replacement - WIN No. 018915.00	
	Analysis Description	Settlement Due to Embankment Widening	
	Drawn By	Sherwood	Company: Haley & Aldrich
SETTLE3D 4.023	Date	4/10/20	File Name: 2020-0410-HAI-Wilson Street-Embankment Settlement-D3.s3z

Settle3D Analysis Information

Wilson Street Bridge Replacement - WIN No. 018915.00

Project Settings

Document Name	2020-0410-HAI-Wilson Street-Embankment Settlement-D3.s3z
Project Title	Wilson Street Bridge Replacement - WIN No. 018915.00
Analysis	Settlement Due to Embankment Widening
Author	Sherwood
Company	Haley & Aldrich
Date Created	4/10/20

Comments

Assume grade raise of 27 ft (Abutment No. 1)
Assume subsurface conditions based on Abutment No. 2

Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	months
Permeability Units	feet/day
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name	Time [months]
1	Existing Conditions	0
2	Existing Embankment Constructed (1980s)	1
3	Estimated Settlement of Existing Embankment (2020)	480
4	Roadway Embankment Widened (2021)	492
5	Settlement 75 Years After 2021 Widening	1392

Results (relative to Stage: Roadway Embankment Widened (2021) = 492 mon)

Time taken to compute: 6.25244 seconds

Stage: Existing Conditions = 0 mon

Data Type	Minimum	Maximum
Total Settlement [in]	-5.35575	0
Total Consolidation Settlement [in]	-2.97977	0
Virgin Consolidation Settlement [in]	-2.01968	0
Recompression Consolidation Settlement [in]	-0.960088	0
Immediate Settlement [in]	-2.38479	0
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-4.40024	0
Loading Stress XX [ksf]	-3.07762	0.978212
Loading Stress YY [ksf]	-3.62475	0.73856
Effective Stress ZZ [ksf]	-4.4	0
Effective Stress XX [ksf]	-7.47762	0.978212
Effective Stress YY [ksf]	-8.02475	0.73856
Total Stress ZZ [ksf]	-4.40024	0
Total Stress XX [ksf]	-7.47762	0.978212
Total Stress YY [ksf]	-8.02475	0.73856
Modulus of Subgrade Reaction (Total) [ksf/ft]	-26.9668	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-28.9785	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-458.353	0
Total Strain	-0.034412	0
Pore Water Pressure [ksf]	-2.8	0
Excess Pore Water Pressure [ksf]	-2.8	0
Degree of Consolidation [%]	-98.7441	0
Pre-consolidation Stress [ksf]	-4.4	0
Over-consolidation Ratio	-2.06946e-013	8.6279
Void Ratio	0	0.0722652
Permeability [ft/d]	-0.000721546	0
Coefficient of Consolidation [ft^2/d]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	-0.316753	0

Stage: Existing Embankment Constructed (1980s) = 1 mon

Data Type	Minimum	Maximum
Total Settlement [in]	-3.01566	0
Total Consolidation Settlement [in]	-2.97977	0
Virgin Consolidation Settlement [in]	-2.01968	0
Recompression Consolidation Settlement [in]	-0.960088	0
Immediate Settlement [in]	-1.47788	0
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-2.8	0
Loading Stress XX [ksf]	-1.82636	0.659375
Loading Stress YY [ksf]	-2.4508	0.381345
Effective Stress ZZ [ksf]	-4.4	0
Effective Stress XX [ksf]	-4.8586	0.659375
Effective Stress YY [ksf]	-4.68711	0.381345
Total Stress ZZ [ksf]	-2.8	0
Total Stress XX [ksf]	-4.62636	0.659375
Total Stress YY [ksf]	-5.2508	0.381345
Modulus of Subgrade Reaction (Total) [ksf/ft]	-26.9567	92.8141
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-28.6483	92.8141
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-458.353	0
Total Strain	-0.034412	0
Pore Water Pressure [ksf]	-2.8	4.4
Excess Pore Water Pressure [ksf]	-2.8	4.4
Degree of Consolidation [%]	-98.7441	0
Pre-consolidation Stress [ksf]	-4.4	0
Over-consolidation Ratio	-2.06946e-013	8.6279
Void Ratio	0	0.0722652
Permeability [ft/d]	-0.000721546	0
Coefficient of Consolidation [ft^2/d]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	-98.8319	0
Undrained Shear Strength	-0.316753	0

Stage: Estimated Settlement of Existing Embankment (2020) = 480 mon

Data Type	Minimum	Maximum
Total Settlement [in]	-1.47788	0
Total Consolidation Settlement [in]	-0.000276917	3.40863e-005
Virgin Consolidation Settlement [in]	-0.000284142	0
Recompression Consolidation Settlement [in]	-2.19952e-005	3.71885e-005
Immediate Settlement [in]	-1.47788	0
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-2.8	0
Loading Stress XX [ksf]	-1.82636	0.659375
Loading Stress YY [ksf]	-2.4508	0.381345
Effective Stress ZZ [ksf]	-0.0117123	0.0175798
Effective Stress XX [ksf]	-1.82636	0.659375
Effective Stress YY [ksf]	-2.4508	0.381345
Total Stress ZZ [ksf]	-2.8	0
Total Stress XX [ksf]	-4.62636	0.659375
Total Stress YY [ksf]	-5.2508	0.381345
Modulus of Subgrade Reaction (Total) [ksf/ft]	-26.9644	15.4442
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-28.6483	92.8141
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-458.353	0.00408056
Total Strain	-0.00933333	8.02022e-006
Pore Water Pressure [ksf]	-2.8	0
Excess Pore Water Pressure [ksf]	-2.8	0
Degree of Consolidation [%]	0	92.9873
Pre-consolidation Stress [ksf]	-0.00417844	0
Over-consolidation Ratio	-0.00185028	0.00119467
Void Ratio	-2.04452e-005	0.000143953
Permeability [ft/d]	0	0
Coefficient of Consolidation [ft^2/d]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	-0.000436915	0.000143579

Stage: Roadway Embankment Widened (2021) = 492 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	0
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	0
Effective Stress XX [ksf]	0	0
Effective Stress YY [ksf]	0	0
Total Stress ZZ [ksf]	0	0
Total Stress XX [ksf]	0	0
Total Stress YY [ksf]	0	0
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0
Pore Water Pressure [ksf]	0	0
Excess Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0	0
Over-consolidation Ratio	0	0
Void Ratio	0	0
Permeability [ft/d]	0	0
Coefficient of Consolidation [ft^2/d]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0

Stage: Settlement 75 Years After 2021 Widening = 1392 mon

Data Type	Minimum	Maximum
Total Settlement [in]	-1.38778e-017	2.06936
Total Consolidation Settlement [in]	0	2.0456
Virgin Consolidation Settlement [in]	0	1.80501
Recompression Consolidation Settlement [in]	0	0.695845
Immediate Settlement [in]	-2.22045e-016	2.22045e-016
Secondary Settlement [in]	0	0.0240956
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	2.8
Effective Stress XX [ksf]	0	2.8
Effective Stress YY [ksf]	0	2.8
Total Stress ZZ [ksf]	-3.55271e-015	2.66454e-015
Total Stress XX [ksf]	-3.55271e-015	3.55271e-015
Total Stress YY [ksf]	-3.55271e-015	3.55271e-015
Modulus of Subgrade Reaction (Total) [ksf/ft]	-11.4288	17.2025
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-3.55271e-015	3.55271e-015
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-420.508	20.1059
Total Strain	0	0.0253674
Pore Water Pressure [ksf]	-2.8	0
Excess Pore Water Pressure [ksf]	-2.8	0
Degree of Consolidation [%]	0	92.9873
Pre-consolidation Stress [ksf]	0	2.8
Over-consolidation Ratio	-7.03985	2.81997e-013
Void Ratio	-0.0532715	0
Permeability [ft/d]	0	0.000721546
Coefficient of Consolidation [ft^2/d]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.290235

Loads

1. Rectangular Load: "Existing Embankment"

Length	100 ft
Width	40 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4000 ft^2
Load	4.4 ksf
Depth	-103.9 ft
Installation Stage	Existing Embankment Constructed (1980s) = 1 mon

Based on 35 ft high embankment with 125 pcf fill

Coordinates

X [ft]	Y [ft]
54.527	1363.61
154.527	1363.61
154.527	1403.61
54.527	1403.61

2. Rectangular Load: "Roadway Embankment Widening"

Length	100 ft
Width	37 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	3700 ft^2
Load	2.8 ksf
Depth	-103.9 ft
Installation Stage	Roadway Embankment Widened (2021) = 492 mon

per MaineDOT 60% plan set grading plans

Based on 22 ft high embankment with 125 pcf fill

Coordinates

X [ft]	Y [ft]
54.527	1326.61
154.527	1326.61
154.527	1363.61
54.527	1363.61

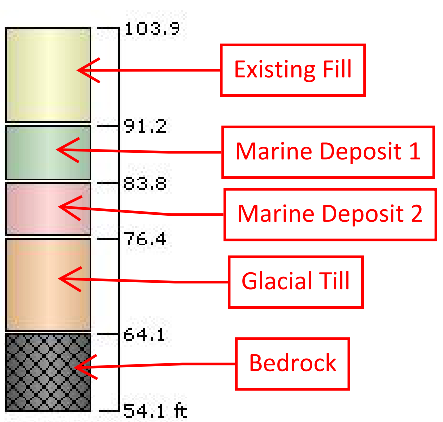
Profile based on average top of strata elevations from nearby test borings BB-BWS-104(OW) and BB-BWS-105)

Ignore anything referenced to depth (model is completely elevation-based)

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Existing Fill	12.7	-103.9	No
2	Marine Deposit 1	7.4	-91.2	No
3	Marine Deposit 2	7.4	-83.8	Yes
4	Glacial Till	12.3	-76.4	No
5	Bedrock	10	-64.1	No



Soil Properties

Property	Existing Fill	Marine Deposit 1	Glacial Till	Bedrock
Color				
Unit Weight [kips/ft^3]	0.12	0.11	0.115	0.15
Saturated Unit Weight [kips/ft^3]	0.12	0.11	0.115	0.15
K0	1	1	1	1
Immediate Settlement	Enabled	Disabled	Enabled	Enabled
Es [ksf]	300	-	3000	10000
Esur [ksf]	300	-	3000	10000
Primary Consolidation	Disabled	Enabled	Disabled	Disabled
Material Type		Non-Linear		
Cce	-	0.116	-	-
Cre	-	0.01	-	-
e0	-	1.1	-	-
OCR	-	10.2	-	-
Cv [ft^2/d]	-	0.2739	-	-
Cvr [ft^2/d]	-	0.2739	-	-
B-bar	-	1	-	-
Secondary Consolidation	Disabled	Standard	Disabled	Disabled
Cae	-	0.0003	-	-
Care	-	0.0003	-	-
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Property	Marine Deposit 2
Color	
Unit Weight [kips/ft^3]	0.11
Saturated Unit Weight [kips/ft^3]	0.11
K0	1
Primary Consolidation	Enabled
Material Type	Non-Linear
Cce	0.116
Cre	0.01
e0	1.1
OCR	2.3
Cv [ft^2/d]	0.2739
Cvr [ft^2/d]	0.2739
B-bar	1
Secondary Consolidation	Standard
Cae	0.0003
Care	0.0003
Undrained Su A [kips/ft2]	0
Undrained Su S	0.2
Undrained Su m	0.8
Piezo Line ID	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	-103.9 ft

based on latest reading from BB-BWS-104(OW)

HALEY ALDRICH	CALCULATIONS	File No.	132076-005
		Sheet	1 of 1
Client	Maine Department of Transportation	Date	9-Apr-20
Project	Wilson Street Bridge Replacement - WIN No. 018915.20	Computed by	NAS
Subject	Estimate of Overconsolidation Ratio for Marine Deposit	Checked by	WAC

PROBLEM STATEMENT & OBJECTIVE

Estimate the overconsolidation ratio (OCR) of the marine deposits, based on the undrained shear strength

EXECUTIVE SUMMARY

Estimate settlements based on two different OCRs (2.3 and 10.2) for the marine deposit in Settle3D

ASSUMPTIONS

1. Apply the SHANSEP system to estimate OCR based on undrained shear strength from vane shear test at test boring BB-BWS-105 (Chuck Ladd, Stability Evaluation During Staged Construction, Journal of Geotechnical Engineering, Vol. 117, No. 4, 1991)
2. Existing vertical effective stress based on subsurface conditions at test boring BB-BWS-105
3. Recompression index = 0.02 (per MaineDOT 1982 settlement calculations/consolidation test data at ex. structure)
4. Virgin compression index = 0.21 (per MaineDOT 1982 settlement calculations/consolidation test data at ex. structure)

AVAILABLE INFORMATION

1. Available vane shear data

CALCULATION

Undrained shear strength, S_u =	1,980	psf (vane shear test data)
S variable =	0.22	unitless (for silts and clays in Maine, based on past practice)
C_r, recompression index =	0.02	unitless (average value from MaineDOT 1982 consolidation test data)
C_c, virgin compression index =	0.21	unitless (average value from MaineDOT 1982 consolidation test data)
m variable =	0.80	unitless (per Ladd, p. 585, 1991)
Existing vertical effective stress, σ_{vo}' =	1,413	psf (based on depth of vane shear test performed at BB-BWS-105)
Estimated OCR (rearranged equation) =	**10.2**	unitless (per Ladd Eq. 13a, p. 584, 1991)

C. Ladd, 1991

sus log OCR plots; and the addition of CAUDSS tests under conditions described for level A programs. He also favors the Geonor apparatus since it provided the data plotted in Fig. 18 and limited comparisons suggest that other DSS devices may tend to give somewhat higher, more scattered strengths.

An alternative program for level B can use CK_0U triaxial compression and extension tests to estimate the average c_u/σ'_{vc} versus OCR relationship. If the results are first treated for strain compatibility, the average strength should be reasonable based on the data contained in Table 4 (except for varved clays due to their very low DSS strength). However, CK_0U triaxial testing, compared to DSS testing, requires more soil and effort and a higher level of experience (hence greater costs) to obtain reliable data, especially for shear in extension.

Level B should not rely on isotropically consolidated triaxial compression testing since *CIUC* strengths will greatly exceed the in situ average for most soils. Section 6.2 evaluates the use and interpretation of *CIUC* data when employed as part of the *QRS* methodology for stability analyses.

Selection of strength parameters at level C uses empirical correlations, rather than CU testing, in the form of

$$\frac{c_u}{\sigma'_{vc}} = S(\text{OCR})^m \quad \text{(13}a\text{)}$$

which, for $m = 1$, becomes

$$\frac{c_u}{\sigma'_p} = S_p \quad \text{(13}b\text{)}$$

Based on the data in Fig. 18 and the related text in Section 4.10, the writer has concluded that: CL and CH clays tend to have lower, less scattered undrained strength ratios than soils plotting below the *A*-line; and the τ_{ave}/σ'_{vc} correlation line for clays is probably more reliable than the c_u/σ'_p line obtained from the case histories for highly plastic clays. In any case, the results in Fig. 18 can be used by readers to select values for S or S_p. When applying Eq. 13*a*, one also has to estimate m, which, according to the "critical state" concepts used to formulate the Modified Cam-Clay model of soil behavior, should equal $1 - C_s/C_c$, where C_s and C_c represent the slopes of the swelling and virgin compression lines, respectively (Roscoe and Burland 1968).

The writer's interpretation of Fig. 18 and other experience lead to the following recommendations (SD = standard deviation).

- Sensitive marine clays ($I_p < 30\%$, $I_L > 1$):
  $S_p = 0.20$, with nominal SD = 0.015
- Homogeneous CL and CH sedimentary clays of low to moderate sensitivity (I_p = 20%–80%):
  $S = 0.20 + 0.05 I_p$, or simply $S = 0.22$.
  $m = 0.88(1 - C_s/C_c) \pm 0.06$ SD, or simply $m = 0.8$.
- Northeastern U.S. varved clays:
  $S = 0.16$ (assumes DSS failure mode predominates).
  $m = 0.75$.
- Sedimentary deposits of silts and organic soils (Atterberg limits plot below

the *A*-line, but
$S = 0.25$, w
$m = 0.88$ (1

The aforementioned *m*
CK_0UDSS data on 13
Levels A, B, and C
of the foundation soils
OCR for most soils (e
means that consolidati
experimental compone

5.4. Stability Analyse
This section deals w
librium stability analy
input of undrained str
use a computerized m
computing factors of s
as recommended in Se
automatically is obvio
circular-arc and Janbu
results should be chec
both force and momer
Spencer (1967).
First consider analy
sign or to represent t
strength anisotropy. F
step 1 in Table 5 can
continuous profiles de
can also be followed
step 3. But this proce
tations to obtain verti
and finally selection o
ment case histories. F
mally consolidated soi
$S\sigma'_{vc}$ within OCR = 1
effort then becomes s
replacing ϕ' and σ'_n wi
Inc. did this for the t
would still be needed
calculate c_u values for
Analyses employing
plex, especially since
inclination of the failu
sional slope stability p
restricted to circular a
ment-type loadings, w
two embankment case
tive and passive wed
strengths had to be

C. Ladd, 1991

; under conditions de-
nor apparatus since it
ıparisons suggest that
›re scattered strengths.
ıxial compression and
OCR relationship. If
verage strength should
‹cept for varved clays
triaxial testing, com-
ınd a higher level of
, especially for shear

l triaxial compression
situ average for most
of *CIUC* data when
/ analyses.
npirical correlations,

. . . . (13*a*)

. . . . (13*b*)

tion 4.10, the writer
ower, less scattered
A-line; and the $\tau_{ave}/$
than the c_u/σ'_p line
ɾs. In any case, the
:s for *S* or S_p. When
:cording to the "crit-
ı-Clay model of soil
›resent the slopes of
Roscoe and Burland

›erience lead to the

to moderate sensitiv-

0.8.

:s).

erg limits plot below

---

the *A*-line, but excluding peats) and clays with shells:

$S = 0.25$, with nominal SD $= 0.05$.

$m = 0.88\ (1 - C_s/C_c) \pm 0.06$ SD, or simply $m = 0.8$.

The aforementioned m versus $(1 - C_s/C_c)$ relation is based on analysis of CK_0UDSS data on 13 soils having maximum OCR's of 5–10.

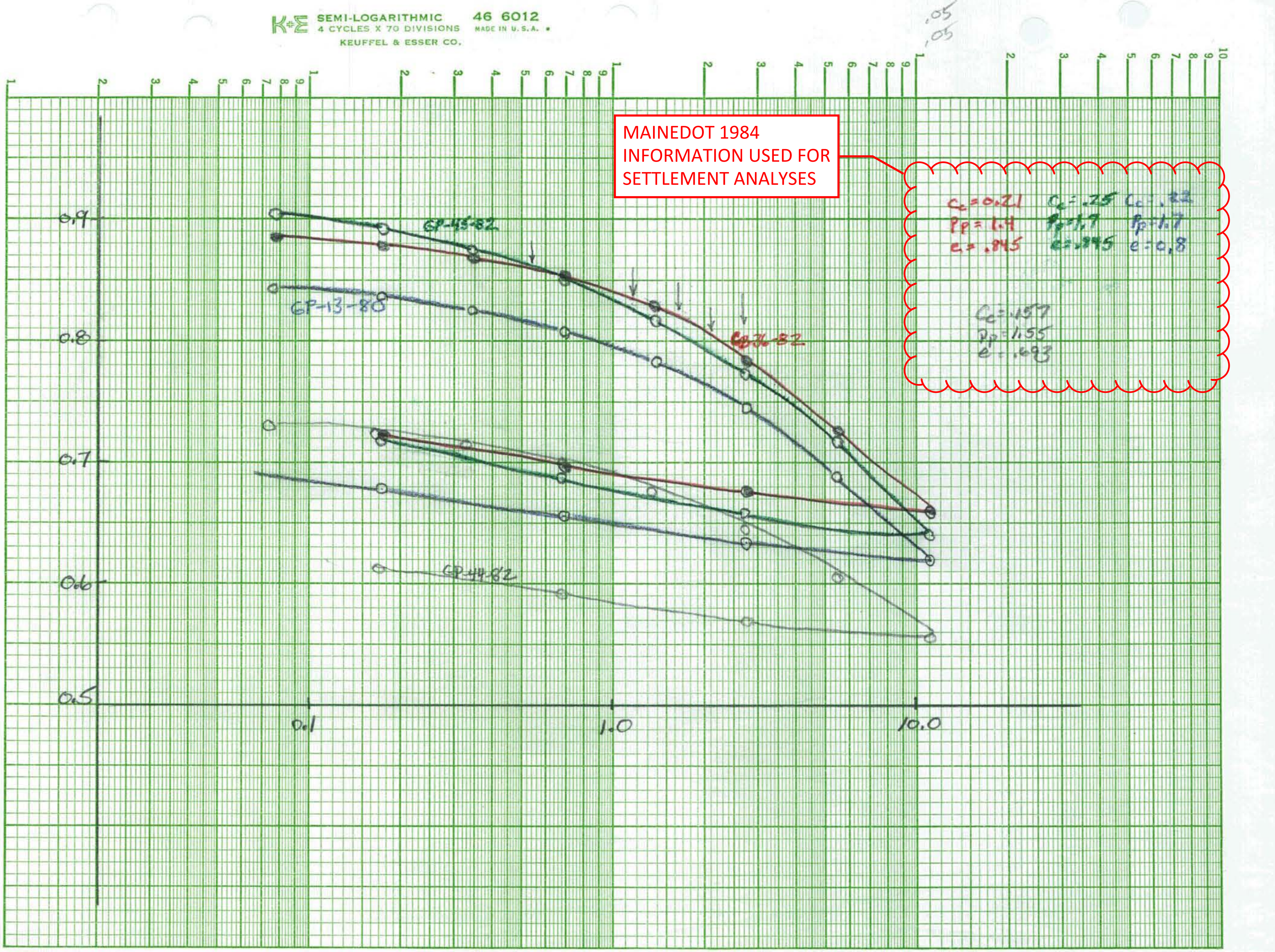
Levels A, B, and C all require a careful assessment of the stress history of the foundation soils. This fact, plus the observation that $c_u(\text{ave})/\sigma'_{vc}$ versus OCR for most soils (except varved clays) falls within a fairly narrow range, means that consolidation testing usually represents the single most important experimental component for the design of staged construction projects.

5.4. Stability Analyses

This section deals with some practical aspects of executing limiting equilibrium stability analyses (component T3 in Table 5), especially regarding input of undrained strengths. It is assumed that final design analyses will use a computerized method of slices that satisfies *moment* equilibrium in computing factors of safety for circular-arc or wedge-shaped failure surfaces as recommended in Section 2.2. The ability to search out the minimum FS automatically is obviously desirable, as commonly done for Bishop (1955) circular-arc and Janbu (1973) wedge-shaped surfaces. However, the latter results should be checked by a "generalized" method of slices that satisfies both force and moment equilibrium, e.g., Morgenstern and Price (1965) or Spencer (1967).

First consider analyses using *isotropic* strengths, either for preliminary design or to represent the average c_u for soils having low to moderate undrained strength anisotropy. For first stage construction, the c_u values obtained from step 1 in Table 5 can be used directly as input data, either as "zones" or continuous profiles depending upon the computer code. The same approach can also be followed for subsequent stages via the c_u values computed in step 3. But this process can get rather cumbersome, i.e., separate computations to obtain vertical consolidation stress (σ'_{vc}) profiles, then c_u values, and finally selection of input strengths. This was done for the two embankment case histories. For construction involving substantial portions of normally consolidated soil, a computer code that automatically calculates $c_u = S\sigma'_{vc}$ within OCR $= 1$ zones would greatly simplify the process. The required effort then becomes similar to a conventional effective stress analysis by replacing ϕ' and σ'_n with S and σ'_{vc} for applicable soils. Bromwell & Carrier, Inc. did this for the tailings dam case history. However, step 3 in Table 5 would still be needed for natural deposits to identify OCR $= 1$ soils and to calculate c_u values for σ'_{vc} less than σ'_p.

Analyses employing *anisotropic* strengths become significantly more complex, especially since most computer codes cannot automatically vary c_u with inclination of the failure surface. One notable exception is the three-dimensional slope stability program developed by Azzouz et al. (1981), although restricted to circular arcs. The writer's experience mainly involves embankment-type loadings, where wedge-shaped surfaces need evaluation. For the two embankment case histories (Figs. 6 and 9), with inclinations of the active and passive wedges being specified at $45 \pm \phi'/2$ degrees, the input strengths had to be changed for each change in the horizontal location of

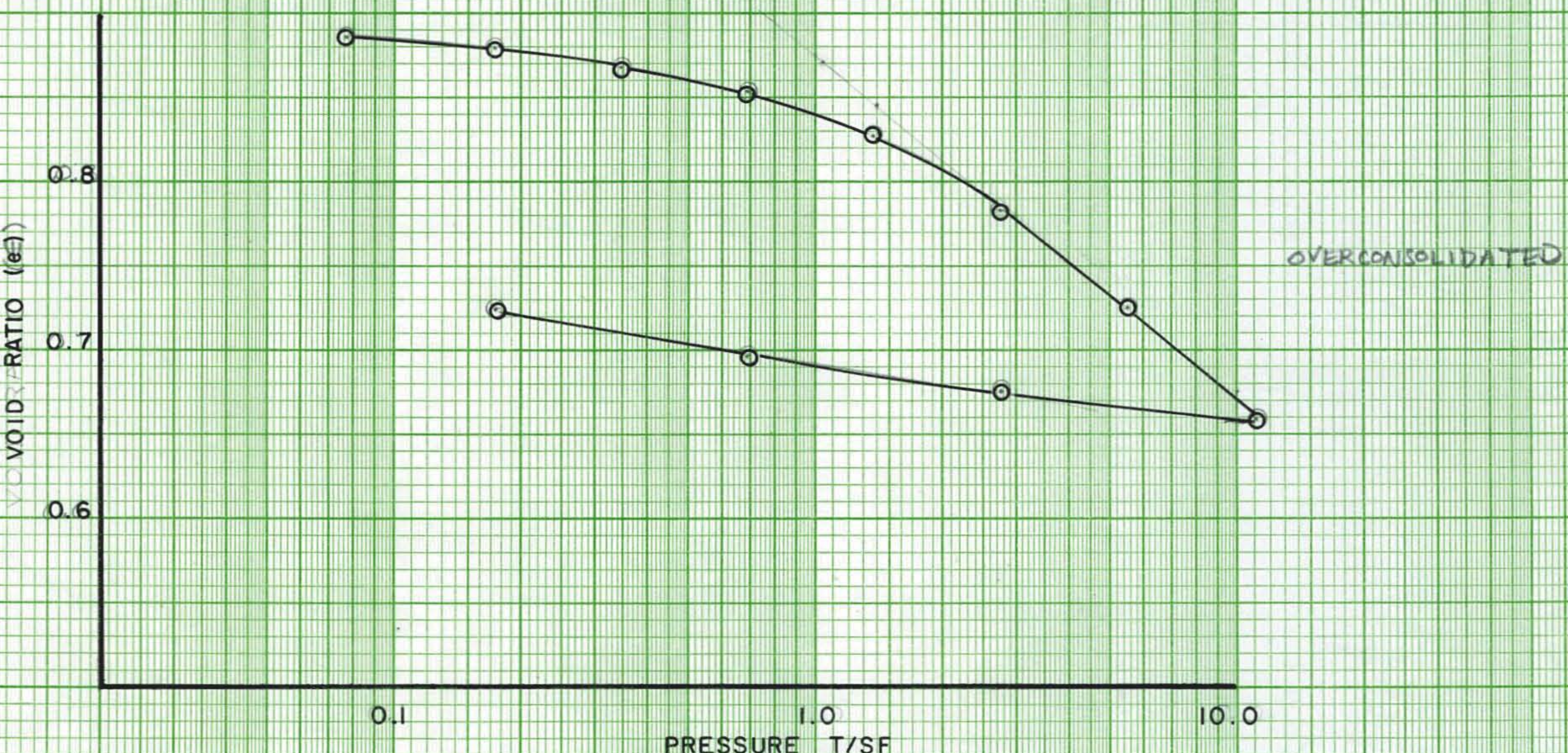


K+E SEMI-LOGARITHMIC 46 6012
4 CYCLES X 70 DIVISIONS MADE IN U.S.A.
KEUFFEL & ESSER CO.
.05
.05
MAINEDOT 1984 INFORMATION USED FOR SETTLEMENT ANALYSES
Cc = 0.21
Pp = 1.4
e = .845
Cc = .25
Pp = 1.7
e = .895
Cc = .22
Pp = 1.7
e = 0.8
Cc = .157
Pp = 1.55
e = .693
GP-45-82
GP-13-80
GP-36-82
GP-44-82
0.9
0.8
0.7
0.6
0.5
0.1
1.0
10.0

stiff gray-brown mottled silty clay w/ silt lenses

VANE	.352	Pmin	1.05	Cv 20#	-301
WC's	30-28	Pmax	2.5±	40	-216
G	2.76	Pp	1.4	80	-215
hs	.4659	e	.845		
		Cc	.21		

LC 19-82



PRESSURE - VOID RATIO DIAGRAM

BREWER

395-8 (79)

BORING CB 36-82 SAMPLE 1 U

JULY, 1982

83D

63+00 120' RT

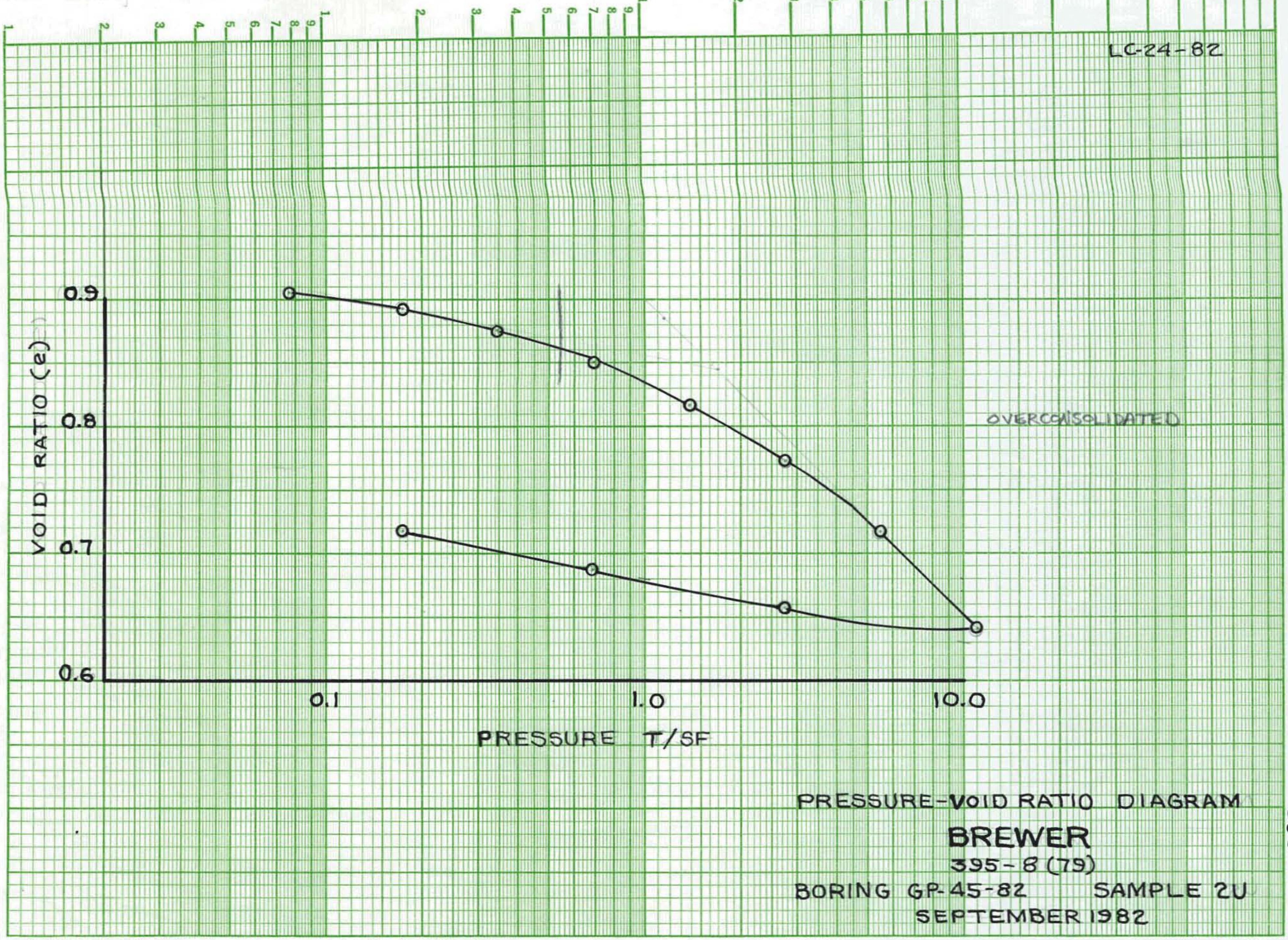
5'-7'

K+E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS 46 6012 MADE IN U.S.A. KEUFFEL & ESSER CO.

VANE .528
WC'S 33-27
G 2.78
h_S .4537

P_{MIN} 1.15
P_{MAX} 45
P_P 1.7
e .845
C_C .25

CV 10# 166
20 - 111
40 - 169
80 - 85



88C

66+00, 79' RT.

8'-10'

K+E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS 46 6012 MADE IN U.S.A.
KEUFFEL & ESSER CO.

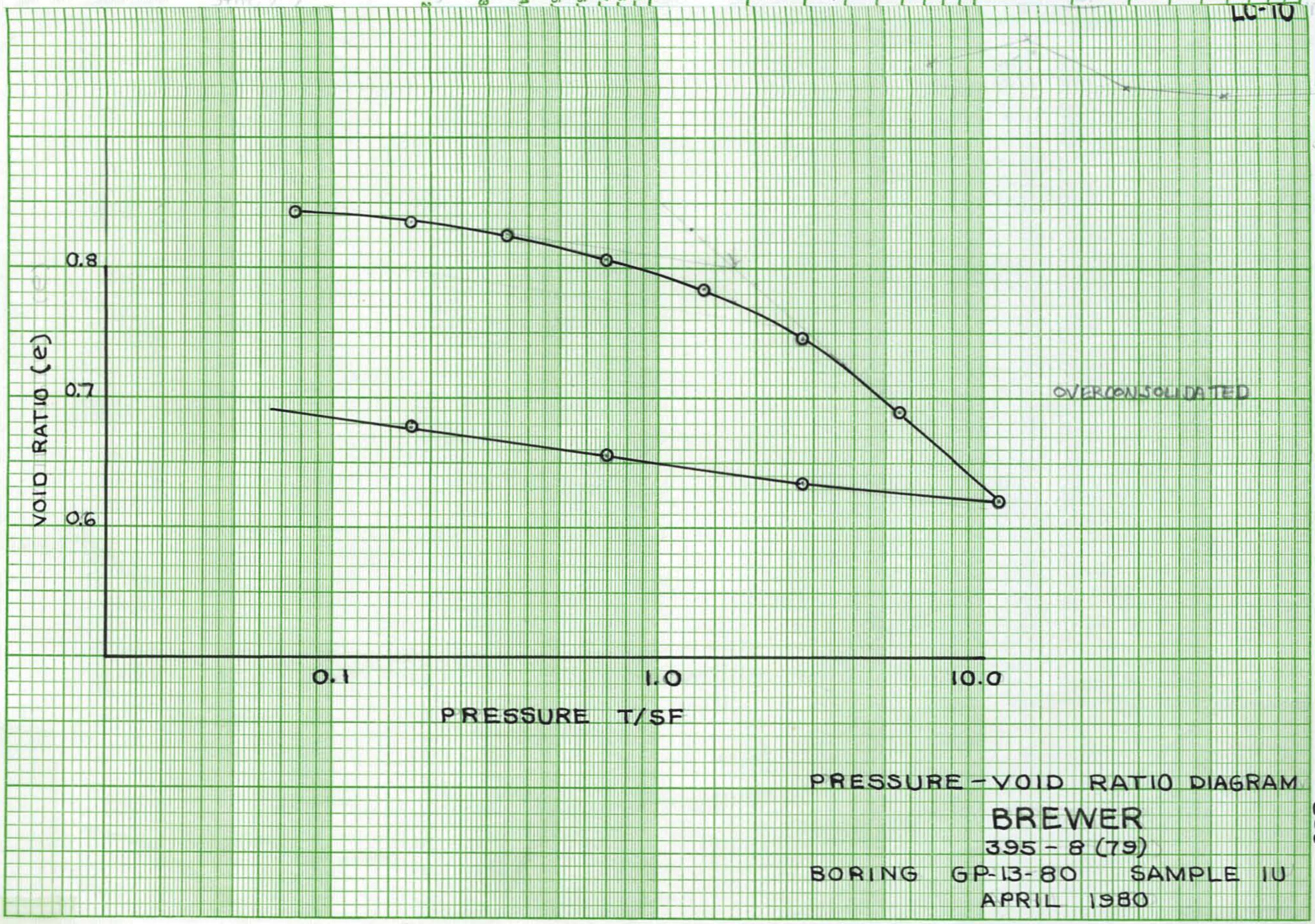
Vane .576
Wc's 32-26
G-2.74
h_s = .474

P_{min} 1.25
P_{max} 3.7
Pp 1.7
e .8
C_c .22

CV 5#-109
10 - 128
20 - 91
40 - 83
80 - 87

LC-10

Stiff gray weathered silty clay

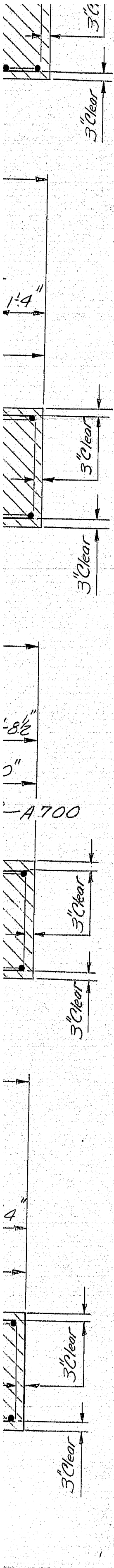


86C

51+50, 5 RT Felts Brook (335+71, 83R)

10'-12'
el. 75.08

MAINEDOT 1984 INFORMATION USED TO HELP CHECK ESTIMATED TOTAL SETTLEMENT



3___Place 4" diameter drains in breastwall and wings at 20 feet maximum spacing. Exact location to be determined by the Engineer in the field.

4___Maximum calculated footing pressures are:

Abutments 5.0 Tons/square foot

Wings 4.0 Tons/square foot.

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILSON STREET
OVER
I-395
BREWER

ABUTMENT FOOTINGS

182-62

SHEET 29 OF 121 AUGUSTA, MAINE

June 12, 1984

Larry Roberts — Design

Pete Coughlan — M & R Soils

Brewer - Wilson Street Over I-395

Several weeks ago it came to our attention that Abutment #2 of this bridge was undergoing some settlement. From observation and survey elevations of the bearing pads, the effects of this settlement can be seen most notably on the easterly half of the abutment. After reviewing the subsurface soil conditions in this area, we decided to do two additional borings adjacent to the footing and attempt to locate any areas of compressible material. The locations of the borings are shown on the accompanying plan and these borings encountered similar conditions to the 1982 washborings. That is, no distinct zone of compressible gray silty clay could be identified. The only potentially compressible material is the typical stiff gray and brown sandy clay-silt material that is located below the old fill embankment. This appears to range between 4 feet and 10 feet in thickness under the entire abutment length.

Using the maximum abutment deadloads and the embankment loads, the theoretical stress increases were calculated and applied to several corresponding pressure-void ratio curves from tests on stiff clay samples. In the first case, calculations were made for the conditions which existed over this past winter - namely, the nearly complete abutment sitting as a deadload at Elevation 113.7. The maximum toe pressure of 1.4 TSF produces a stress increase of 0.5± TSF within the stiff clay layer. Theoretically, this creates a consolidation of the clay layer of less than 1 inch. This corresponds roughly with the measured change in elevation of the easterly three bearing pads.

The second case includes the backfill material constructed to finished grade and this yields a stress increase of 0.75± TSF within the clay layer. This produces a consolidation of the clay layer of approximately 1 inch.

And the third case represents the completed structure and approach fills. The maximum toe load combined with the embankment load produces a total stress increase of 2.5± TSF in the stiff clay layer. This theoretically yields a total consolidation amount of 3± inches.

Thus, it is fair to say that a total settlement of approximately 3 inches can be anticipated for this abutment. As of May 22, 1984, it appeared that only the east wing and breastwall section were settling. Survey elevations of pads #1 through #6 were relatively unchanged between last November and May 22nd and pads #7, #8 and #9 showed amounts of 1/2 inch, 3/4 inch, and 1 1/8 inches, respectively. In the following two weeks after May 22nd, more settlement was recorded including some movement on the westerly end. As of June 5, 1984 settlement of 3/4 inch was measured on the west end (near pad #1) and the east end had increased to 1 7/8 inches (near pad #9). Thus, it can be seen that the east end has experienced a greater total settlement.

Calculations were also made to determine the average rate of consolidation of the clay layer. Because the clay thickness is relatively small and the coefficient of consolidation (Cv) is approximately 100 ft^2/year, then an average of 95 percent consolidation is expected within 4 months of loading. This estimate

Axial Compressive Resistance of Steel H-Piles

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	25-Mar-2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	NAS
Subject:	Geotechnical and Structural Resistance of H-Piles	Checked by:	EAF

GEOTECHNICAL RESISTANCE ASSUMPTIONS

1. Axial compressive geotechnical resistance for piles end-bearing on bedrock as determined by the IRM was proposed by Thomas Sandford of the University of Maine (MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on the Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, 2006.
2. Laboratory test data (unconfined compressive strength testing) and common published values used for design.
3. Equations used herein are from AASHTO LRFD 2017.

CALCULATIONS

1. Summarize unconfined bedrock compressive strength data for samples located near proposed substrctures.

Proposed Substructure	Test Boring No.	Approximate Sample Elevation (ft, NAVD 88)	Unconfined Compressive Strength (ksi)
Abutment No. 2	BB-BWS-104	59.6	2.5
Abutment No. 1	BB-BWS-202	64.1	2.1
Pier	BB-BWS-203	65.9	4.4
Abutment No. 2	BB-BWS-206	54.8	14.7

Common unconfined compressive strength of schist from literature:

R. Goodman (1980), Table 3.1 =	8.0	ksi
Average value LRFD Table 4.4.8.1.2B =	11.2	ksi

2. Select a design unconfined compressive strength of the bedrock for all foundation piles.

Neglect the unconfined compressive strengths of 2.1-4.4 ksi from lab testing from above, since these samples experienced discontinuity failures, whereas the unconfined compressive strength of 14.7 ksi experienced an intact material failure. We believe that intact material failures better represent the strength of the rock mass. In our experience, samples that experience discontinuity failures, result in unrealistically low compressive strength values.

Assume the design unconfined compressive strength of the bedrock is the average value of the green highlighted values from above.

Design unconfined compressive strength of bedrock, $q_{u,design}$ = 11.3 ksi (based on green values)

3. Determine the end-bearing capacity of the bedrock per Sandford (2014), Eq. 3.15:

End-bearing capacity of the bedrock, q_p = 28.3 ksi

4. Determine the factored axial geotechnical resistance of a steel HP14x117 pile at the Service, Strength, and Extreme Limit States

Service and Extreme Limit States Resistance Factor, ϕ =	1	(per LRFD 10.5.5.1 and 10.5.5.3)
Strength Limit State Resistance Factor, $\phi_{strength}$ =	0.45	(per Canadian Geotechnical Society)
Area of steel H-pile, A_s =	34.4	in^2 (HP14x117)

Factored axial geotechnical resistance at Service and Extreme Limit States, $P_{r,service/extreme}$ = 973 kips
Factored axial geotechnical resistance at Strength Limit State, $P_{r,strength}$ = 438 kips

STRUCTURAL RESISTANCE ASSUMPTIONS

1. H-Piles are driven to hard rock and the structural axial compression resistance of the pile is limited by the geotechnical axial resistance of the pile section (see Sheet 1).
2. H-Piles are subject to damage due to severe driving conditions where use of a pile tip is necessary. Resistance factor for axial resistance of piles in compression is 0.5 (Section 6.5.4.2 AASHTO LRFD 2014).
3. No corrosion loss is assumed.
4. H-Piles are completely embedded in soil and there is no reduction in axial resistance due to buckling.
5. This calculation only addresses axial resistance, if the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated.
6. The pile section has fy = 50 ksi.
7. Equations used herein are from AASHTO LRFD 2017.

CALCULATIONS

P_n shall be determined as follows:

- If $\frac{P_e}{P_o} \geq 0.44$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad (6.9.4.1.1\text{-}1)$$

- If $\frac{P_e}{P_o} < 0.44$, then:

$$P_n = 0.877 P_e \qquad (6.9.4.1.1\text{-}2)$$

where:

A_g = gross cross-sectional area of the member (in.^{2})

F_y = specified minimum yield strength (ksi)

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kips)

P_o = equivalent nominal yield resistance = QF_yA_g (kips)

Q = slender element reduction factor determined as specified in Article 6.9.4.2. Q shall be taken equal to 1.0 for bearing stiffeners.

6.9.4.1.2—Elastic Flexural Buckling Resistance

The elastic critical buckling resistance, P_e, based on flexural buckling shall be taken as:

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad (6.9.4.1.2\text{-}1)$$

where:

A_g = gross cross-sectional area of the member (in.^{2})
K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5
ℓ = unbraced length in the plane of buckling (in.)
r_s = radius of gyration about the axis normal to the plane of buckling (in.)

Section	Corrosion (in)	Effective A_s (in)	f_y (ksi)	Q	$P_0=Q\cdot f_y\cdot A_s$ (kip)	P_e (kip)	P_0/P_e	**P_n (kip)**	**ϕP_n (kip)**
HP14x117	0.000	34.4	50	1.0	1722	∞	0	**1722**	**861**

Notes:

1. Q·fy·As from Section 6.9.4.1.1 of AASHTO LRFD, Q=1 for nonslender elements. Pile is considered nonslender because the unbraced length is zero (i.e., completely embedded in soil).
2. P_e from Eq. 6.9.4.1.2-1 (Elastic Flexural Buckling Resistance). With the unbraced length=0, $P_e=\infty$.
3. P_n from Eq. 6.9.4.1.1-1.
4. Resistance factor for axial resistance of piles in compression is 0.5 (Section 6.5.4.2 AASHTO LRFD 2014).

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations.
[2]Not including oil shale.

IGNORE THIS MARKUP

USED AVERAGE VALUE (11.2 KSI) FOR DESIGN CONSIDERATION

$$\rho = q_o\,(1 - \nu^2)BI_p/E_m, \text{ with } I_p = (L/B)^{1/2}/\beta_z \qquad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

Goodman, 1980

Unconfined compression (Figure 3.3*a*) is the most frequently used strength test for rocks, yet it is not simple to perform properly and results can vary by a factor of more than two as procedures are varied. The test specimen should be a rock cylinder of length-to-width ratio in the range 2 to 2.5 with flat, smooth, and parallel ends cut perpendicularly to the cylinder axis. Procedures are recommended in ASTM designation D2938-71a and by Bieniawski and Bernede (1979). Capping of the ends with sulfur or plaster to specified smoothness is thought to introduce artificial end restraints that overly strengthen the rock. However, introduction of Teflon pads to reduce friction between the ends and the loading surfaces can cause outward extrusion forces producing a premature splitting failure, especially in the harder rocks. When mine pillars are studied, it is sometimes preferable to machine the compression specimen from a large cylinder to achieve loading through rock of the upper and lower regions into the more slender central region. In the standard laboratory compression test, however, cores obtained during site exploration are usually trimmed and compressed between the crosshead and platen of a testing machine. The compressive strength q_u is expressed as the ratio of peak load P to initial cross-sectional area A:

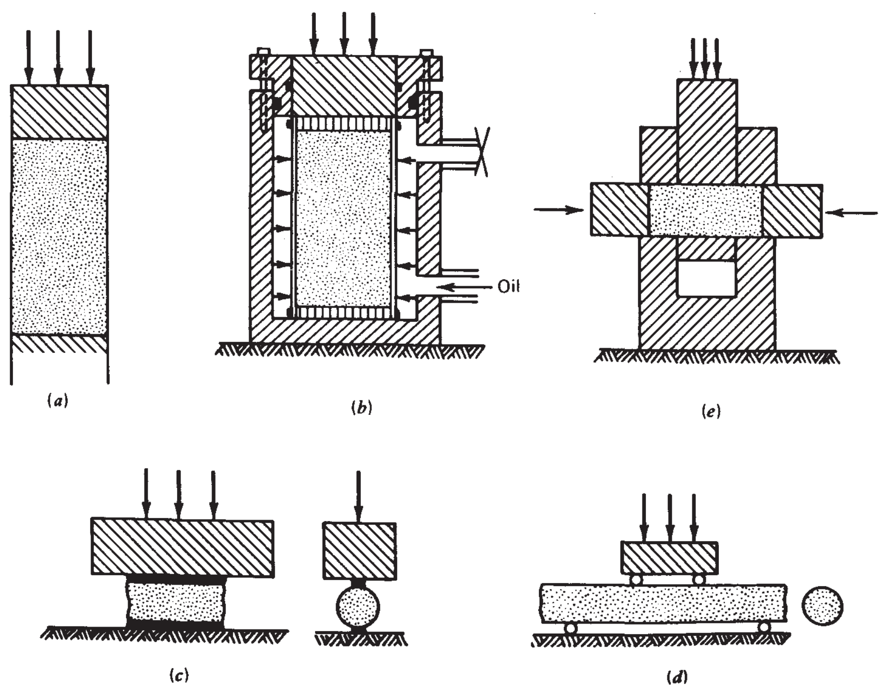


Figure 3.3 Common laboratory tests for characterizing rock strength criteria. (*a*) Unconfined compression. (*b*) Triaxial compression. (*c*) Splitting tension (Brazilian). (*d*) Four-point flexure. (*e*) Ring shear.

$$q_u = \frac{P}{A} \tag{3.1}$$

Representative values of q_u are listed in Table 3.1.

Triaxial compression (Figure 3.3*b*) refers to a test with simultaneous compression of a rock cylinder and application of axisymmetric confining pressure. Recommended procedures are described in ASTM designation D2664-67 (1974) and in an ISRM Committee report by Vogler and Kovari (1978).

***Table 3.1** Unconfined Compressive Strength (q_u) and Ratio of Compressive to Indirect Tensile Strength (q_u/T_0) for Specimens of Representative Rocks*

Description[a]	q_u MPa	q_u psi	q_u/T_0[b]	Reference[c]
Berea sandstone	73.8	10,700	63.0	5
Navajo sandstone	214.0	31,030	26.3	5
Tensleep sandstone	72.4	10,500		1
Hackensack siltstone	122.7	17,800	41.5	5
Monticello Dam s.s. (greywacke)	79.3	11,500		4
Solenhofen limestone	245.0	35,500	61.3	5
Bedford limestone	51.0	7,400	32.3	5
Tavernalle limestone	97.9	14,200	25.0	5
Oneota dolomite	86.9	12,600	19.7	5
Lockport dolomite	90.3	13,100	29.8	5
Flaming Gorge shale	35.2	5,100	167.6	3
Micaceous shale	75.2	10,900	36.3	2
Dworshak Dam gneiss 45° to foliation	162.0	23,500	23.5	5
Quartz mica schist ⊥ schistocity	55.2	8,000	100.4	5
Baraboo quartzite	320.0	46,400	29.1	5
Taconic marble	62.0	8,990	53.0	5
Cherokee marble	66.9	9,700	37.4	5
Nevada Test Site granite	141.1	20,500	12.1	7
Pikes Peak granite	226.0	32,800	19.0	5
Cedar City tonalite	101.5	14,700	15.9	6
Palisades diabase	241.0	34,950	21.1	5
Nevada Test Site basalt	148.0	21,500	11.3	7
John Day basalt	355.0	51,500	24.5	5
Nevada Test Site tuff	11.3	1,650	10.0	7

[a] Description of rocks listed in Table 3.1:

Berea sandstone, from Amherst, Ohio; fine grained, slightly porous; cemented. *Navajo sandstone*, from Glen Canyon Dam site, Arizona; friable, fine to medium grained. (Both sandstones are

Table Footnote (*continued*)

on bridge projects and assume q_u from correlations provided in AASHTO *Standard Specifications for Highway Bridges* 17th ed. (2002). In the cases where q_u was unavailable from the reports a value was assumed based on the rock type and q_u for those rock types on similar projects. When the calculations for the CGS method were provided in the geotechnical design reports, they were used in this study. However, for projects that did not include the calculation a procedure was needed to evaluate the input parameters without the bedrock samples.

The piles installed on the bridge projects included in this study were rarely socketed into bedrock, so the depth of penetration into bedrock (L_s) was set equal to zero. Although there was no specified driving of the pile into bedrock, the equation would not function properly if B was set equal to zero. Therefore B was set to 1 foot (approximate width of the piles) for calculations. The thickness of fractures (t_d) in the bedrock and the vertical spacing between fractures (s_v) were interpreted from the bedrock descriptions in the boring logs. The boring logs provide bedrock core descriptions including rock type, dip, spacing, tightness and infilling of the discontinuities. MaineDOT (Krusinski 2012) indicated that from the rock samples examined, the s_v can range from inches to feet and the fractures range from 1/64-inch for tight joints/bedding to <1/4 for open/healed joints. To determine the total capacity, the value calculated in the CGS method must be multiplied by the cross sectional area of the pile on bedrock.

3.4.2. Proposed Intact Rock Method For End Bearing

The proposed Intact Rock Method (IRM) for end bearing is equivalent to the Rowe and Armitage (1987) equation (cited by Turner, 2006) that relates the ultimate bearing capacity of intact rock to the compressive strength of the bedrock. The equation is presented below:

$$q_p = 2.5q_u \quad [3.15]$$

Where:

q_p = end bearing capacity of the bedrock

q_u = unconfined compressive strength of the bedrock

3.5. Tip Capacity for Piles Bearing in Till

There are a few piles included in the study that were designed to obtain support from soils without bearing on bedrock. There are also some piles that fetched up in the till or other overlying strata. There were not any piles that experienced end bearing in cohesive strata, so tip capacity in cohesive soils will not be considered in this report. The methods for determining end bearing on piles above bedrock are described in this section.

3.5.1. Nordlund Method

The Nordlund method (Nordlund 1963) comprised a bearing capacity relation from Berezantzes et al (1961) which did not have a limiting value. Since the Nordlund (1963) paper, the bearing capacity relation has been changed and a limiting value from Meyerhof (1976) has been added by Hannigan et al (2006a) based on Bowles (1977). Subsequent editions (Bowles 1982; Bowles 1988) do not use this method. The end bearing capacity of the soil now associated with the Nordlund method is detailed below (Hannigan et al, 2006a):

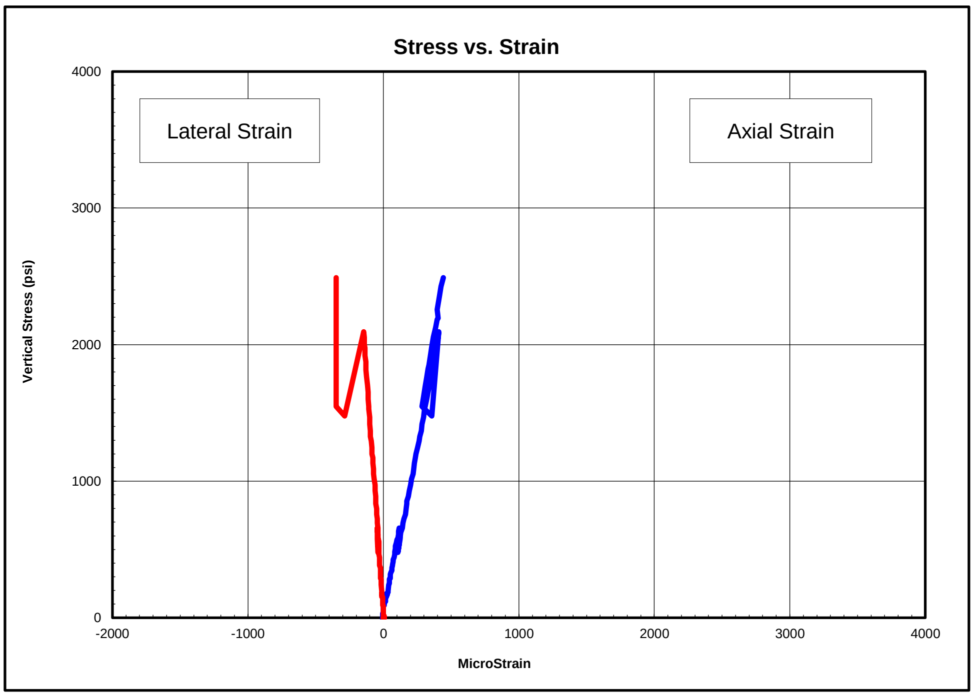
$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \quad [3.16]$$

Where:

α_t = coefficient determined from Figure 3-9.

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Wilson St Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308858
Test Date:	10/1/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BWS-104
Sample ID:	R1
Depth, ft:	41.3-41.9
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 2,490 psi

The strain gauges picked up an initial failure within the specimen and then continued reading until total failure occured.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
200-900	4,240,000	0.28
900-1600	5,230,000	0.41
1600-2000	5,490,000	0.38

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/I-395 Wilson St Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308858		
Boring ID:	BB-BWS-104		
Sample ID:	R1		
Depth:	41.3-41.9 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.38	4.38	4.38	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	594.33			
Bulk Density, lb/ft^3	168	**Minimum Diameter Tolerence Met?**		**YES**
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**		**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040

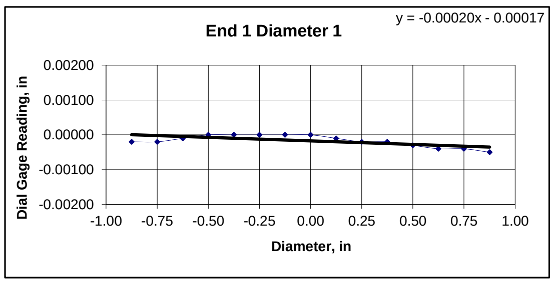
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00050

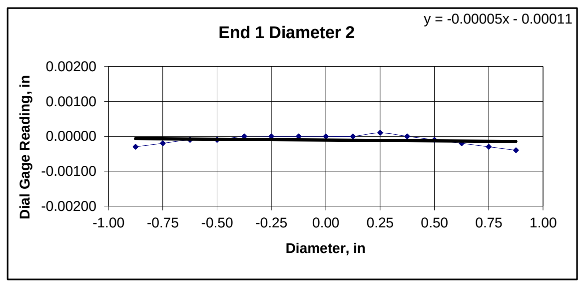
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040

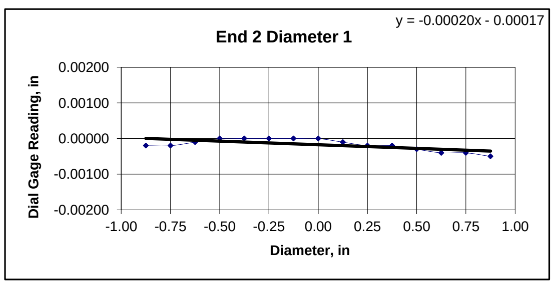
Difference between max and min readings, in:
0° = 0.0005 90° = 0.0005

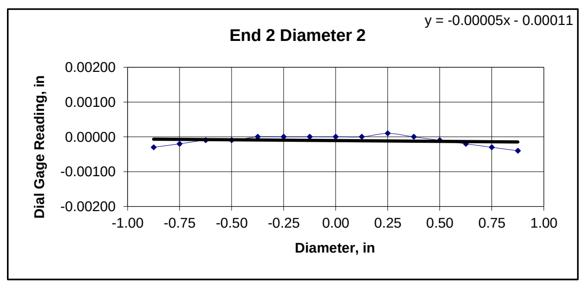
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01162

End 2:

Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01162

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00262

End 2:

Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00262

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	1.980	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	1.980	0.00025	0.014	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

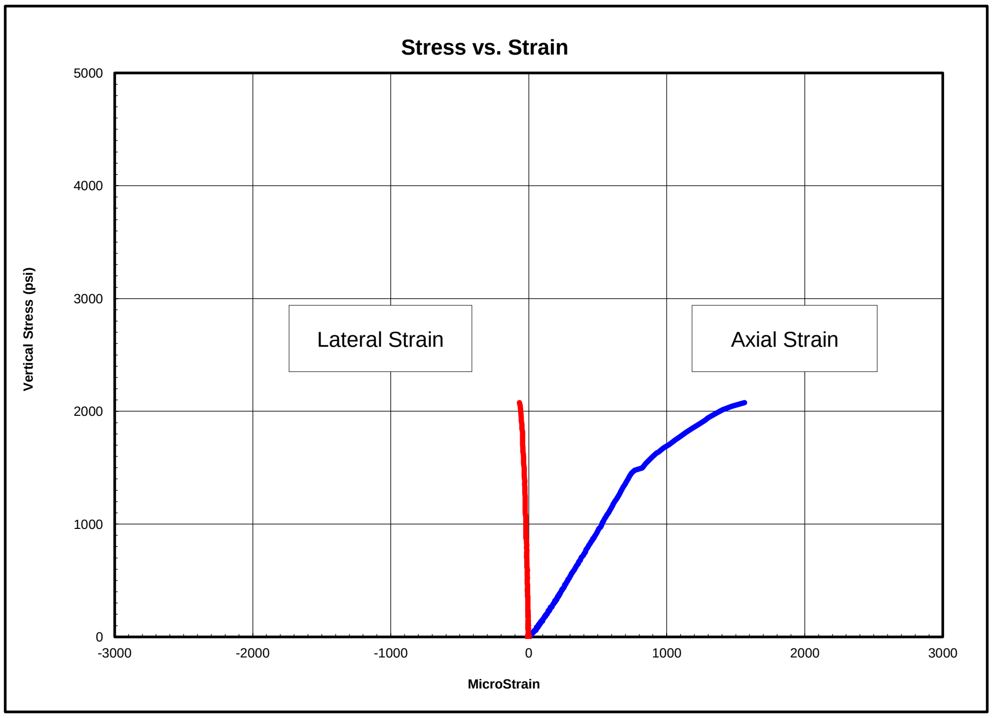
Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Wilson St Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308858
Test Date:	10/1/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BWS-104
Sample ID:	R1
Depth, ft:	41.3-41.9

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-202
Sample ID:	R3
Depth, ft:	79.7-80.2
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 2,077 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
200-700	1,980,000	0.05
700-1300	2,070,000	0.05
1300-1900	986,000	0.04

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-202		
Sample ID:	R3		
Depth:	79.7-80.2 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.22	4.22	4.22		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	597.46				
Bulk Density, lb/ft^3	173			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030	0.00040

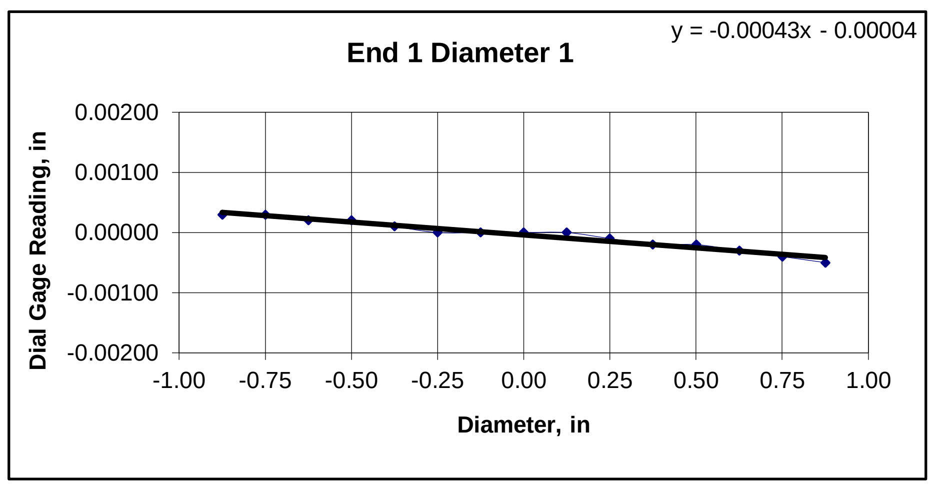
Difference between max and min readings, in:
0° = 0.00080 90° = 0.00080

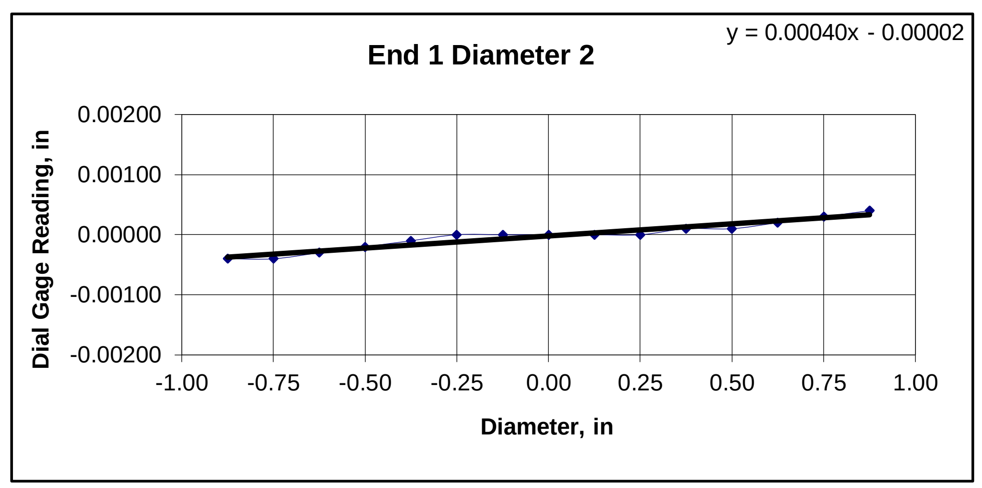
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00050	0.00040	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	0.00010	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040

Difference between max and min readings, in:
0° = 0.001 90° = 0.0008

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00050

Flatness Tolerance Met? **YES**

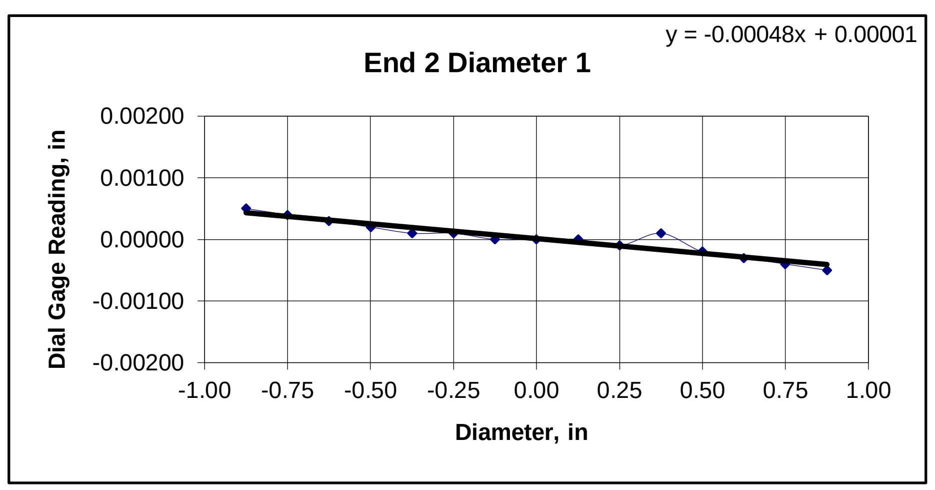


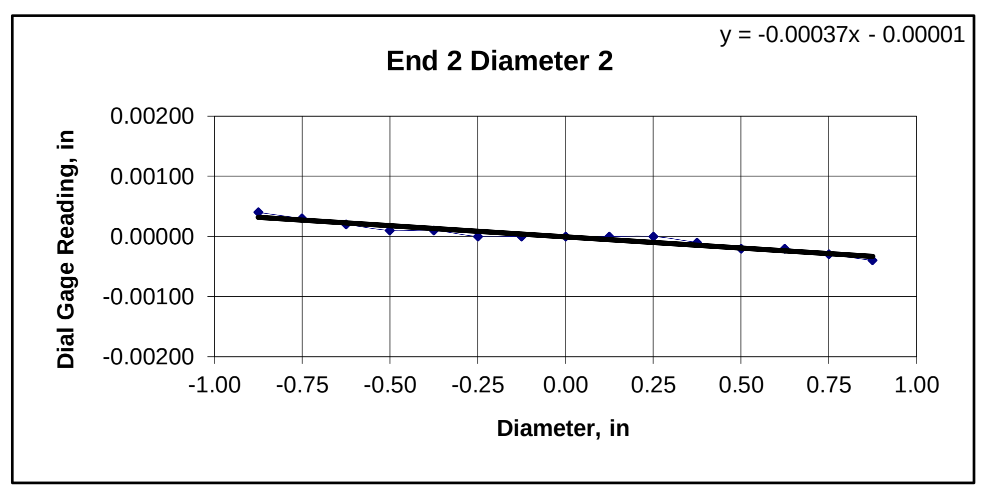


DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00043
	Angle of Best Fit Line:	0.02456
End 2:		
	Slope of Best Fit Line	0.00048
	Angle of Best Fit Line:	0.02750
Maximum Angular Difference:		0.00295

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00040
	Angle of Best Fit Line:	0.02308
End 2:		
	Slope of Best Fit Line	0.00037
	Angle of Best Fit Line:	0.02128
Maximum Angular Difference:		0.00180

Parallelism Tolerance Met? **YES**
Spherically Seated

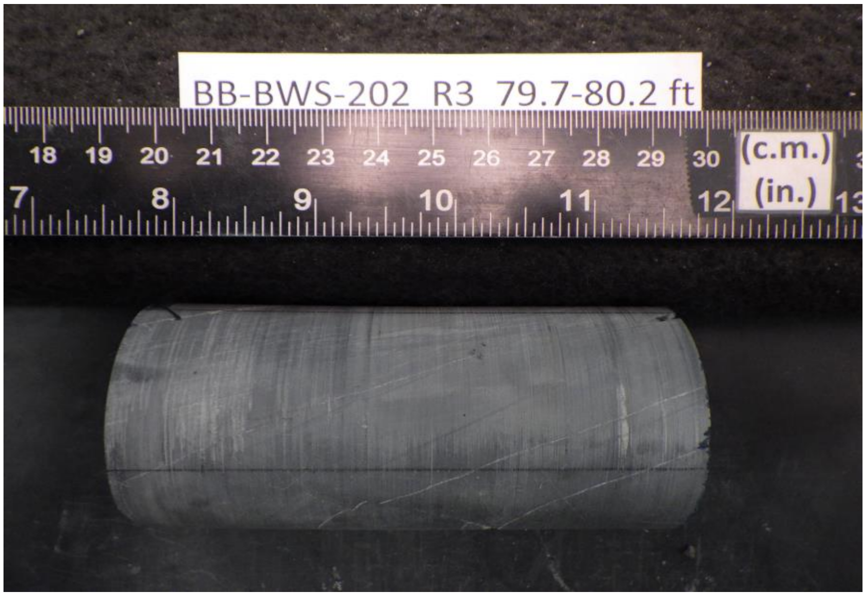
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00080	1.990	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES
END 2					
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES

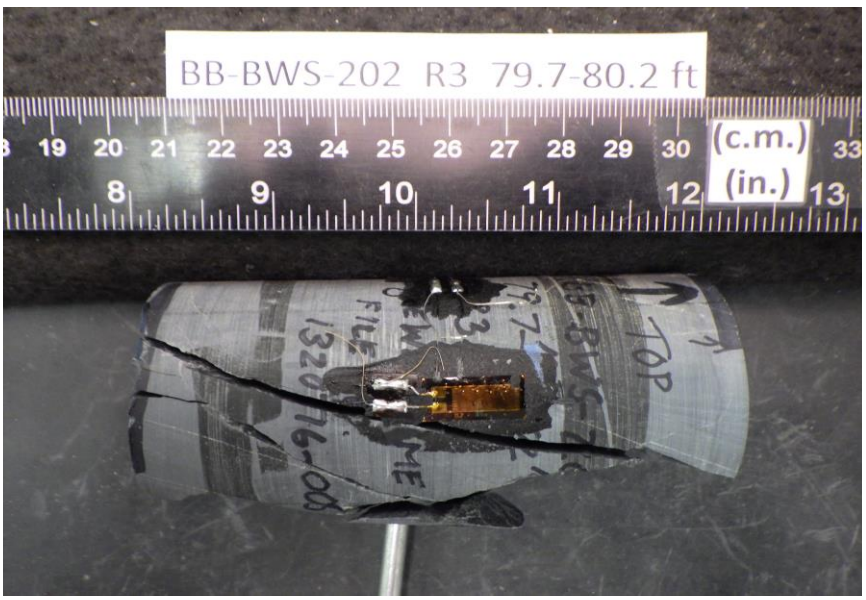
Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-202
Sample ID:	R3
Depth, ft:	79.7-80.2



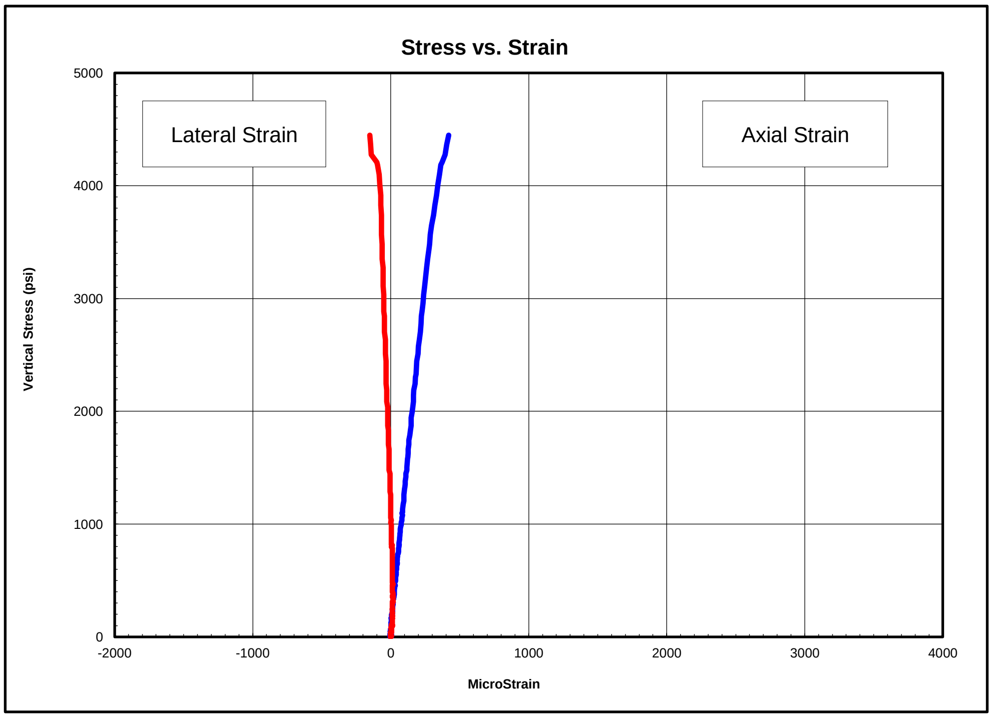
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-203
Sample ID:	R1
Depth, ft:	45.3-45.9
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 4,448 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
400-1600	12,000,000	0.27
1600-2800	12,300,000	0.36
2800-4000	10,100,000	0.26

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-203		
Sample ID:	R1		
Depth:	45.3-45.9 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.53	4.53	4.53
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	612.63		
Bulk Density, lb/ft^3	165	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010

Difference between max and min readings, in:

0° = 0.00070 90° = 0.00020

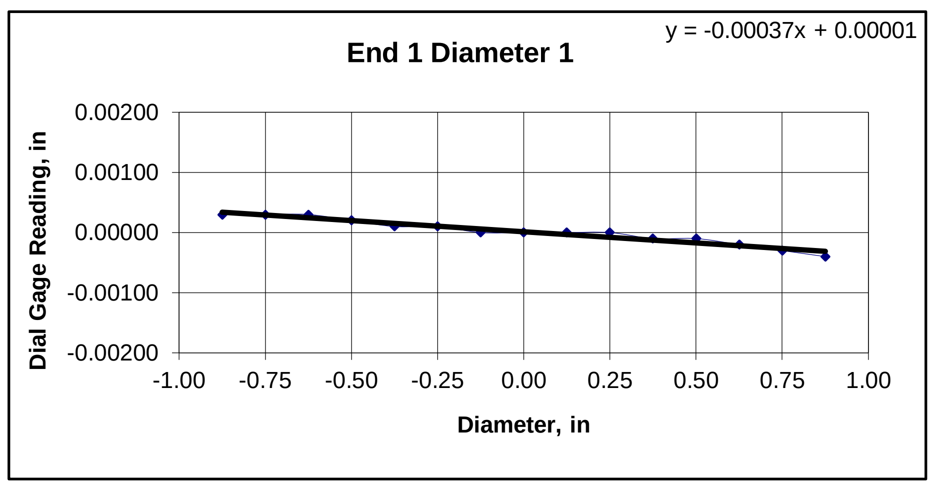
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

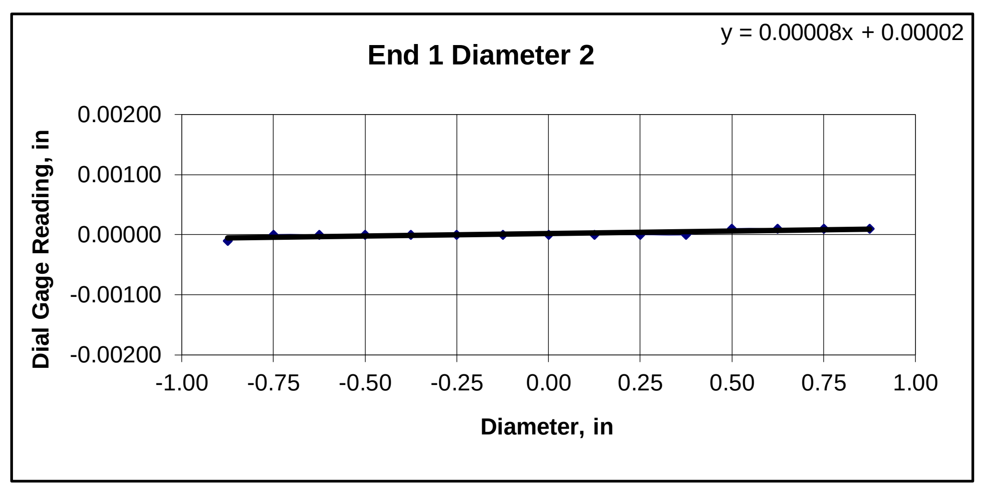
Difference between max and min readings, in:

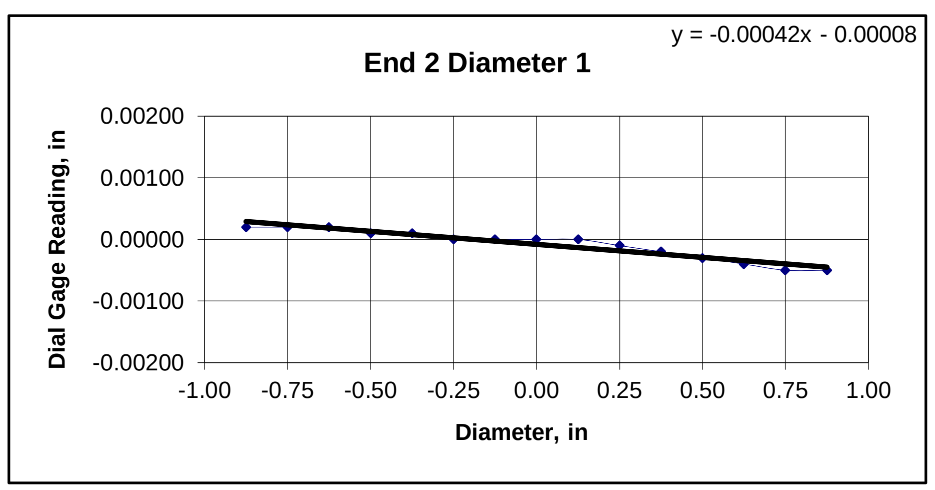
0° = 0.0007 90° = 0.0001

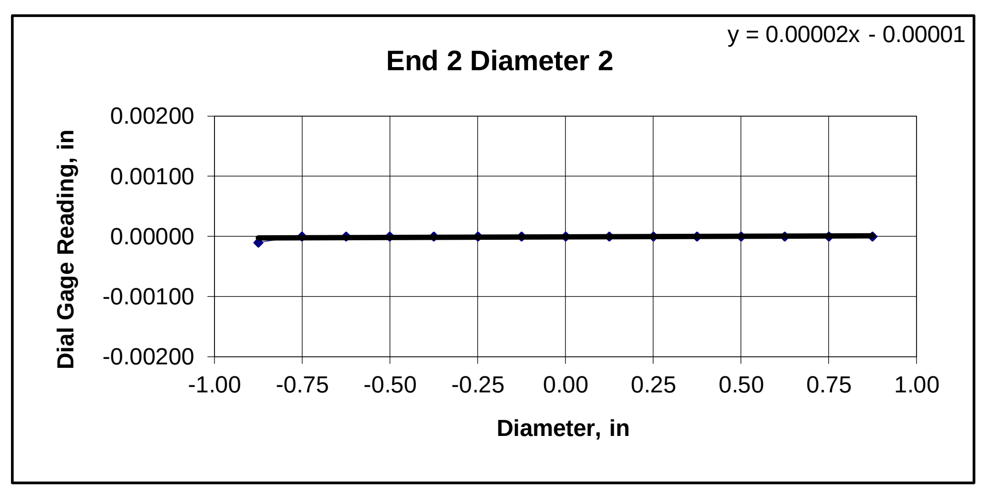
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00035

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00037
Angle of Best Fit Line:	0.02128

End 2:

Slope of Best Fit Line	0.00042
Angle of Best Fit Line:	0.02423

Maximum Angular Difference: 0.00295

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00475

End 2:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

Maximum Angular Difference: 0.00360

Parallelism Tolerance Met? YES

Spherically Seated

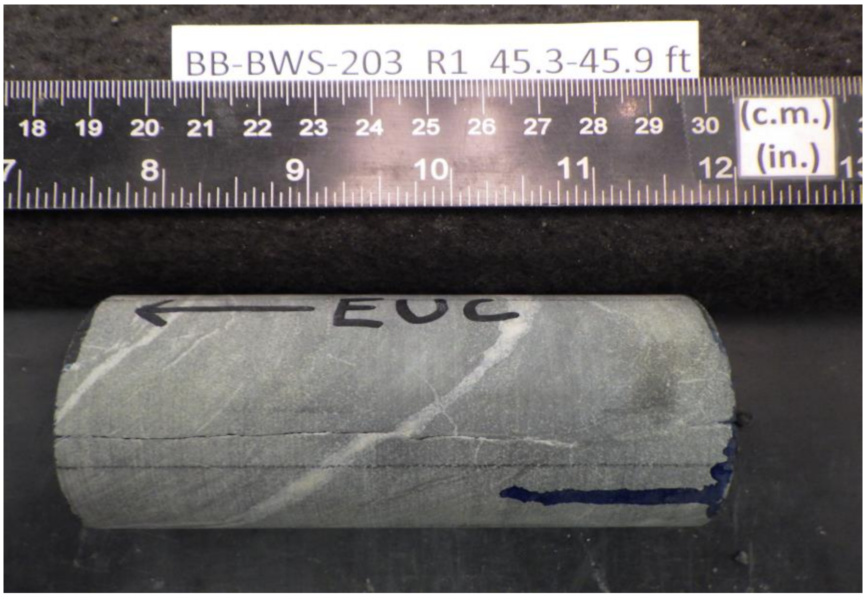
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES

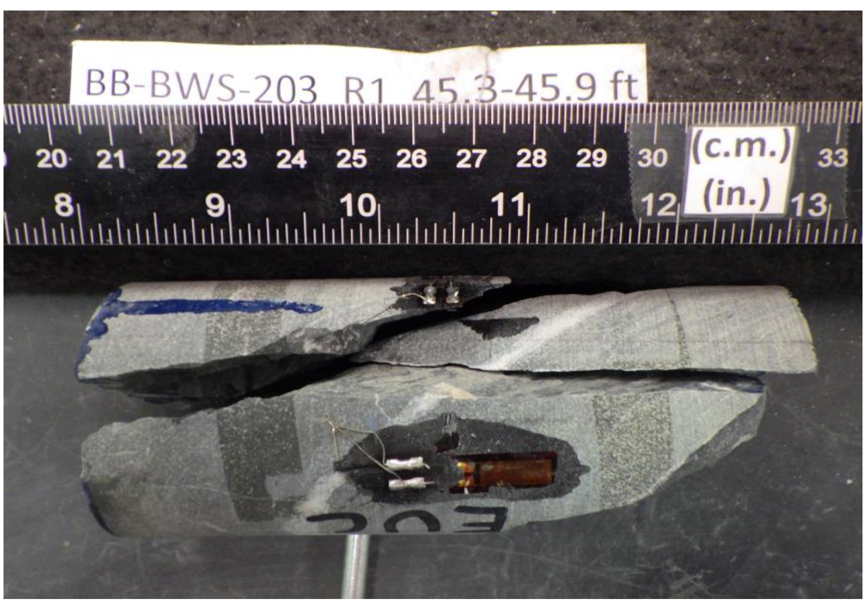
Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-203
Sample ID:	R1
Depth, ft:	45.3-45.9



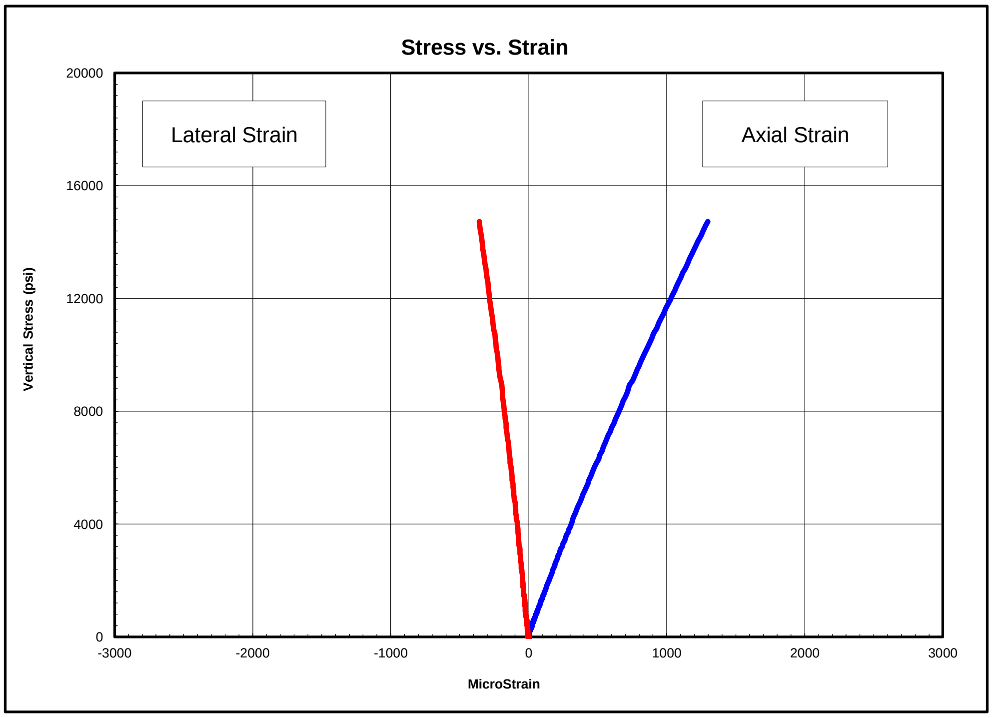
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-206
Sample ID:	R1
Depth, ft:	79.8-80.5
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 14,729 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1500-5400	12,400,000	0.26
5400-9300	11,300,000	0.27
9300-13300	10,300,000	0.28

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-396 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-206		
Sample ID:	R1		
Depth:	79.8-80.5 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.55	4.56	4.56		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	630.45				
Bulk Density, lb/ft^3	169			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00050	0.00040	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060

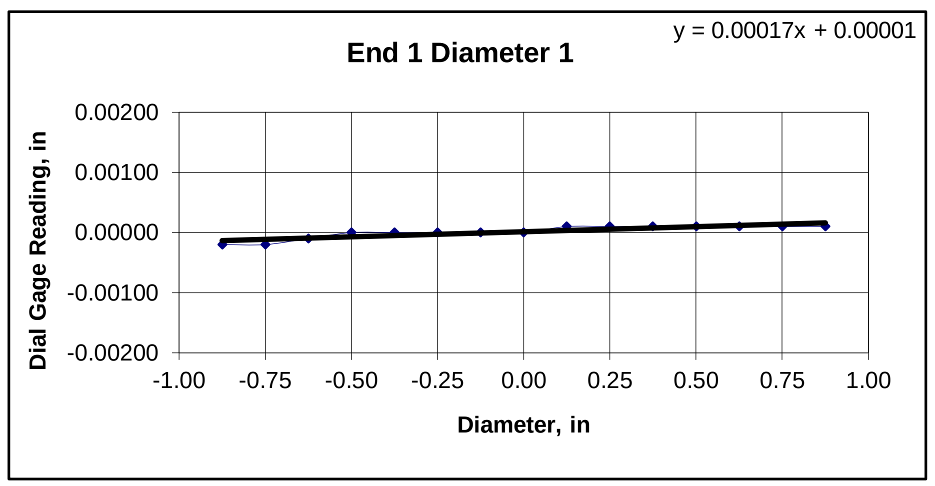
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00110

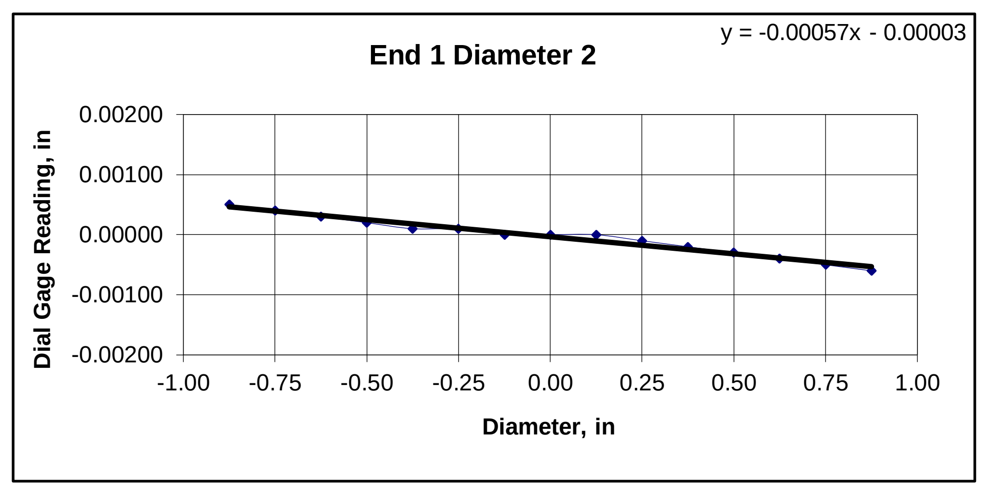
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00030	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00010	0.00030	0.00030	0.00030	0.00040	0.00050	0.00070

Difference between max and min readings, in:
0° = 0.0004 90° = 0.0012

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00060

Flatness Tolerance Met? YES





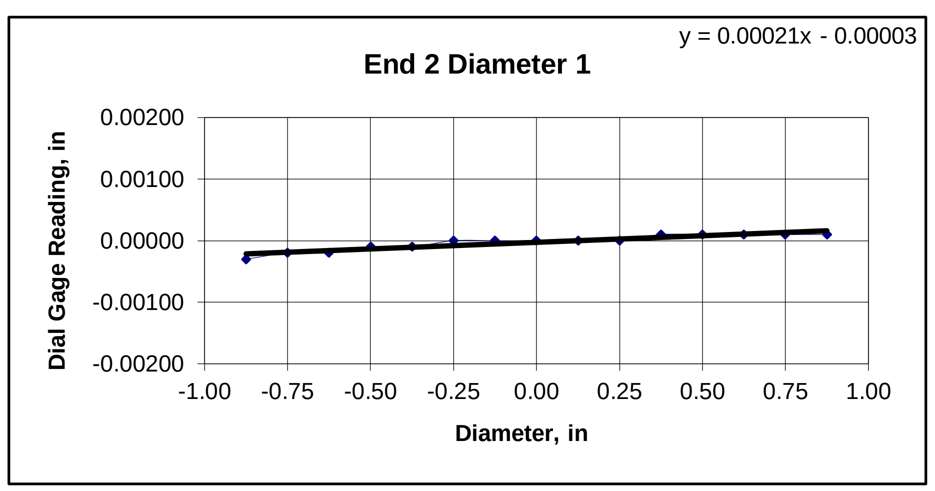
DIAMETER 1

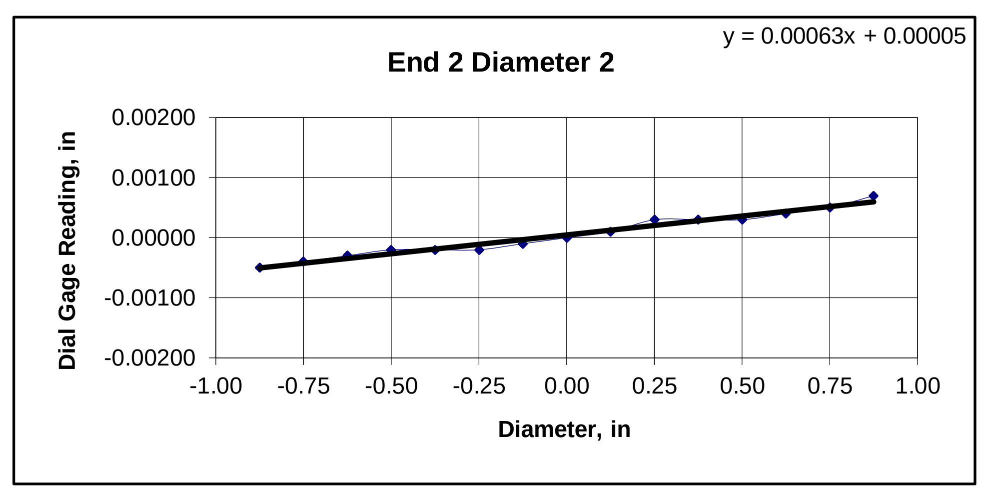
End 1:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00966

End 2:
Slope of Best Fit Line 0.00021
Angle of Best Fit Line: 0.01228

Maximum Angular Difference: 0.00262

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00057
Angle of Best Fit Line: 0.03258

End 2:
Slope of Best Fit Line 0.00063
Angle of Best Fit Line: 0.03601

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES
Spherically Seated

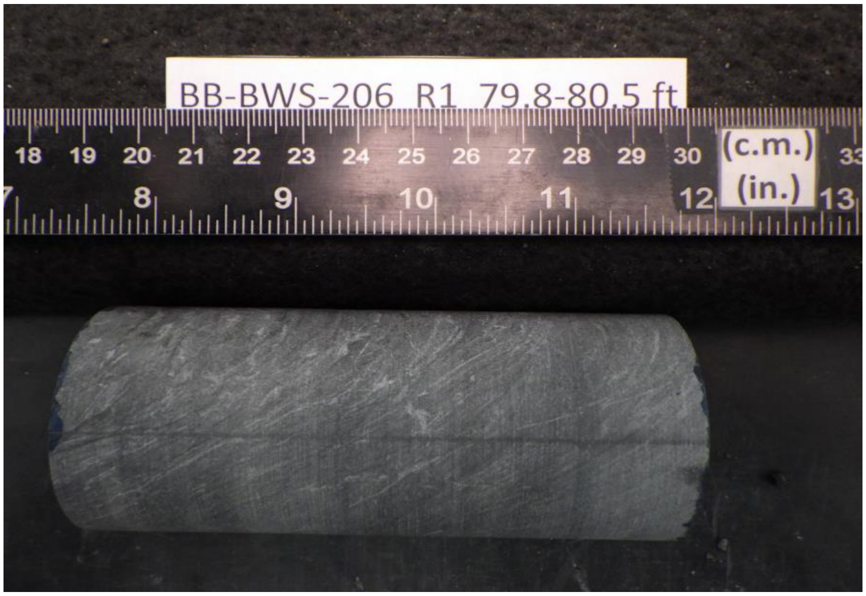
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00110	1.990	0.00055	0.032	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00120	1.990	0.00060	0.035	YES

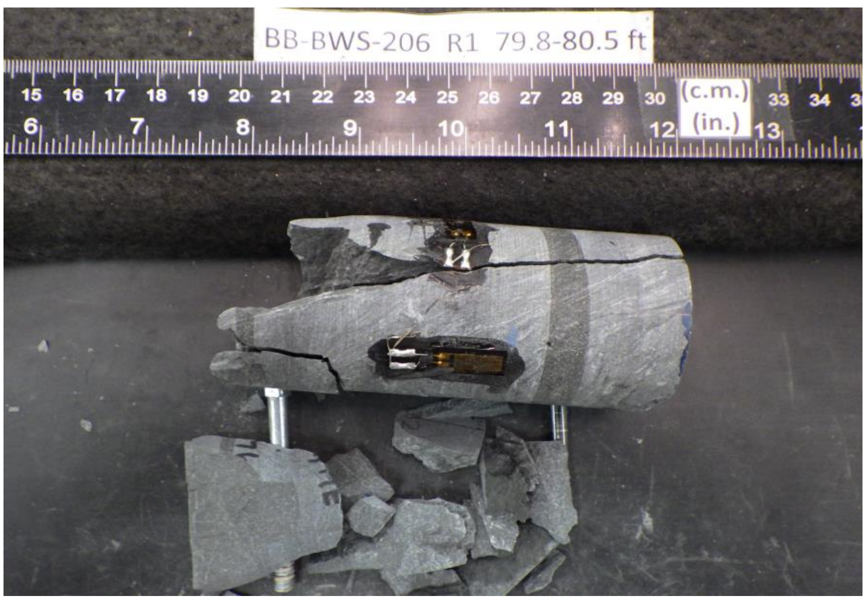
Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-206
Sample ID:	R1
Depth, ft:	79.8-80.5



After cutting and grinding



After break

Elastic Compression of Steel H-Piles

Client: Maine Department of Transportation

Project: Wilson Street Bridge Replacement - WIN No. 018915.20

Subject: Elastic Compression for Piles

PROBLEM STATEMENT & OBJECTIVE

Estimate settlement under service vertical loads for substructure foundation piles.

EXECUTIVE SUMMARY

The total elastic settlement for a single pile at Abutment 1 is =	0.20	in.
The total elastic settlement for a single pile at the Pier is =	0.17	in.
The total elastic settlement for a single pile at Abutment 2 is =	0.23	in.

ASSUMPTIONS

1. The H-Pile section is HP14x117 with no corrosion loss.
2. Top of pile elevation will be taken as the elevation at the bottom of the pile cap at each abutment.
3. Top of bedrock depths per borings BB-BWS-202 (Abutment No. 1), BB-BWS-203 (Pier), and BB-BWS-206 (Abutment No. 2).

AVAILABLE INFORMATION:

1. Haley & Aldrich boring logs BB-BWS-202, BB-BWS-203, and BB-BWS-206.
2. Emails from MaineDOT with abutments and pier maximum service axial loads of 284 kips and 326 kips, respectively.
3. 60% draft plan set titled "Wilson Street Bridge over I-395 and Route 9, Route 1A, 018915.20,
Project Length 0.489 mi., Bridge No. 1564."

PROCEDURE

$$\delta = PL/AE$$

δ = settlement
P = vertical service load on pile
L = length of pile
A = cross sectional area of pile
E = elastic modulus of pile

CALCULATIONS:

	Abutment No. 1	Pier	Abutment No. 2	
Max Vertical Service Load, P:	284.0	326.0	284.0	kips
Elevation of Top of Pile:	131.0	111.0	121.6	ft
Elevation of Tip of Pile:	71.3	66.7	55.6	ft
Length, L:	59.7	44.3	66.0	ft
Area of Pile, A:	34.4	34.4	34.4	in^2
Elastic Modulus of Pile, E:	29,000	29,000	29,000	ksi
Total Settlement of Pile, δ_{total}:	**0.20**	**0.17**	**0.23**	**in.**

Lateral Earth Pressure Coefficients

HALEY ALDRICH

CALCULATIONS		File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	16-Mar-2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	NAS
Subject:	Lateral Earth Pressures	Checked by:	BCS

PASSIVE LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 with interim revisions. See plot below.

ϕ'_f (deg)	θ (deg)	δ (deg)	$-\delta/\phi'_f$	k_p	R	$R\cdot k_p$
32	90	24	-0.75	7.7	0.86	6.6

Note: $R\cdot k_p$ should only be applied to effective earth pressures (i.e., do not apply to hydrostatic pressures).

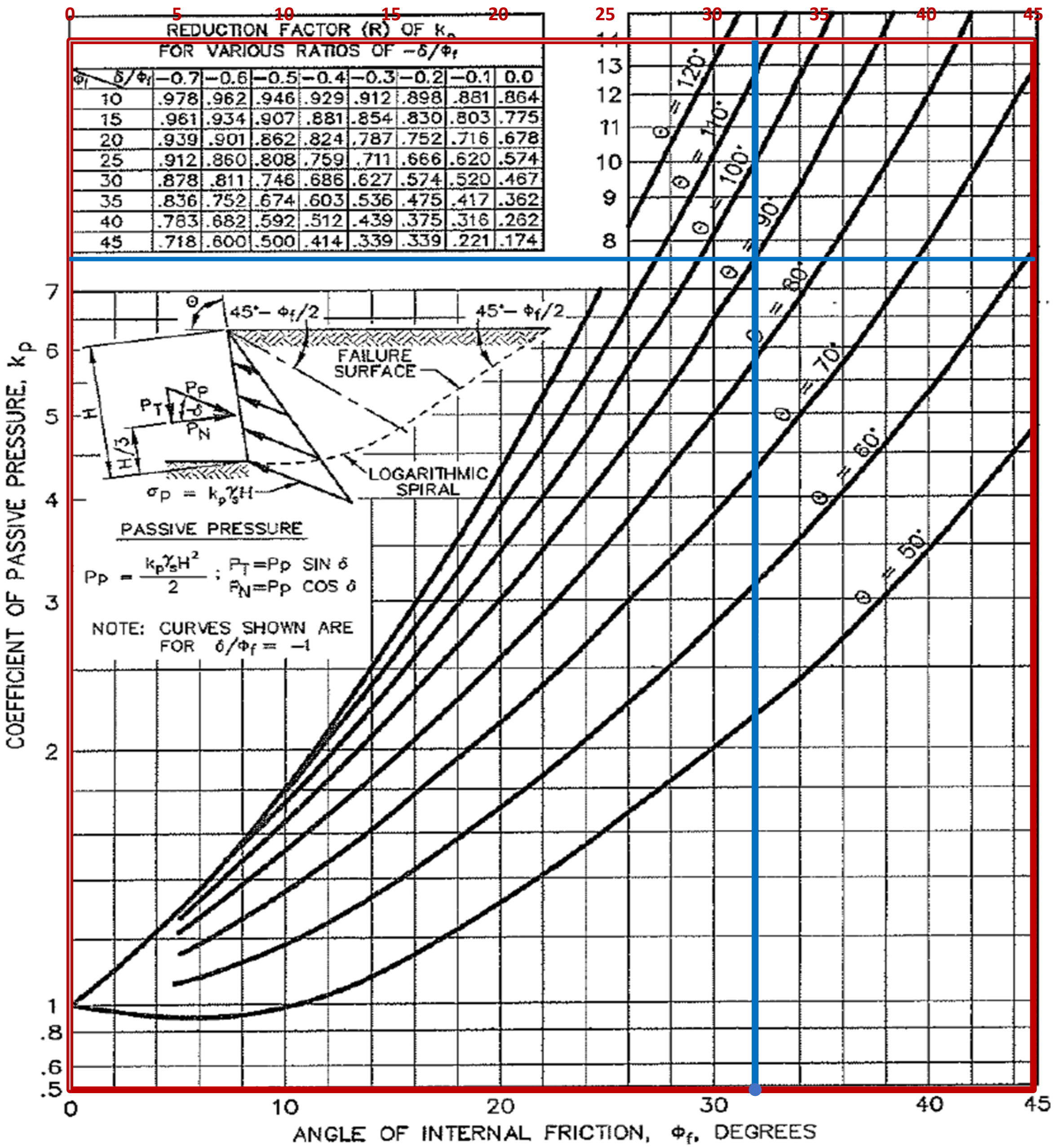


Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)

	CALCULATIONS	File No.:	132076-005
		Sheet:	2 of 2
Client:	Maine Department of Transportation	Date:	16-Mar-2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	NAS
Subject:	Lateral Earth Pressures	Checked by:	BCS

SEISMIC LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 with interim revisions. Excerpts below.

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - \beta)}{\cos\theta_{MO}\cos^2\beta\cos(\delta+\beta+\theta_{MO})} \times \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_{MO}-i)}{\cos(\delta+\beta+\theta_{MO})\cos(i-\beta)}}\right]^{-2} \quad \text{(A11.3.1-1)}$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1-k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

Note that K_{AE} includes the static (k_a) and dynamic (Δk_{AE}) components of the lateral soil pressure. According to Section A11.3.1 of AASHTO LRFD, a reasonable approach for routine walls is to apply the combined resultant of the static and seismic force at the same location as the static earth pressure but no less than h/3. Assuming hydrodynamic effects are negligible, total soil unit weight should be used when calculating the static+seismic force using K_{AE}.

CALCULATIONS

ϕ'_f (deg)	PGA (g)	F_{PGA}	k_h (g)	k_v (g)	θ_{MO} (deg)	δ (deg)	β (deg)	i (deg)	K_{AE}
32	0.066	1.20	0.04	0.00	2.3	24	0.0	0.00	0.30

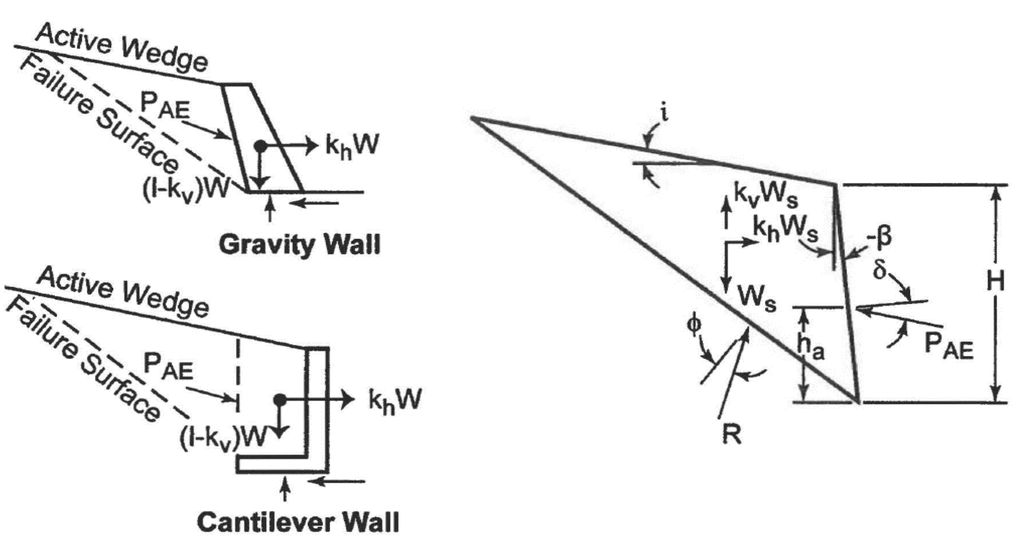


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

Wave Equation Analysis

PROBLEM STATEMENT & OBJECTIVE

Perform impact hammer drivability analyses for the HP14x117 steel H-Piles proposed. Calculate penetration resistances and driving stresses at the target geotechnical resistance.

EXECUTIVE SUMMARY

An impact driving hammer that can transfer about 44 kip*ft of energy to an HP14x117 steel pile can drive the pile to rock and achieve the target pile nominal resistances of 609 kips at the abutment and 632 kips at the pier without overstressing the pile.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2017.
2. Maine DOT standard specifications.
3. AISC Steel design manual.
4. GRL WEAP 2010 software manual.

AVAILABLE INFORMATION

1. Recent boring logs: BB-BWS-202 (Abutment 1), BB-BWS-203 (Pier), BB-BWS-206 (Abutment 2).
2. Strength Limit State maximum pile load at abutment from MEDOT equal to 396 kips (received by email from MEDOT).
3. Strength Limit State maximum pile load at pier from MEDOT equal to 411 kips (received by email from MEDOT).

ASSUMPTIONS

1. Elevation Units and Datum: feet, North American Vertical Datum of 1988 (NAVD88).
2. Soil conditions at each substructure are based on the borings indicated above.
3. Piles will be driven to top of rock. Pile section is an HP14x117.
   Two models for geotechnical resistance distribution were considered near the end of driving:
   - Before pile tip reaches rock, side resistance is much greater than tip resistance.
   - Pile tip reaches rock, tip resistance is much greater than side resistance.
3. Piles are driven from ground surface which ranges from El. 111.2 to 143.8 at the substructure locations.
4. Top of rock at the substructure locations range from El. 55.6 to 71.3.
5. Groundwater assumed at El. 106.5.
6. Pile embedment is based on estimated depth from ground surface to top of rock at each location.
   Pile length is based on pile embedment plus 5 ft.
8. Factored resistances are based on factored pile STR loads from ME DOT at the abutments and pier.
9. Nominal resistance (set as the target ultimate resistance in WEAP) is taken as the factored resistance divided by 0.65 (resistance factor for CAPWAP dynamic testing).
10. Shaft (skin) quake and damping values used are the suggested values in WEAP.
    Toe (end) quake and damping values used are suggested values in WEAP. The smaller quake value represents the condition where the pile tip reaches rock.
11. Acceptable penetration resistance according to MEDOT standard specifications is 3 to 15 bl/in.
12. Pile hammer energy was adjusted in order to produce penetration resistance results ranging from 3.5 to 12 bl/in.
13. Limit pile compressive stress is assumed to be 0.9fy or 45 ksi for fy=50 ksi steel.

CALCULATIONS AND RESULTS

(see next page)

CALCULATIONS AND RESULTS

Results: Select hammer setting to produce penetration resistance on the lower range of 3 to 15 bl/in.

Structure	Boring	Case	Hammer	Rated Energy (kip*ft)	GS El. (ft)	GW El. (ft)	Top of Rock El. (ft)	Pile Type	Pile L (ft)	Pile emb. (ft)	Factored Res. (kip)	Nominal Res. (kip)	Shaft Quake (in.)	Toe Quake (in.)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Side Resist. (%)	Pen. Res. (bl/ft)	Pen. Res. (bl/in.)	Max. Comp. Stress (ksi)	WEAP Enthru (kip*ft)
Abut 1	BB-BWS-202	Hi shaft	D36-32	90.6	143.8	106.5	71.3	HP14x117	78	73	396	609	0.1	0.1	0.09	0.15	88.0	41.8	3.5	37.1	37.6
		Hi tip	D36-32	90.6	143.8	106.5	71.3	HP14x117	78	73	396	609	0.1	0.04	0.09	0.15	20.0	44.4	3.7	39.2	43.3
Pier	BB-BWS-203	Hi shaft	D36-32	90.6	111.2	106.5	66.7	HP14x117	50	45	411	632	0.1	0.1	0.17	0.15	79.0	68.3	5.7	41.0	35.6
		Hi tip	D36-32	90.6	111.2	106.5	66.7	HP14x117	50	45	411	632	0.1	0.04	0.17	0.15	20.0	50.1	4.2	40.4	39.4
Abut 2	BB-BWS-206	Hi shaft	D36-32	90.6	134.6	106.5	55.6	HP14x117	85	80	396	609	0.1	0.1	0.1	0.15	87.0	43.6	3.6	37.4	38.0
		Hi tip	D36-32	90.6	134.6	106.5	55.6	HP14x117	85	80	396	609	0.1	0.04	0.1	0.15	20.0	45.8	3.8	38.0	43.6

Notes:

1. Soil conditions at each substructure are based on the borings indicated.
2. Hammer used in analysis is a Delmag diesel hammer with model and rated energy shown above.
3. Cases described as Hi shaft (side resistance > tip resistance during driving) and Hi tip (tip resistance > side resistance during driving, e.g., when pile reaches rock).
4. Piles are driven from ground surface shown (GS El.). Groundwater assumed at the elevation indicated (GW El.).
5. Pile embedment is based on estimated depth from ground surface to top of rock at each location. Pile length is based on pile embedment plus 5 ft.
6. Factored resistances are based on factored pile STR loads from ME DOT at the abutments and pier.
7. Nominal resistance (set as the target ultimate resistance in WEAP) is taken as the factored resistance divided by 0.65 (resistance factor for CAPWAP dynamic testing).
8. Shaft quake and damping are suggested values in WEAP.
9. Toe quake and damping are suggested values in WEAP. The smaller quake value represents the condition where the pile tip reaches rock.
10. Two side resistance percentages (as a fraction of the total nominal resistance) are considered. Higher side resistance fraction (WEAP calculated) represents condition before pile reaches rock. Lower side resistance fraction (assumed 20%) represents condition when pile reaches rock.
11. Penetration resistance falls within MEDOT required 3 to 15 bl/in.
12. Limit compressive stress is assumed to be 0.9fy or 45 ksi for fy=50 ksi steel.
13. Max. transferred energy in WEAP also shown.

Abutment 1

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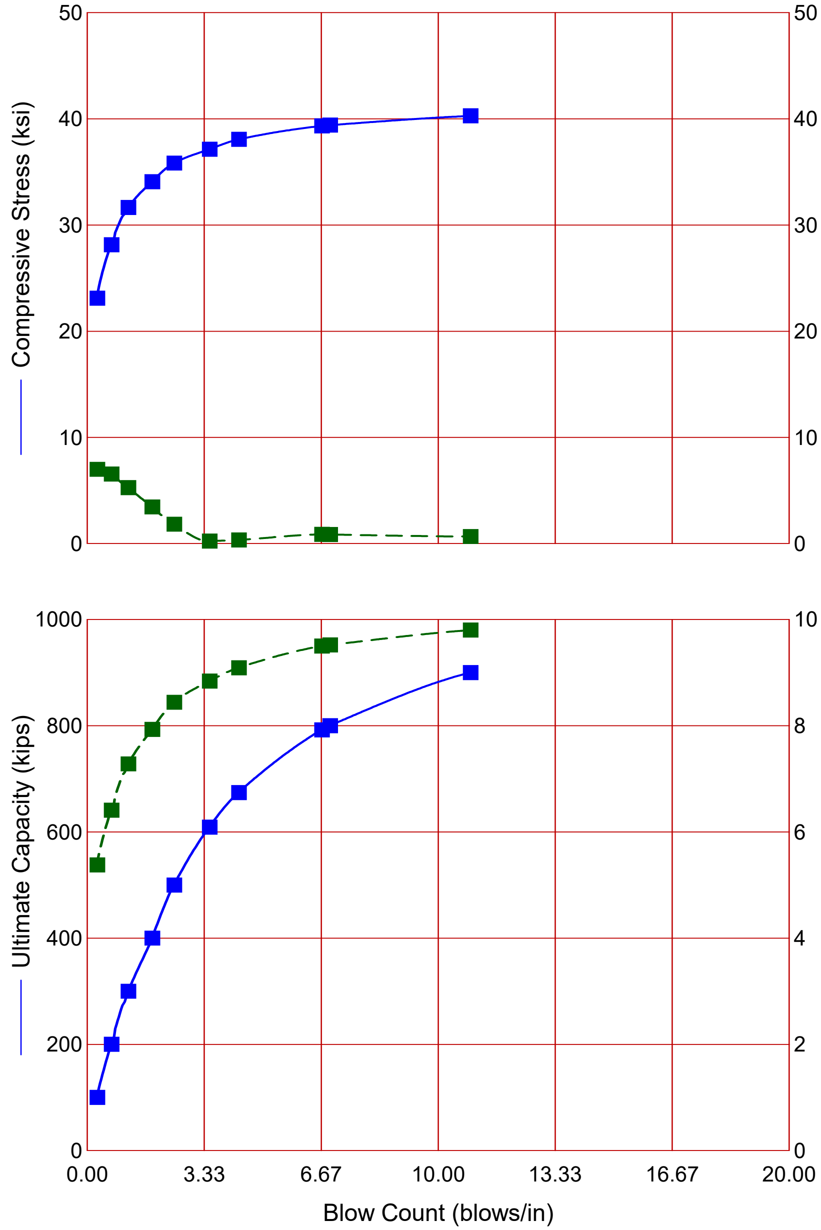
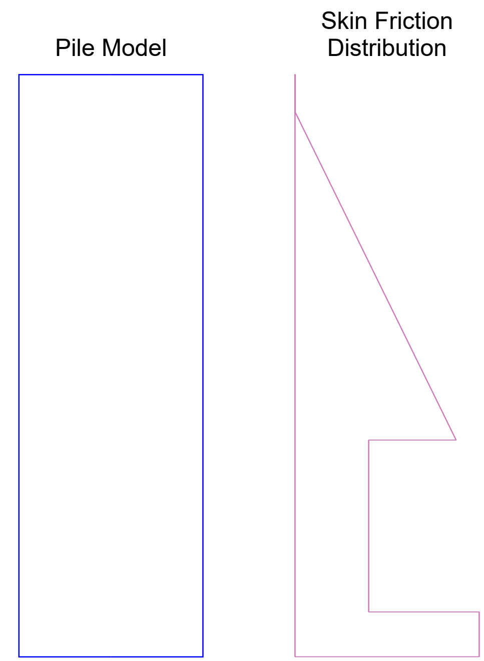


DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	78.00	ft
Pile Penetration	73.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 88 %
(Proportional)

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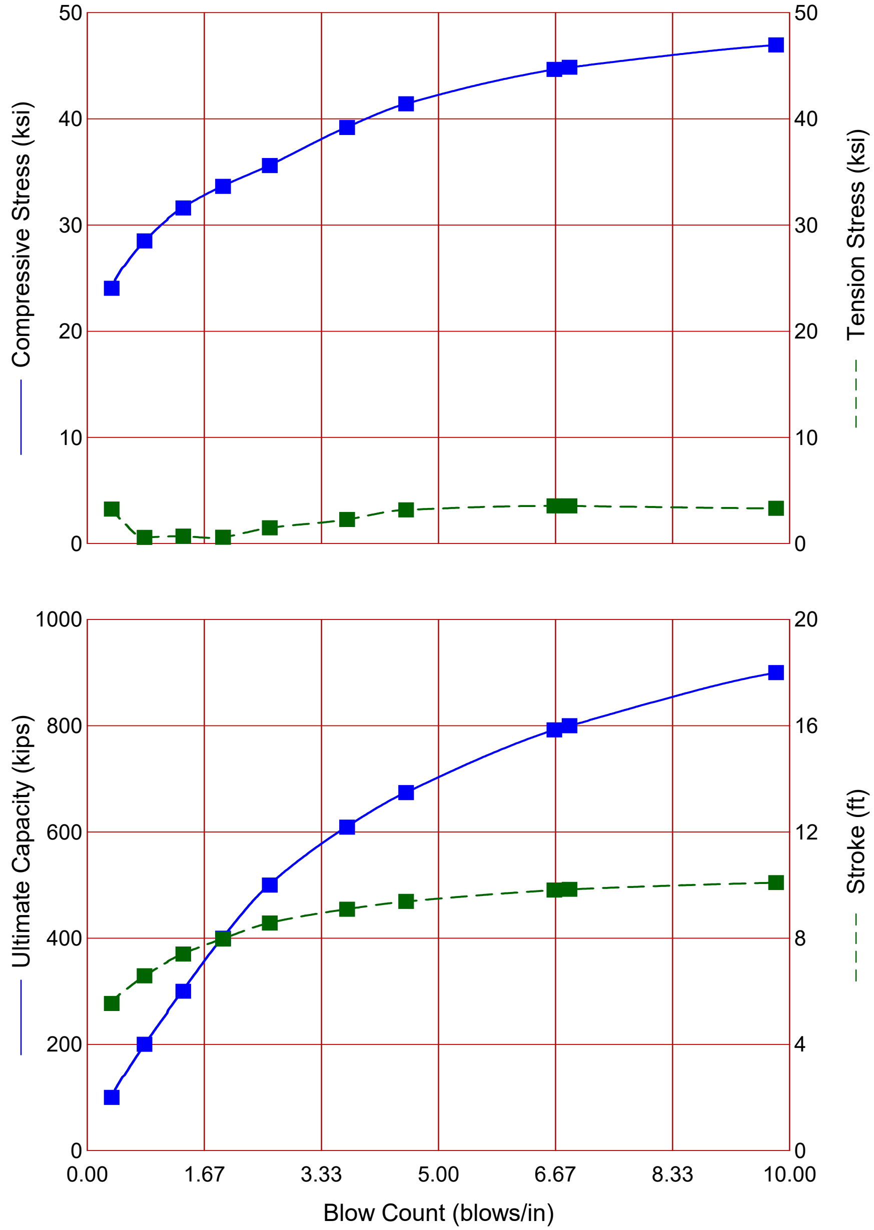
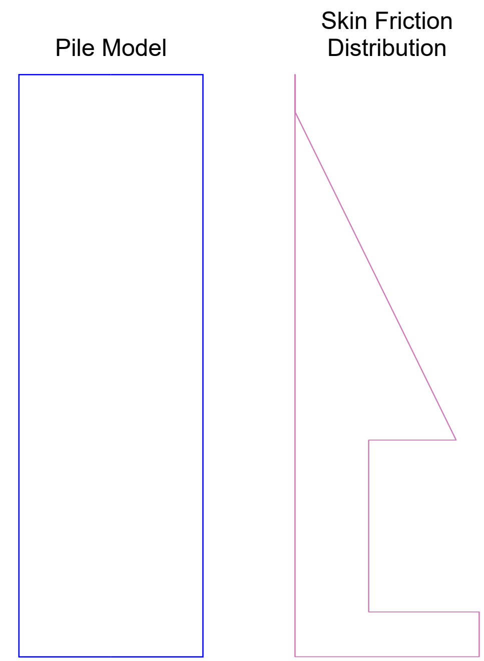
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	23.11	7.00	0.3	5.38	47.20
200.0	28.14	6.56	0.7	6.41	41.79
300.0	31.65	5.27	1.2	7.28	39.73
400.0	34.08	3.46	1.9	7.93	38.30
500.0	35.83	1.84	2.5	8.44	36.80
609.0	37.12	0.24	3.5	8.84	37.55
674.0	38.05	0.35	4.3	9.09	38.10
792.0	39.33	0.88	6.7	9.50	39.10
800.0	39.40	0.86	6.9	9.52	39.16
900.0	40.27	0.67	10.9	9.80	39.89



DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	78.00	ft
Pile Penetration	73.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 20 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	24.04	3.25	0.3	5.54	46.35
200.0	28.49	0.59	0.8	6.58	41.22
300.0	31.60	0.70	1.4	7.40	39.80
400.0	33.64	0.60	1.9	7.97	39.79
500.0	35.59	1.49	2.6	8.57	41.35
609.0	39.18	2.27	3.7	9.09	43.31
674.0	41.40	3.19	4.5	9.38	44.46
792.0	44.64	3.56	6.6	9.81	46.27
800.0	44.84	3.55	6.9	9.84	46.26
900.0	46.95	3.33	9.8	10.09	47.26

Pier

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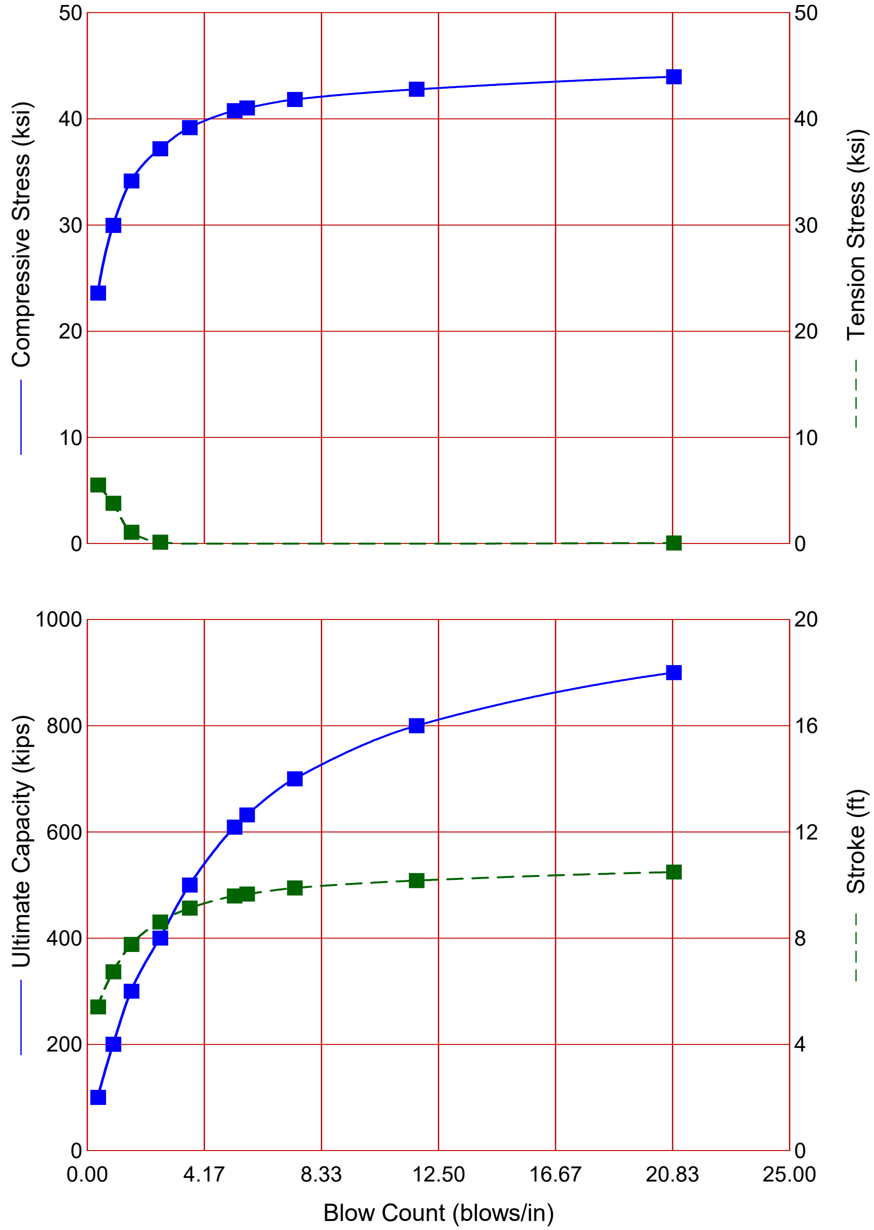
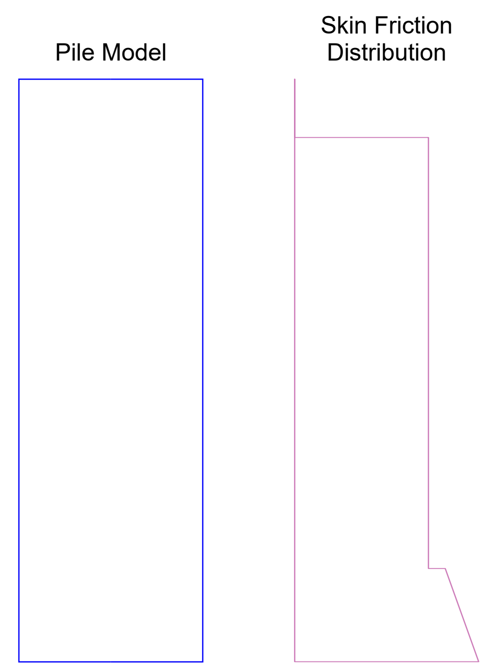


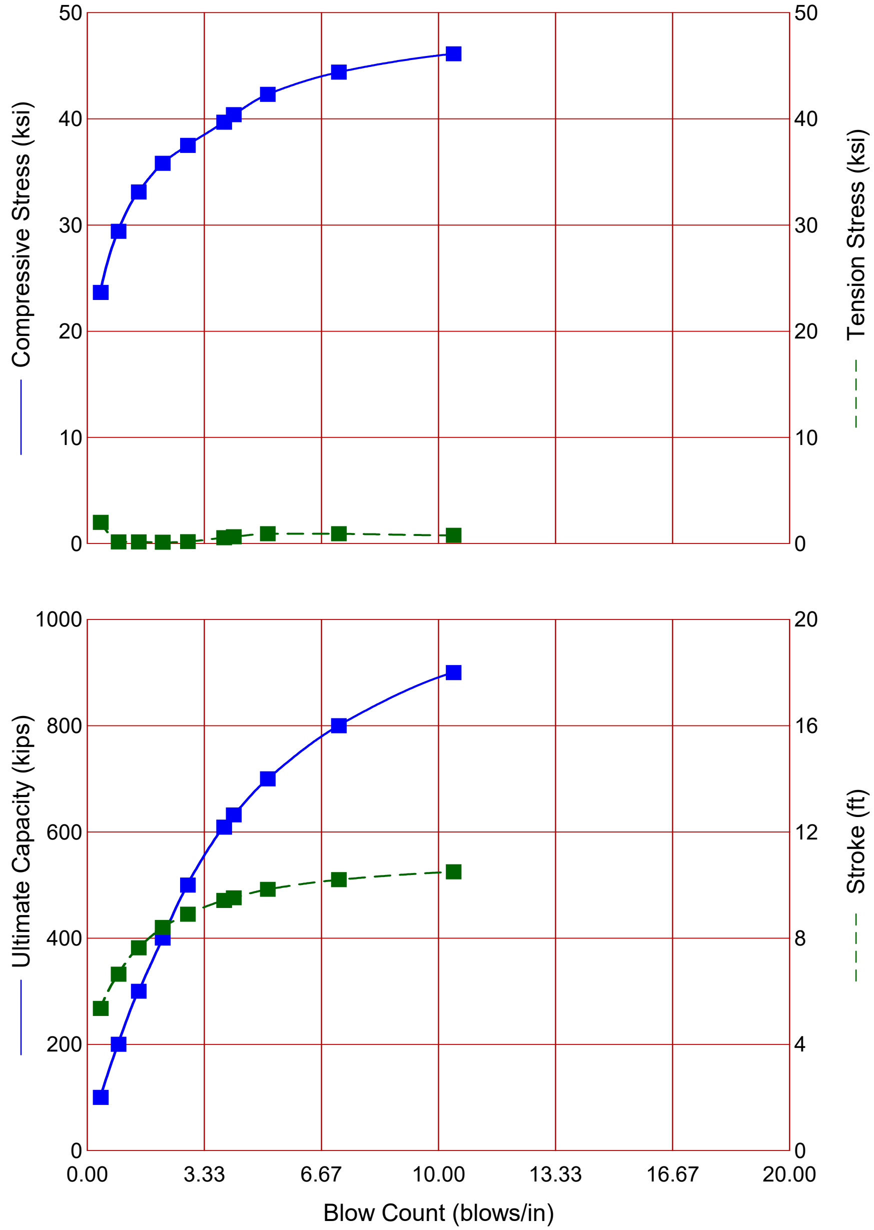
DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.168	s/ft
Toe Damping	0.150	s/ft
Pile Length	50.00	ft
Pile Penetration	45.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 79 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	23.59	5.52	0.4	5.41	46.42
200.0	29.97	3.80	0.9	6.73	40.38
300.0	34.13	1.08	1.6	7.76	37.56
400.0	37.17	0.14	2.6	8.61	36.25
500.0	39.16	0.00	3.6	9.13	35.06
609.0	40.76	0.00	5.3	9.59	35.46
632.0	41.02	0.00	5.7	9.66	35.63
700.0	41.82	0.00	7.4	9.89	36.08
800.0	42.77	0.00	11.7	10.17	36.64
900.0	43.95	0.06	20.9	10.49	37.18

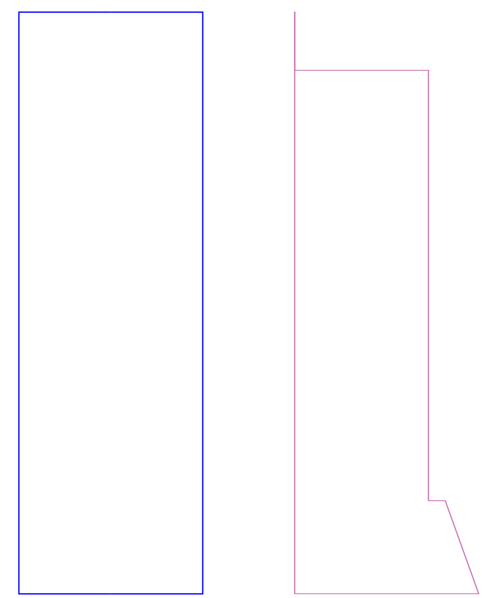


DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.168	s/ft
Toe Damping	0.150	s/ft
Pile Length	50.00	ft
Pile Penetration	45.00	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	23.65	2.00	0.4	5.35	46.68
200.0	29.40	0.17	0.9	6.64	40.95
300.0	33.11	0.17	1.5	7.64	38.71
400.0	35.80	0.13	2.1	8.40	37.98
500.0	37.49	0.19	2.9	8.90	38.31
609.0	39.67	0.56	3.9	9.42	39.16
632.0	40.38	0.66	4.2	9.52	39.38
700.0	42.29	0.94	5.1	9.84	40.55
800.0	44.38	0.94	7.2	10.20	41.97
900.0	46.11	0.79	10.4	10.50	43.07

Abutment 2

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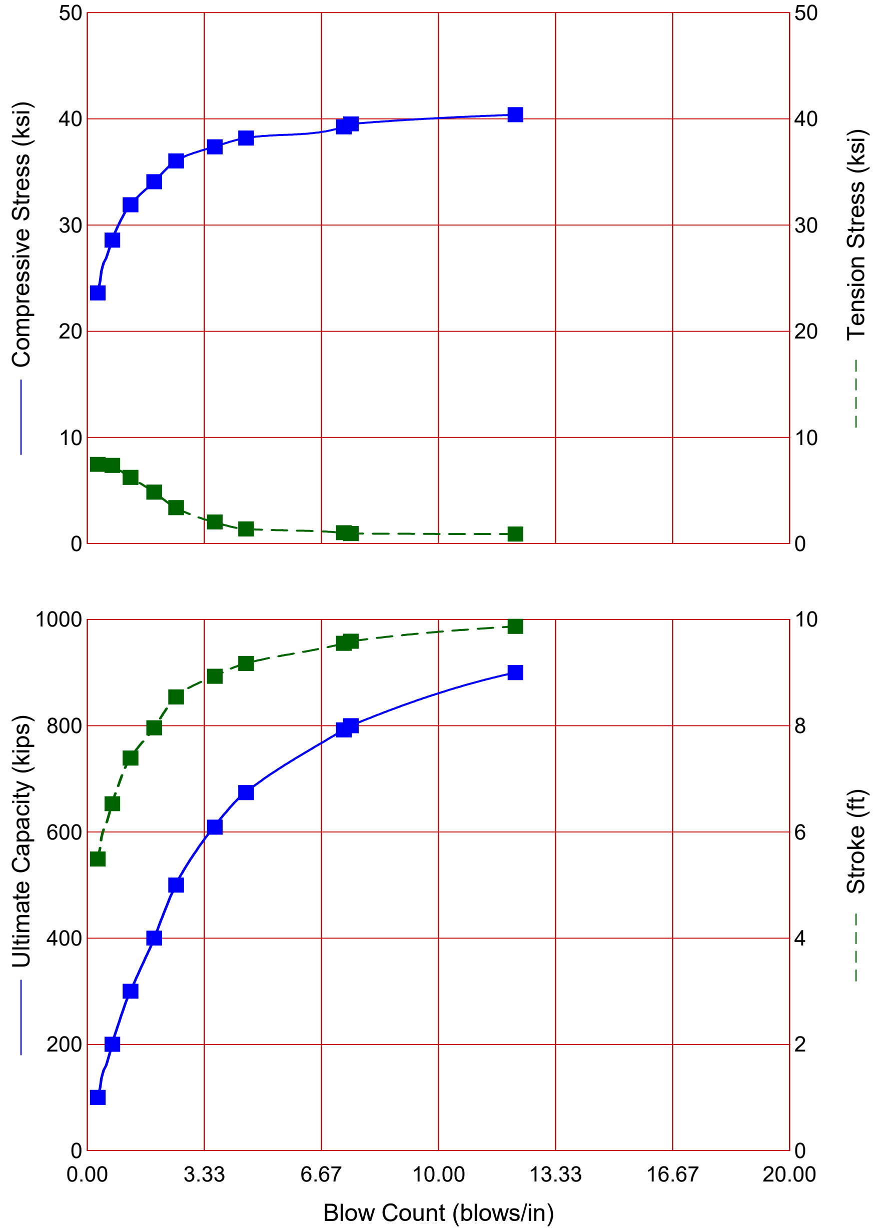
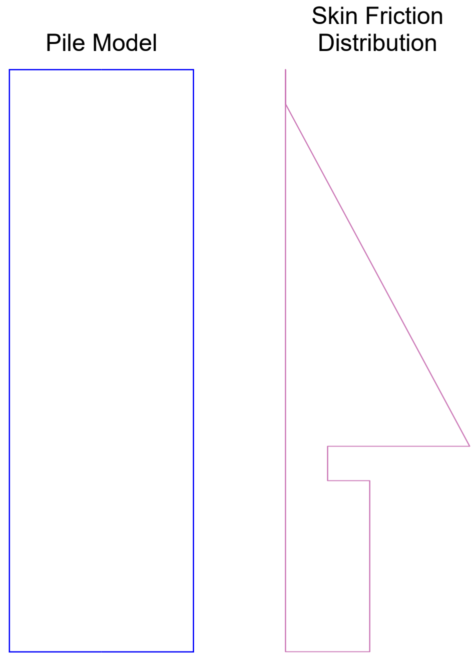


DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.100	s/ft
Toe Damping	0.150	s/ft
Pile Length	85.00	ft
Pile Penetration	80.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 87 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	23.61	7.47	0.3	5.49	46.56
200.0	28.57	7.37	0.7	6.53	41.21
300.0	31.90	6.25	1.2	7.39	39.45
400.0	34.07	4.85	1.9	7.96	37.36
500.0	36.03	3.39	2.5	8.54	37.22
609.0	37.36	2.04	3.6	8.93	37.98
674.0	38.18	1.39	4.5	9.17	38.65
792.0	39.23	1.03	7.3	9.55	39.42
800.0	39.51	0.96	7.5	9.59	39.66
900.0	40.38	0.91	12.2	9.87	40.45

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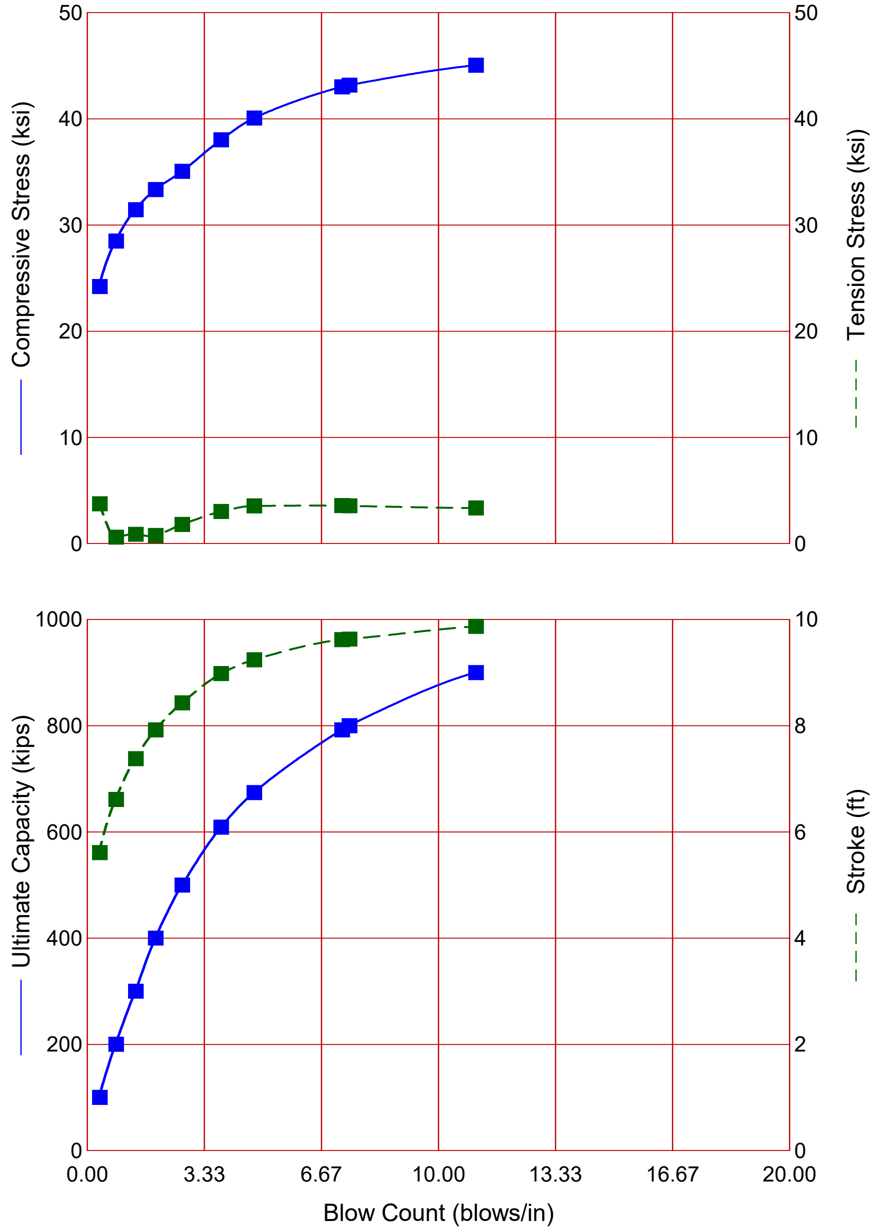
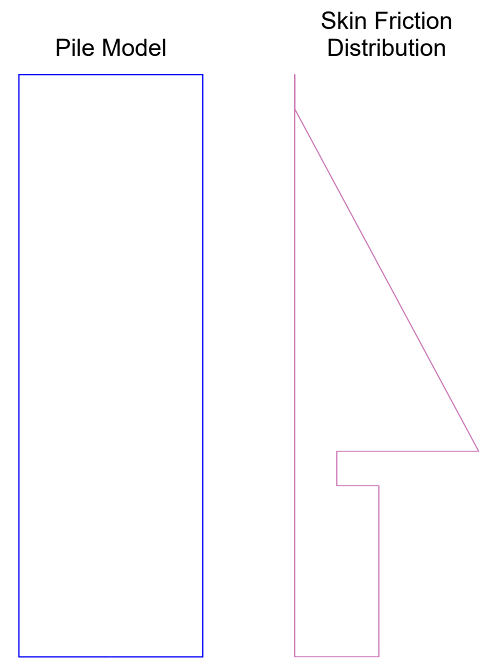


DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1500 (100%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.100	s/ft
Toe Damping	0.150	s/ft
Pile Length	85.00	ft
Pile Penetration	80.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 20 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	24.21	3.75	0.4	5.61	46.01
200.0	28.49	0.61	0.8	6.61	41.21
300.0	31.43	0.88	1.4	7.38	39.85
400.0	33.34	0.77	1.9	7.92	39.83
500.0	35.05	1.81	2.7	8.43	41.26
609.0	38.02	3.03	3.8	8.98	43.61
674.0	40.06	3.56	4.8	9.24	44.45
792.0	43.01	3.58	7.3	9.62	45.82
800.0	43.16	3.56	7.5	9.63	45.90
900.0	45.04	3.36	11.1	9.87	46.72

Lateral Pile Evaluations

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 6
Client:	Maine Department of Transportation	Date:	02MAR2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	NAS
Subject:	Lateral Analysis of Abutment H-Piles Under Expansion & Contraction Displacement	Checked by:	JLL

PROBLEM STATEMENT & OBJECTIVE

Determine the bending moment and shear in the Abutment 1 and 2 H-Pile foundations induced by the anticipated deck expansion and contraction lateral displacements.

EXECUTIVE SUMMARY

The prescribed displacements along with a fixed-head assumption and linear elastic (non-yielding) pile model combine to result in excessively high lateral forces and bending moments in the pile.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2017 Edition.
2. FHWA NHI-06-088, NHI Course No. 132012, Soils and Foundations, Reference Manual - Volume 1, December 2006.
3. Ensoft, Inc., LPile 2018, Technical Manual.
4. Effects of Soil Slope on Lateral Capacity of Piles in Cohesive and Cohesionless Soils, Nimityongskul et al. (2012), Caltrans Report No. CA-11-0932. (P-multipliers applied to free-field to model effect of sloped ground surface).

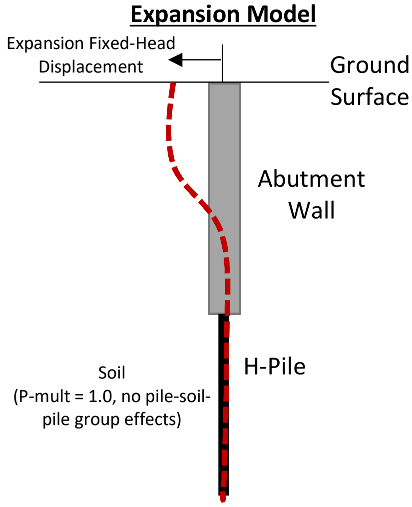
AVAILABLE INFORMATION

1. Recent boring logs BB-BWS-202 (Abutment 1) and BB-BWS-206 (Abutment 2) and related lab testing results.
2. Site & Subsurface Exploration Location Plan & Interpretive Subsurface Profile for I-395 Route 9 Connection I-95 Brewer-Eddington Penobscot County, Sheets 2 and 3 from Bridge Plans, dated 19 February 2020.
3. Abutment No. 1, Sheet 51 of Bridge Plans, dated 19 February 2020.
4. Pile type, orientation, axial load, and horizontal contraction and expansion translations from MaineDOT, by email, dated 11 February 2020.
5. Additional information re: model fixity, girder elevations, and preferred output from MaineDOT, by email, dated 14 February 2020.

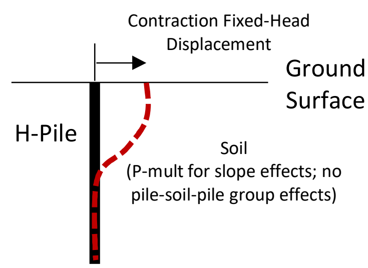
ASSUMPTIONS

1. Elevation Units and Datum: feet, North American Vertical Datum of 1988 (NAVD88).
2. Single pile model using LPile. Piles are driven to bedrock.
3. Bridge Plans show 3% grade (1.7 deg. slope) but assumed flat ground in our LPile analyses for simplicity.
4. Pile-soil-pile lateral effects (i.e., group effects) are negligible because piles are widely spaced (>6 times pile dimension).
5. P-multipliers are used to consider sloping ground effect for the contraction case (from reference 4).
6. Piles are HP14x117 oriented such that deck contraction and expansion cause bending about the pile weak axis.
7. No corrosion reduction on the pile section.
8. Piles are linear-elastic, non-yielding (i.e., no ceiling on moment generated within the pile).
9. Piles are "fixed" to the wall/pile cap.
10. Top of model element restrained against rotation but allowed to translate.
11. Abutment and pile are modeled in the expansion case; only pile is modeled in contraction case.
12. Contraction and expansion cases are modeled differently in LPile as illustrated schematically below:

Expansion Model



Contraction Model



ASSUMPTIONS (continued)

13. Four models were created in LPile representing the following:
    - Abutment 1, Expansion
    - Abutment 1, Contraction
    - Abutment 2, Expansion
    - Abutment 2, Contraction
14. For the **expansion** LPile models, the following structural elements and section properties were used:

Model Section	Section Type	Width (in.)	Depth (in.)	A (in.^2)	I (in.^4)	E (ksi)
1	Abutment Wall	112	42	4704	691488	4031
2	H-Pile	14	14	34.4	443	29000

Note: Width of wall section is based on the average pile spacing on the drawings which ranged from 8 ft to 11 ft.

For the **contraction** LPile models, only the H-Pile element is used with the following properties:

Model Section	Section Type	Width (in.)	Depth (in.)	A (in.^2)	I (in.^4)	E (ksi)
1	H-Pile	14	14	34.4	443	29000

15. Groundwater is assumed to be at El. 108 for Abutment 1 and El. 105 for Abutment 2. In LPile, total soil unit weights are used above groundwater and buoyant soil unit weights below groundwater to calculate p-y lateral soil springs.
16. LPile soil properties for the four models are shown below:

Abutment 1 Expansion (Wall and H-Pile)

Layer	Applicable Structure	LPile Model	Cohesive/ Cohesionless	Top El. (ft)	Depth from Top of Abut. Wall to Top of Layer (ft)	Total Unit Wt., γ_t (pcf)(note 3)	S_u or ϕ (psf, deg)	k or ε_{50} (pci, %)
1 & 2	Abut. Wall, H-Pile	Sand (Reese, et. al)	Cohesionless	144	0	125	32	225
3	H-Pile	Sand (Reese, et. al)	Cohesionless	128	16	120	34	158
4	H-Pile	Sand (Reese, et. al)	Cohesionless	108	36	120	34	93
5	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	100	44	120	4000	0.5
6	H-Pile	Sand (Reese, et. al)	Cohesionless	79	65	130	40	125

Note:
A portion of Layer 1&2 will be applicable to the Abutment Wall (i.e., passive resistance against the back of the wall).

Abutment 1 Contraction (H-Pile Only)

Layer	Applicable Structure	LPile Model	Cohesive/ Cohesionless	Top El. (ft)	Depth from top of Pile to Top of Layer (ft)	Total Unit Wt., γ_t (pcf)(note 3)	S_u or ϕ (psf, deg)	k or ε_{50} (pci, %)
1	H-Pile	Sand (Reese, et. al)	Cohesionless	131	0	125	32	225
2	H-Pile	Sand (Reese, et. al)	Cohesionless	128	3	120	34	158
3	H-Pile	Sand (Reese, et. al)	Cohesionless	108	23	120	34	93
4	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	100	31	120	4000	0.5
5	H-Pile	Sand (Reese, et. al)	Cohesionless	79	52	130	40	125

Overall Notes for the LPile Soil Parameters:

1. S_u = undrained shear strength, ϕ = soil internal friction angle, k = lateral subgrade modulus, ε_{50} = percent strain at 50% of shear strength
2. ϕ and k apply to Cohesionless type materials, S_u and ε_{50} apply to cohesive type materials.
3. Buoyant unit weights are used in LPile to calculate p-y curves for layers below the water table.

LPile soil properties for the four models continued…

Abutment 2 Expansion (Wall and H-Pile)

Layer	Applicable Structure	LPile Model	Cohesive/ Cohesionless	Top El. (ft)	Depth from Top of Abut. Wall to Top of Layer (ft)	Total Unit Wt., γ_t (pcf)(note 3)	S_u or ϕ (psf, deg)	k or ε_{50} (pci, %)
1	Abut. Wall, H-Pile	Sand (Reese, et. al)	Cohesionless	135	0	125	32	225
2	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	121	14	125	4000	0.5
3	H-Pile	Sand (Reese, et. al)	Cohesionless	105	30	130	38	125
4	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	85	50	120	2000	0.7
5	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	81	54	130	4000	0.5

Note:
A portion of Layer 1 will be applicable to the Abutment (i.e., passive resistance against the back of the Abutment).

Abutment 2 Contraction (H-Pile Only)

Layer	Applicable Structure	LPile Model	Cohesive/ Cohesionless	Top El. (ft)	Depth from top of Pile to Top of Layer (ft)	Total Unit Wt., γ_t (pcf)(note 3)	S_u or ϕ (psf, deg)	k or ε_{50} (pci, %)
1	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	121	0	125	4000	0.5
2	H-Pile	Sand (Reese, et. al)	Cohesionless	105	16	130	38	125
3	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	85	36	120	2000	0.7
4	H-Pile	Stiff Clay (w/out Free Water)	Cohesive	81	40	130	4000	0.5

Overall Notes for the LPile Soil Parameters:

1. S_u = undrained shear strength, ϕ = soil internal friction angle, k = lateral subgrade modulus, ε_{50} = percent strain at 50% of shear strength
2. ϕ and k apply to cohesionless type materials, S_u and ε_{50} apply to cohesive type materials.
3. Buoyant unit weights are used in LPile to calculate p-y curves for layers below the water table.

17. The model top and bottom elevations of the structural elements in each LPile model are shown below:

Abutment 1 Expansion (Wall and H-Pile)

Element	Top El. (ft)	Bottom El. (ft)	Length (ft)
Abutment	144	131	13
H-Pile	131	71	60

Abutment 1 Contraction (H-Pile Only)

Element	Top El. (ft)	Bottom El. (ft)	Length (ft)
H-Pile	131	71	60

Abutment 2 Expansion (Wall and H-Pile)

Element	Top El. (ft)	Bottom El. (ft)	Length (ft)
Abutment	135	121	14
H-Pile	121	56	65

Abutment 2 Contraction (H-Pile Only)

Element	Top El. (ft)	Bottom El. (ft)	Length (ft)
H-Pile	121	56	65

LPile soil properties for the four models continued…

18. For the contraction case, the lateral p-y of sloping ground was estimated using reference 4 (figure below), (using recommendations for cohesionless material even if we have mixed conditions, P-mult for pile located just behind crest of slope are about the same as that for cohesive material).

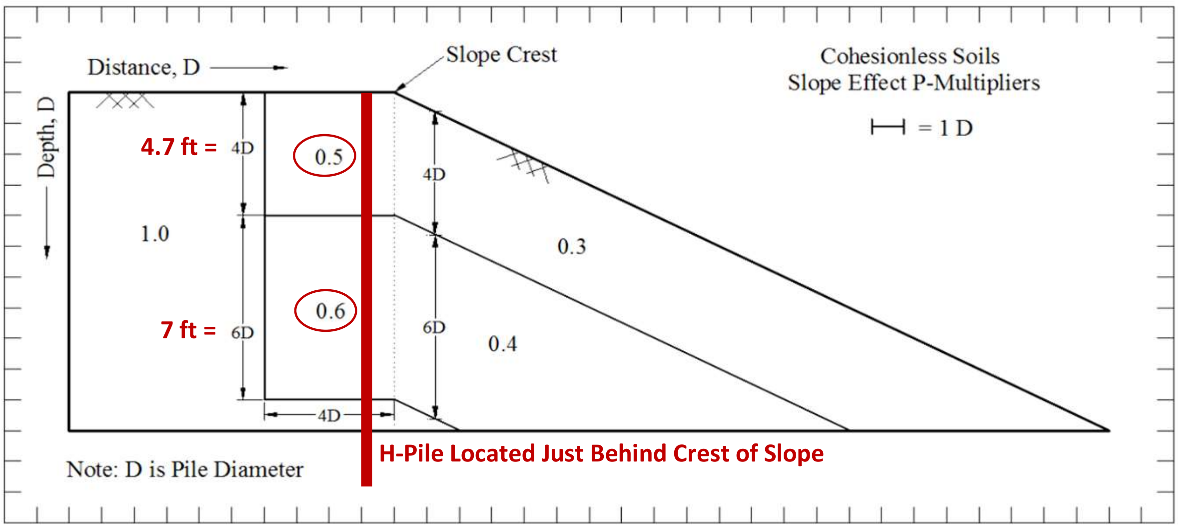


Figure 14-2 Recommended p-Multipliers for a Generalized Cohesionless Slope

SUMMARY OF LPILE RESULTS

Abutment	Case	Pile Top Lat. Displ. (in.)	Slope of Top Model Element (rad.)	Pile Axial Load (kip)	Pile Max. Shear (kip)	Pile Max. Moment (kip*ft)	Estimated Combined Stress (ksi)	
1	Expansion	0.89	0	396	286	531	112	excessive!
	Contraction	1.34	0	396	76	419	91	excessive!
2	Expansion	0.89	0	396	157	363	80	excessive!
	Contraction	1.34	0	396	109	490	104	excessive!

Notes:

Slope of zero at top of pile is equivalent to a "fixed" head condition (i.e., free to translate, but not rotate).

The shear and bending moment results above are for piles only, the wall forces and bending moments (expansion models) are not shown.

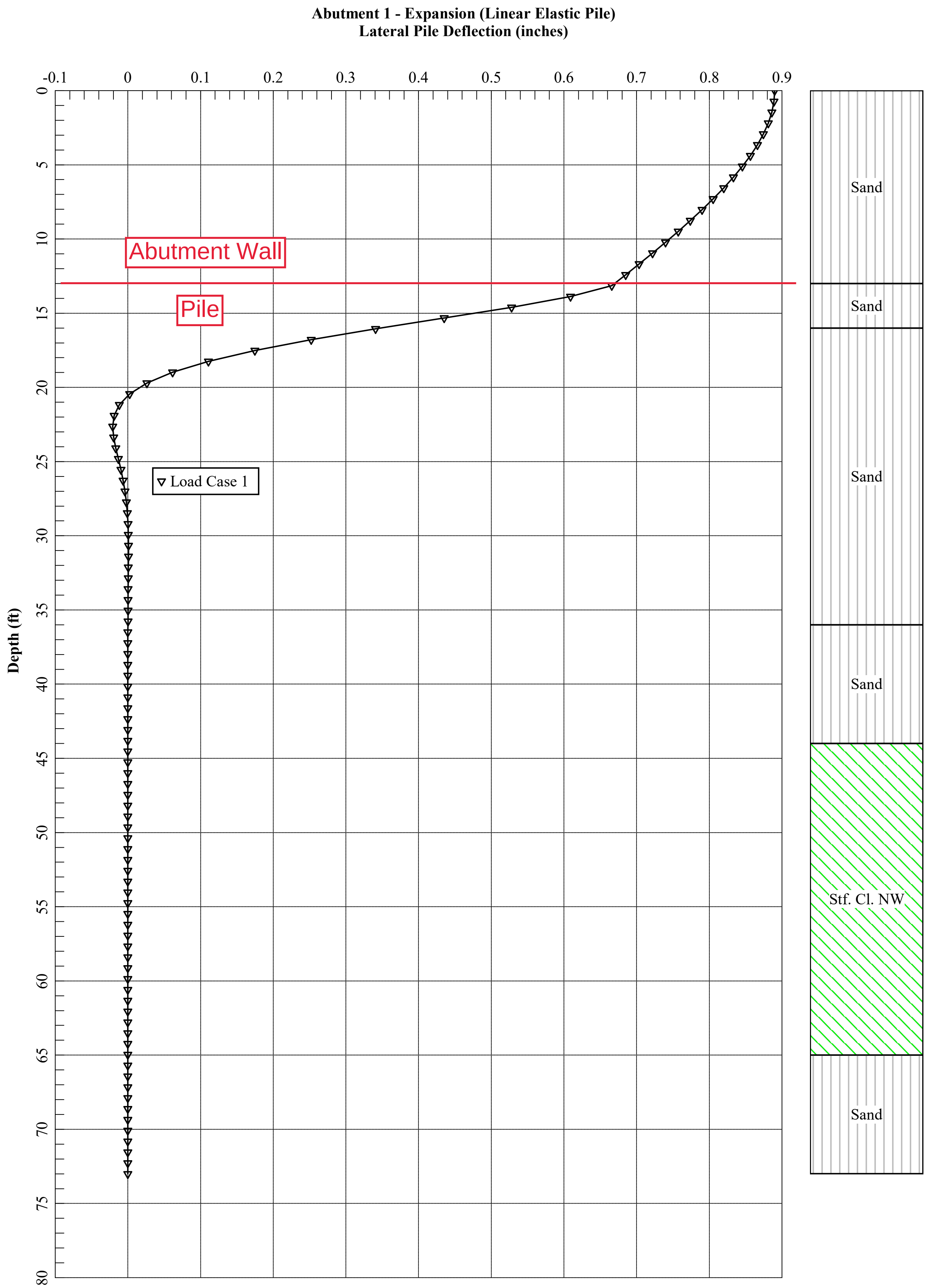
The prescribed displacements along with a fixed-head assumption and linear elastic (non-yielding) pile model combine to result in excessively high lateral forces and bending moments in the pile.

ATTACHMENTS

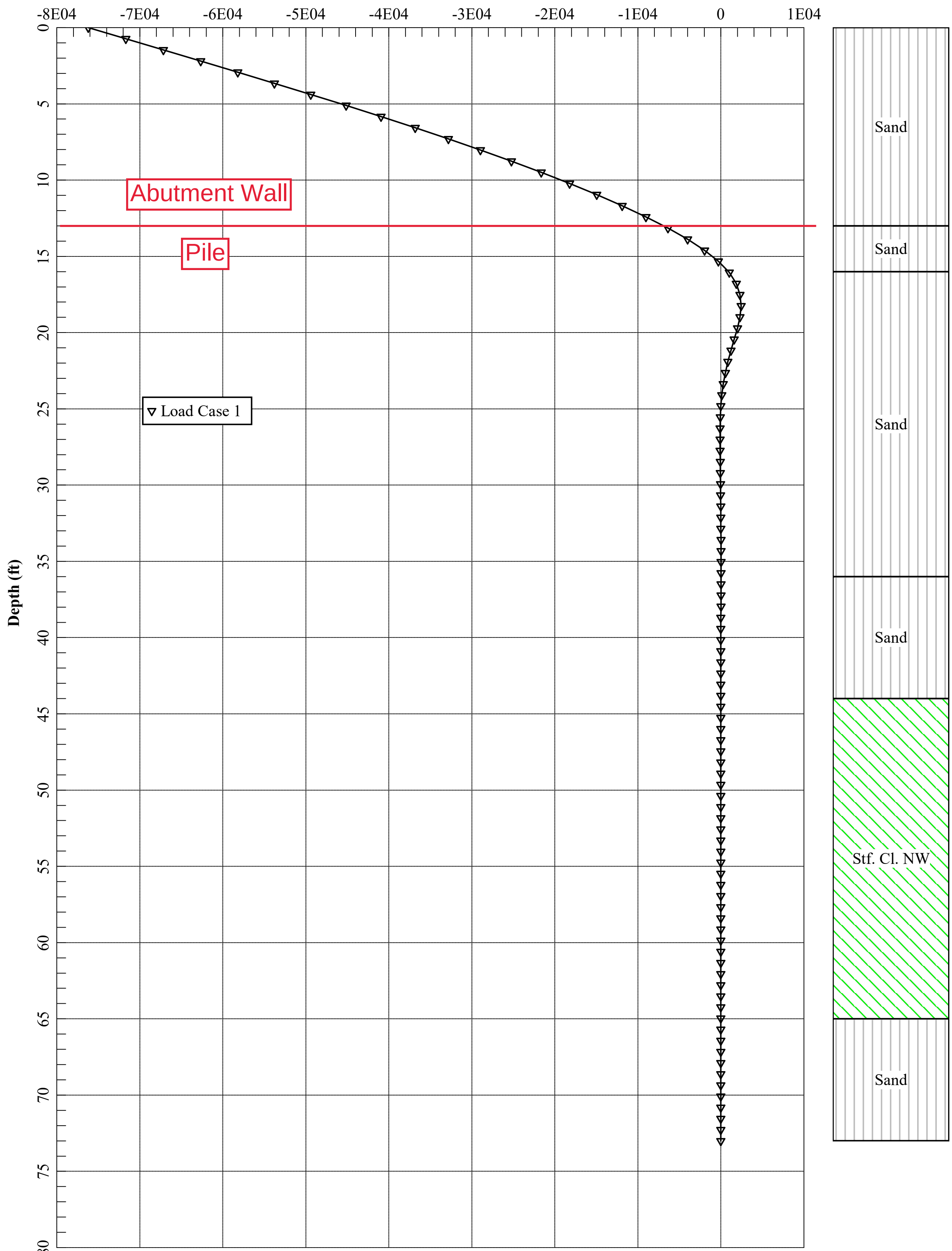
1. P-multiplier recommendations, selected pages from reference 4.
2. Recent boring logs BB-BWS-202 and BB-BWS-206.
3. Plots of LPile results (displacement, shear, and bending moment versus depth) for Abutment 1 and 2 contraction and expansion cases.

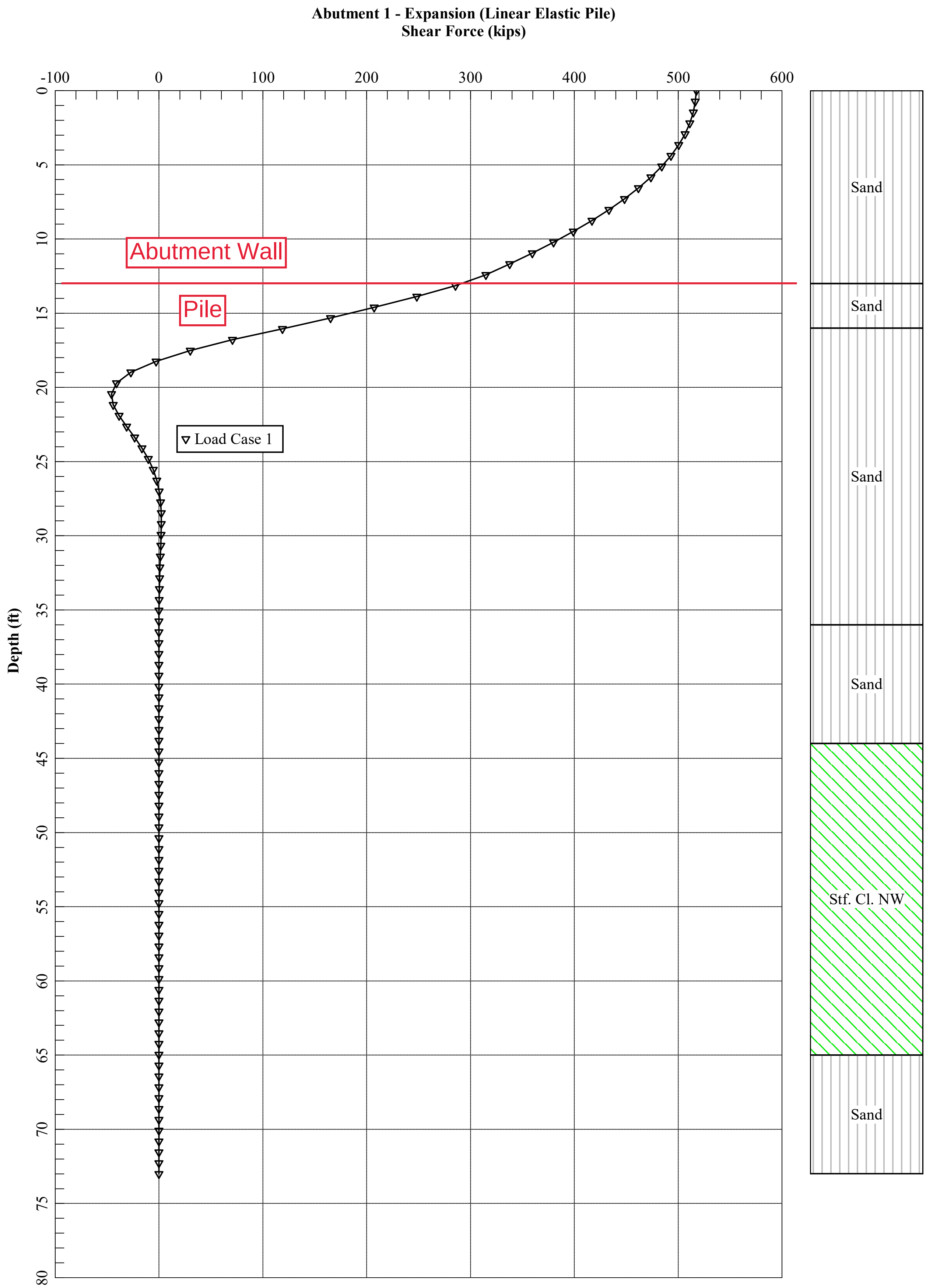
LPile Results

LPile Results

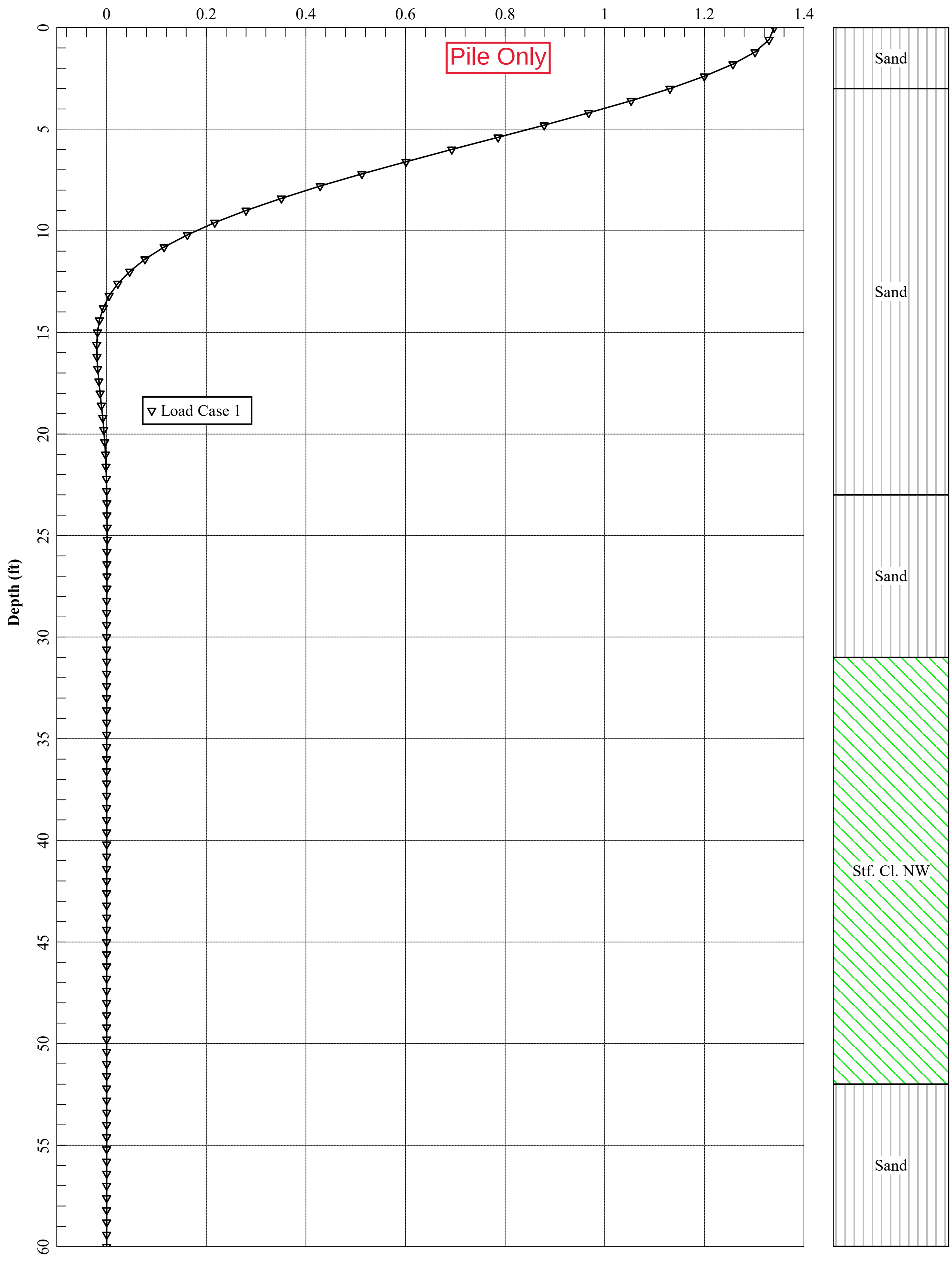


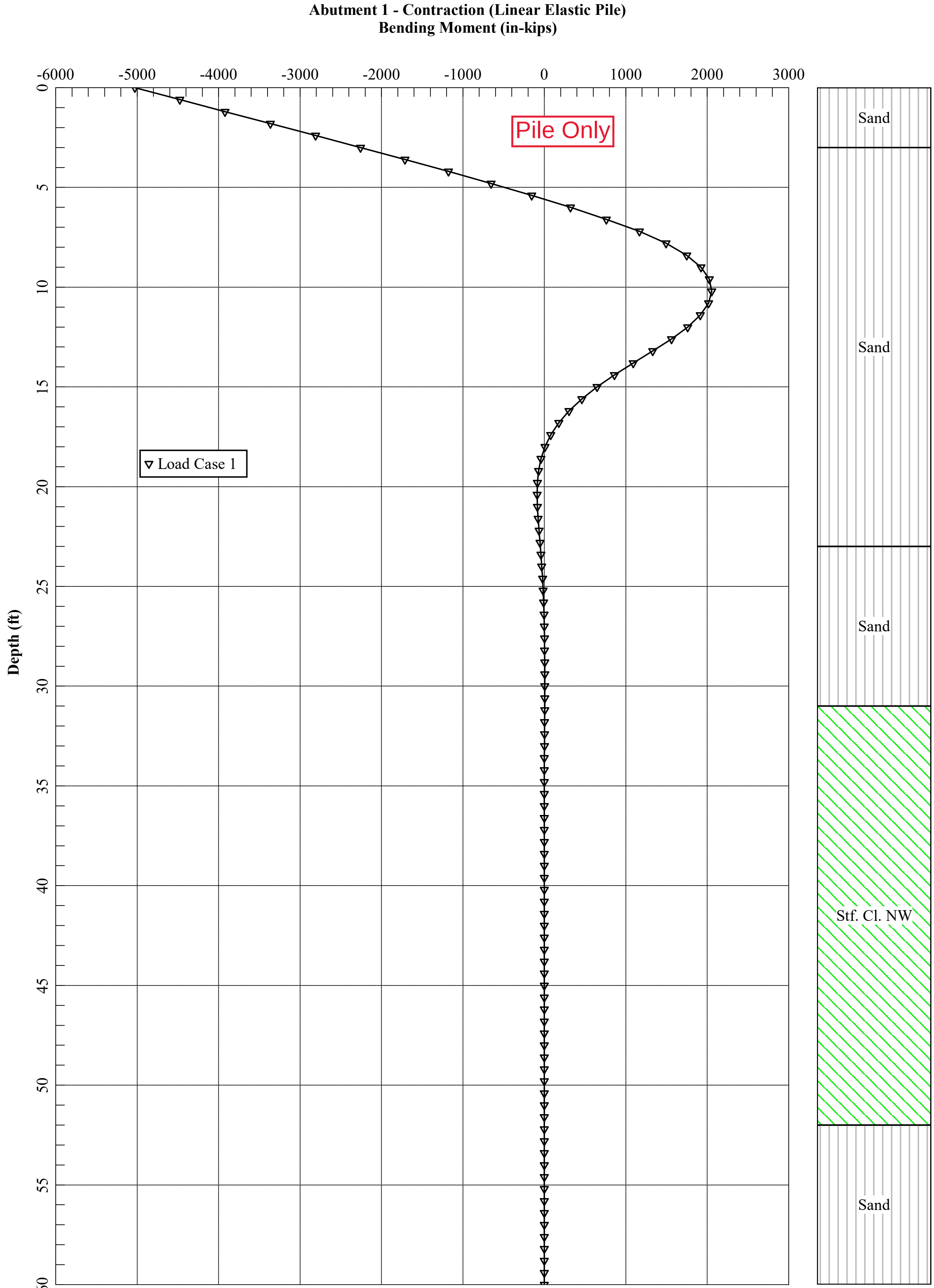
Abutment 1 - Expansion (Linear Elastic Pile)
Bending Moment (in-kips)

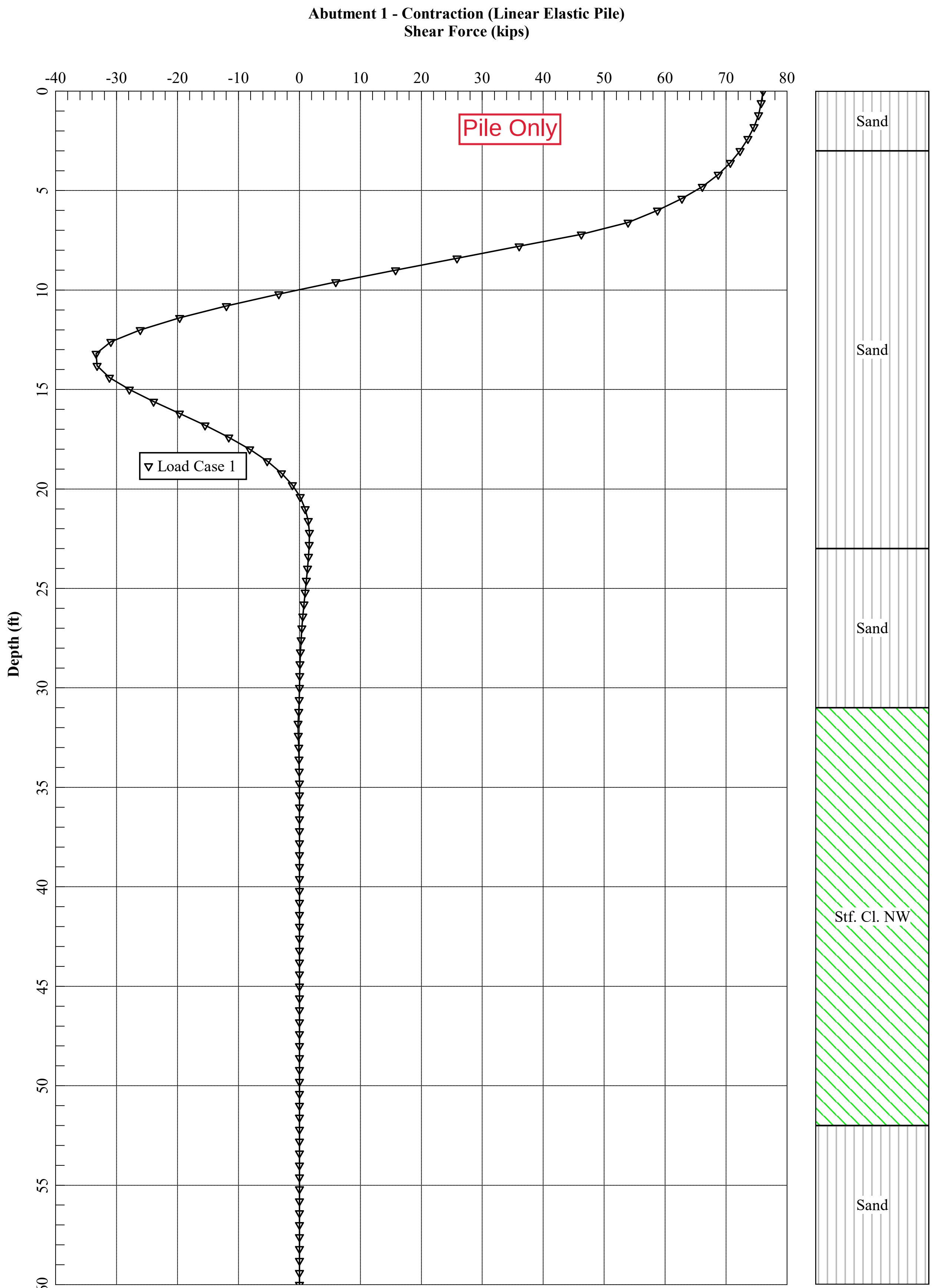


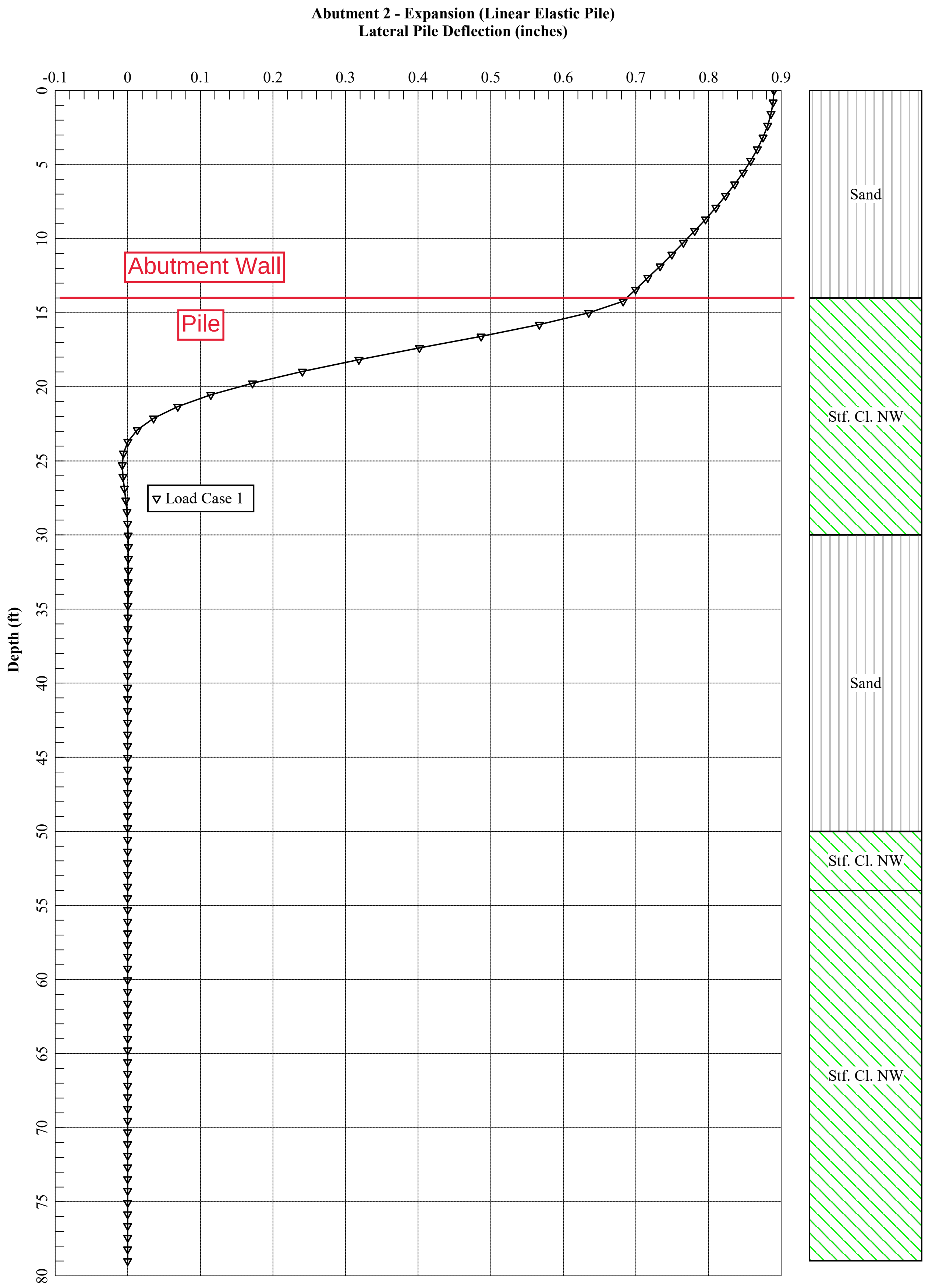

Abutment 1 - Expansion (Linear Elastic Pile)
Shear Force (kips)
-100
0
100
200
300
400
500
600
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
Abutment Wall
Pile
Load Case 1
Sand
Sand
Sand
Sand
Stf. Cl. NW
Sand

Abutment 1 - Contraction (Linear Elastic Pile)
Lateral Pile Deflection (inches)

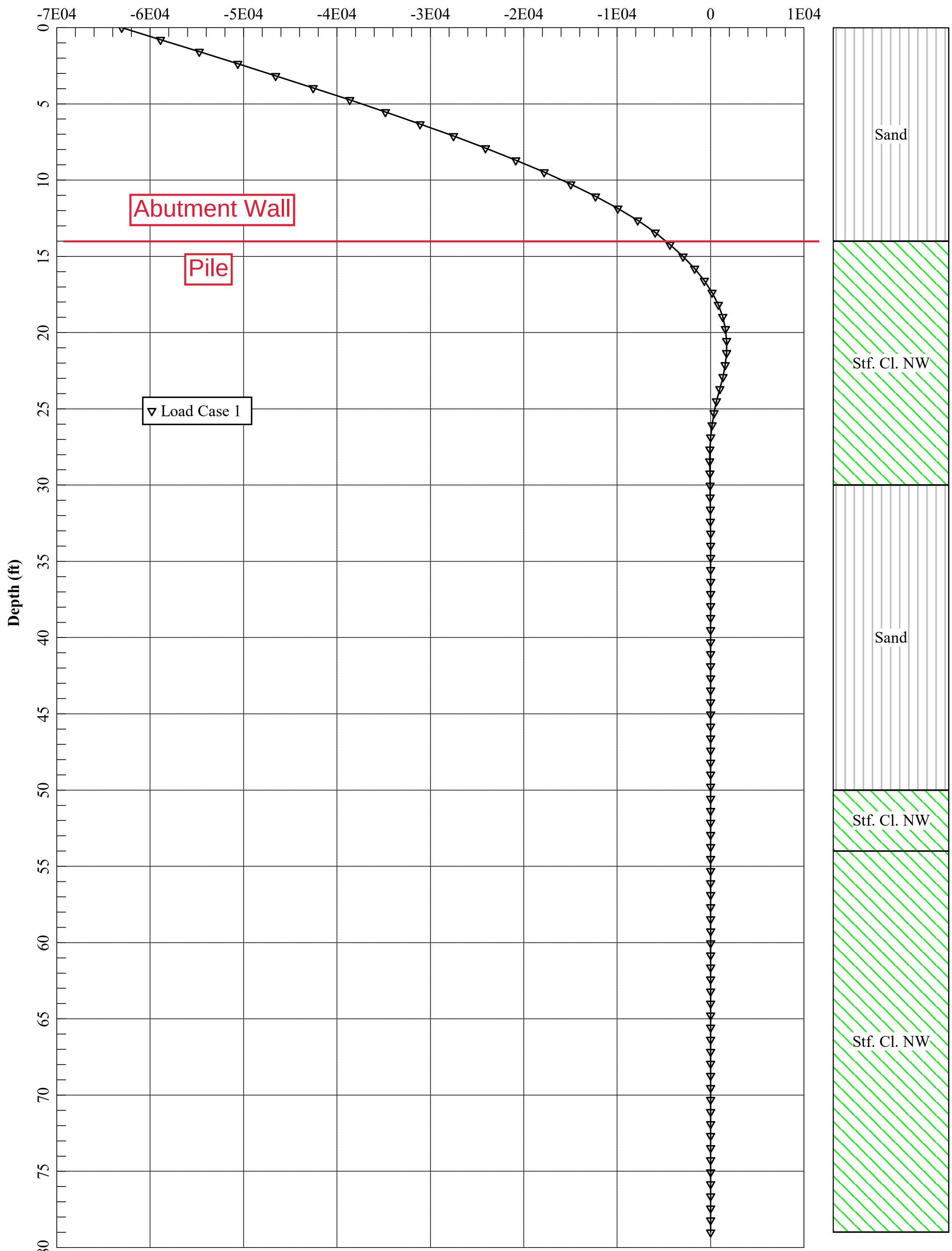



Abutment 1 - Contraction (Linear Elastic Pile)
Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Pile Only
Load Case 1
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Sand
Sand
Sand
Stf. Cl. NW
Sand

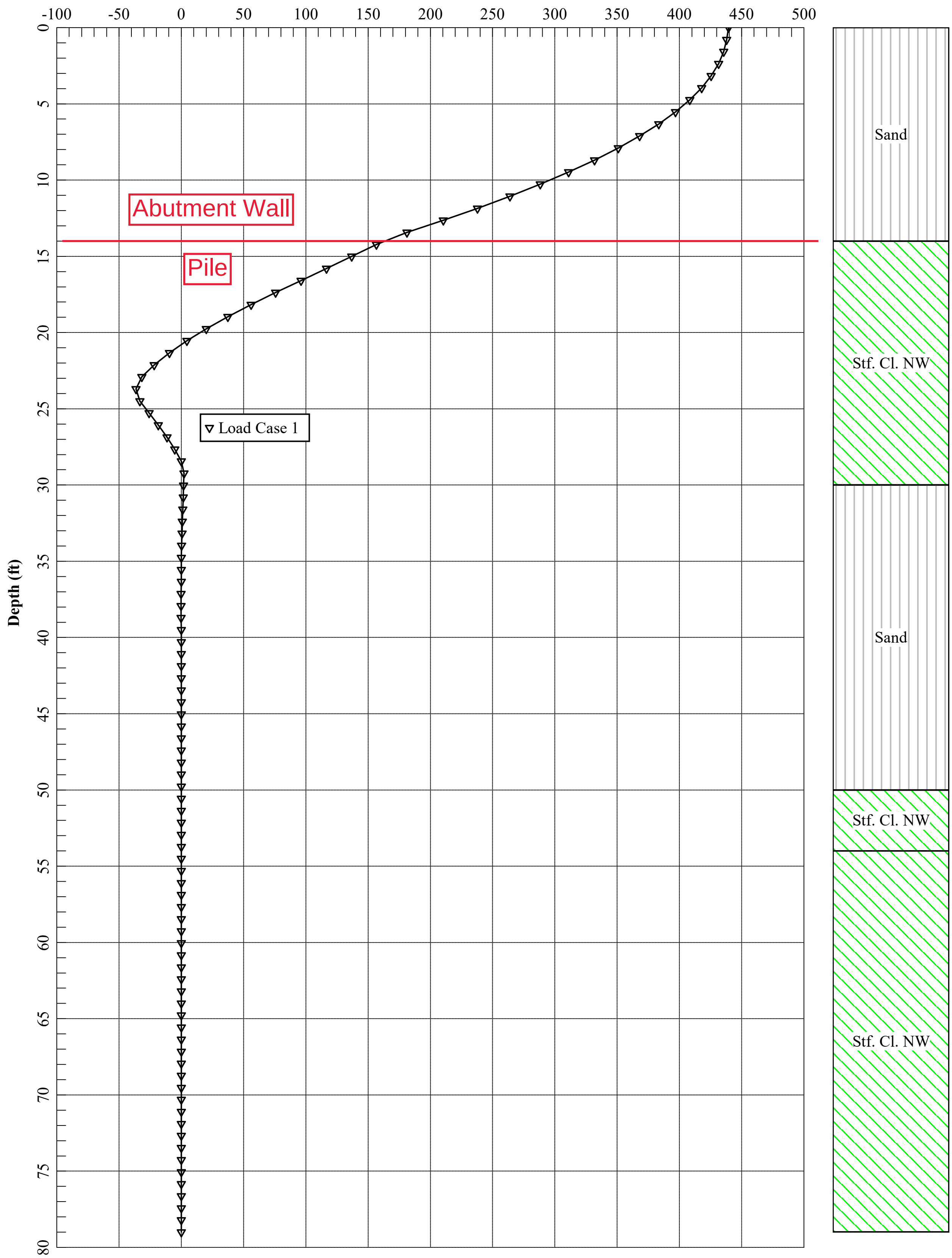

Abutment 1 - Contraction (Linear Elastic Pile)
Shear Force (kips)
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Pile Only
Load Case 1
Sand
Sand
Sand
Stf. Cl. NW
Sand

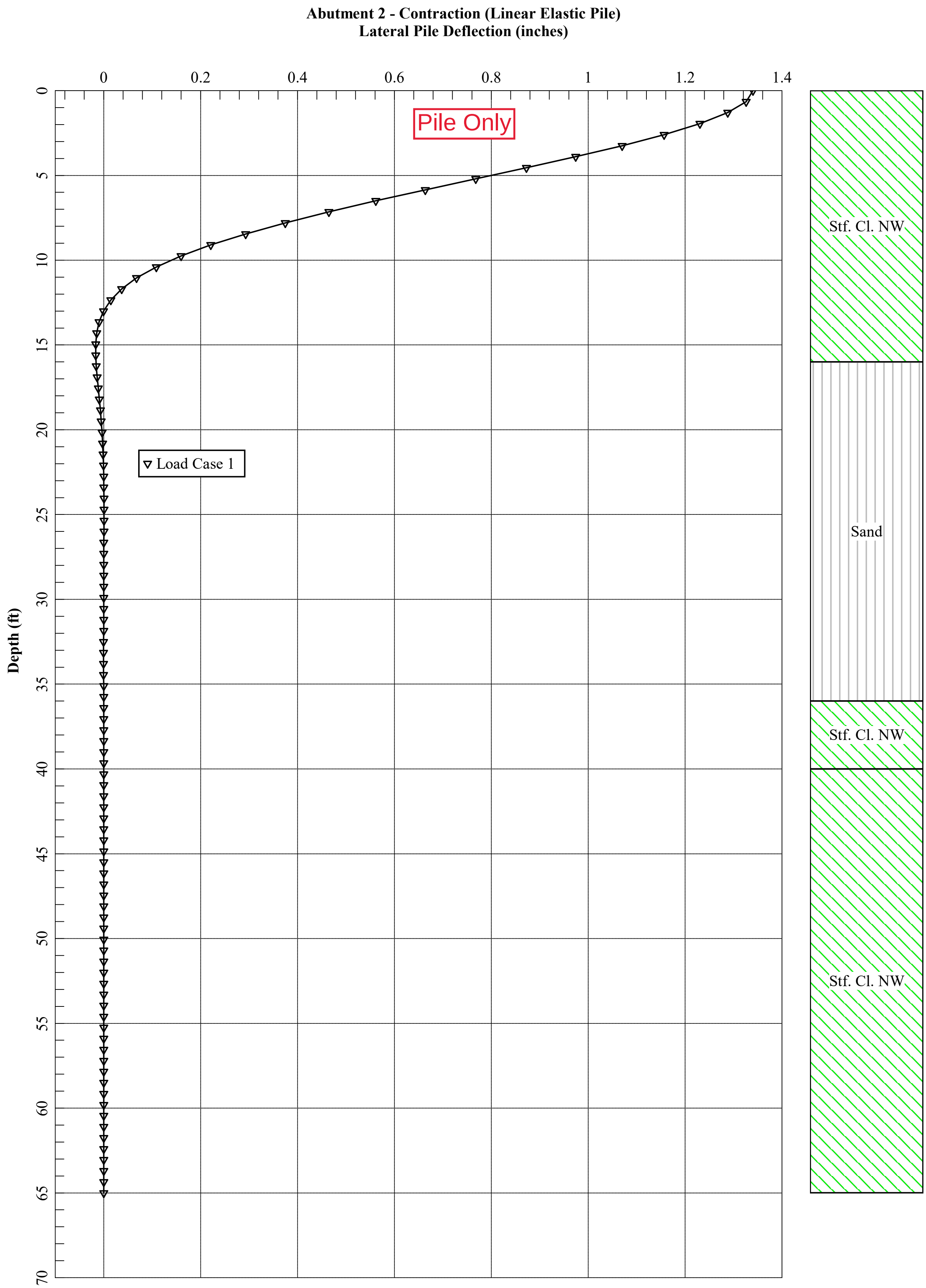

Abutment 2 - Expansion (Linear Elastic Pile)
Lateral Pile Deflection (inches)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
Abutment Wall
Pile
Load Case 1
Sand
Stf. Cl. NW
Sand
Stf. Cl. NW
Stf. Cl. NW

Abutment 2 - Expansion (Linear Elastic Pile)
Bending Moment (in-kips)

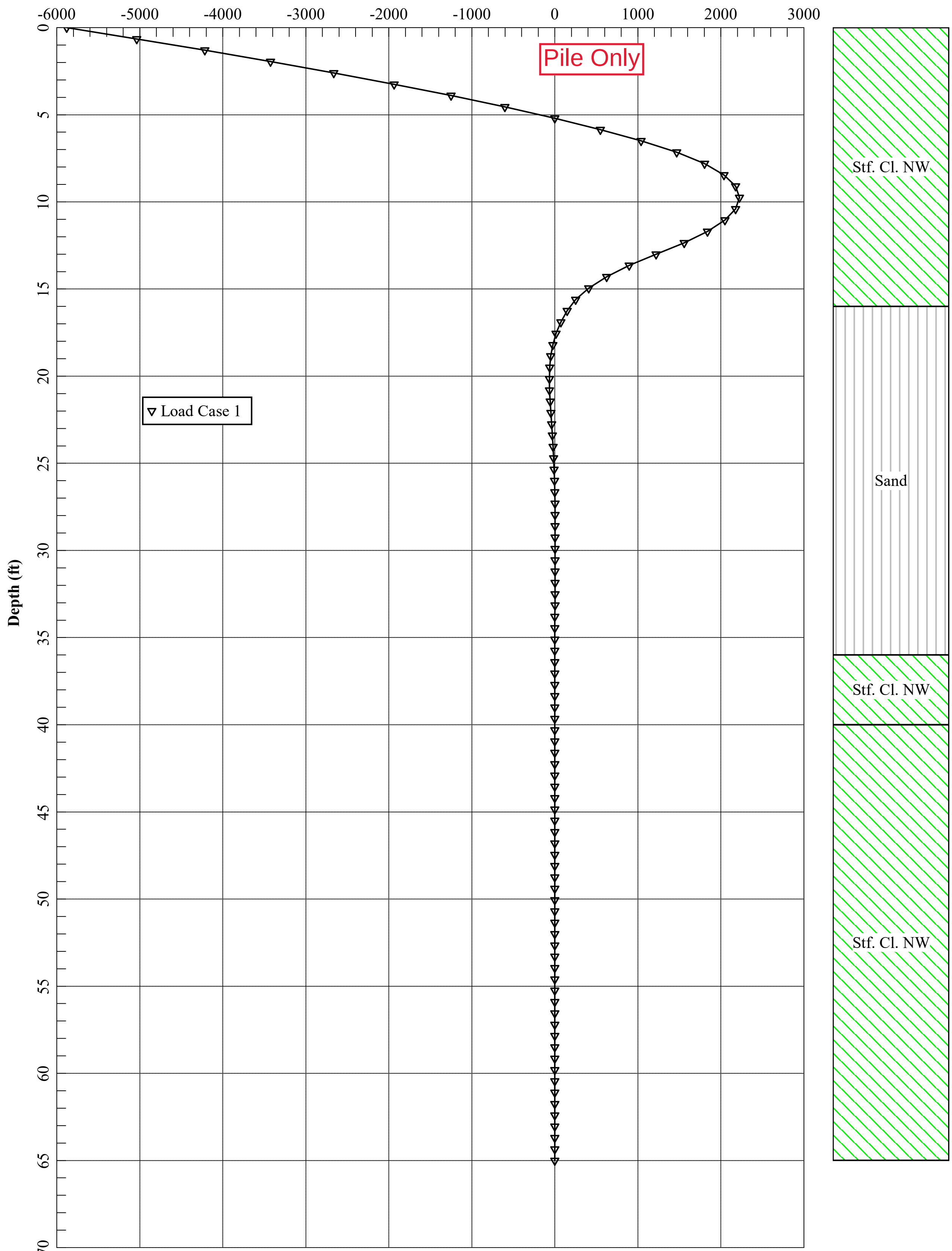


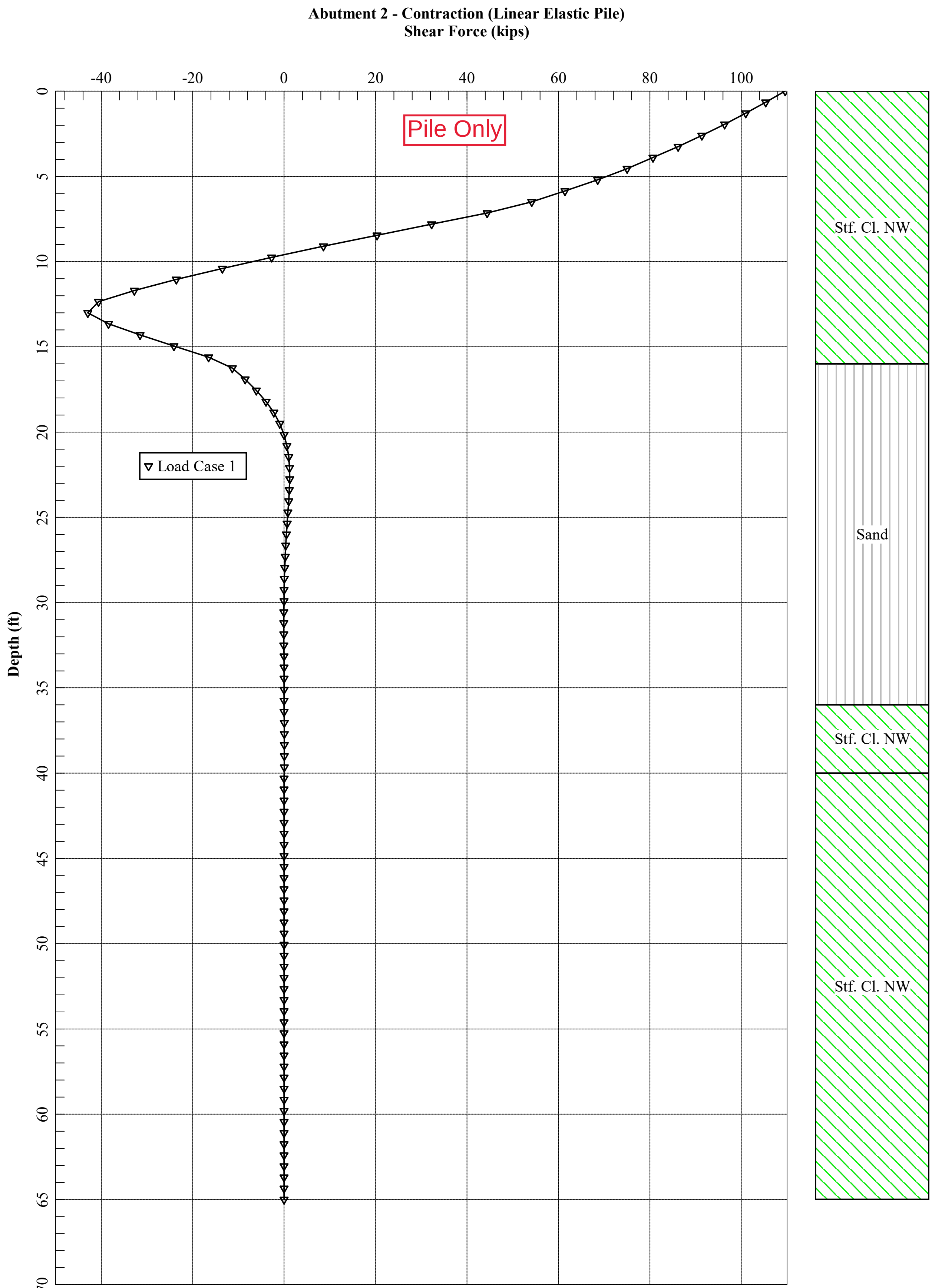
Abutment 2 - Expansion (Linear Elastic Pile)
Shear Force (kips)




Abutment 2 - Contraction (Linear Elastic Pile)
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Pile Only
Load Case 1
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Stf. Cl. NW
Sand
Stf. Cl. NW
Stf. Cl. NW

Abutment 2 - Contraction (Linear Elastic Pile)
Bending Moment (in-kips)





Abutment 2 - Contraction (Linear Elastic Pile)
Shear Force (kips)
-40
-20
0
20
40
60
80
100
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Pile Only
Load Case 1
Stf. Cl. NW
Sand
Stf. Cl. NW
Stf. Cl. NW

LPile Analysis Using Plastic Moments

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	5 of 6
Client:	Maine Department of Transportation	Date:	02APR2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	JLL
Subject:	Lateral Analysis of Abutment H-Piles Under Expansion & Contraction Displ. & Plastic Moment	Checked by:	NAS

PROBLEM STATEMENT & OBJECTIVE

Determine the bending moment and shear in the Abutment 1 and 2 H-Pile foundations induced by the anticipated deck expansion and contraction lateral displacements together with plastic moments applied to the top of the pile.

EXECUTIVE SUMMARY

With plastic moments applied and the lateral displacements prescribed, the maximum moment within the pile is equal to the plastic moment. The estimated combined stress (axial + bending) is in excess of fy.

REFERENCES

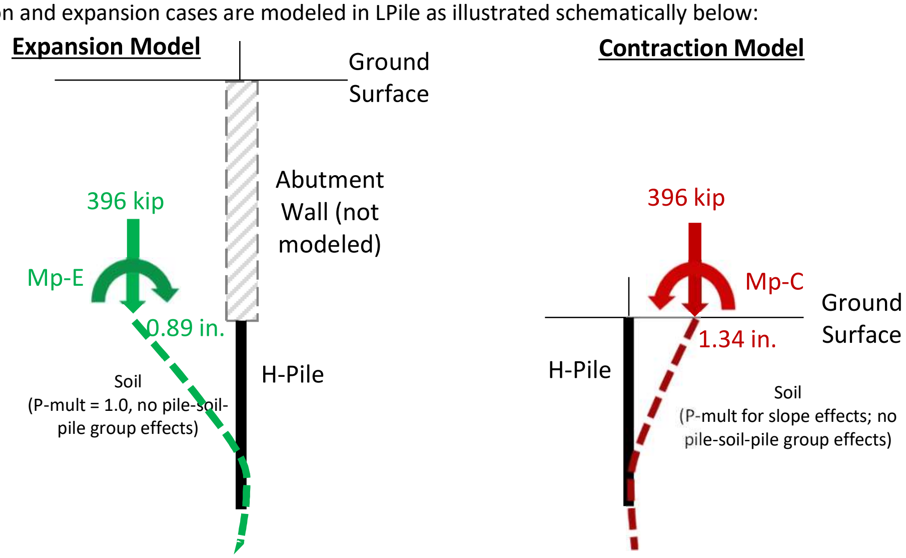
1. AASHTO LRFD Bridge Design Specifications, 2017 Edition.
2. FHWA NHI-06-088, NHI Course No. 132012, Soils and Foundations, Reference Manual - Volume 1, December 2006.
3. Ensoft, Inc., LPile 2018, Technical Manual.
4. Effects of Soil Slope on Lateral Capacity of Piles in Cohesive and Cohesionless Soils, Nimityongskul et al. (2012), Caltrans Report No. CA-11-0932. (P-multipliers applied to free-field to model effect of sloped ground surface).

AVAILABLE INFORMATION

1. Recent boring logs BB-BWS-202 (Abutment 1) and BB-BWS-206 (Abutment 2) and related lab testing results.
2. Site & Subsurface Exploration Location Plan & Interpretive Subsurface Profile for I-395 Route 9 Connection I-95 Brewer-Eddington Penobscot County, Sheets 2 and 3 from Bridge Plans, dated 19 February 2020.
3. Abutment No. 1, Sheet 51 of Bridge Plans, dated 19 February 2020.
4. Pile type, orientation, axial load, and horizontal contraction and expansion translations from MaineDOT, by email, dated 11 February 2020.
5. Additional information re: model fixity, girder elevations, and preferred output from MaineDOT, by email, dated 14 February 2020.
6. Plastic moment values and application in LPile from MaineDOT, by email dated 24 March 2020.

ASSUMPTIONS

1. Elevation Units and Datum: feet, North American Vertical Datum of 1988 (NAVD88).
2. Single pile model using LPile. Piles are driven to bedrock.
3. Bridge Plans show 3% grade (1.7 deg. slope) but assumed flat ground in our LPile analyses for simplicity.
4. Pile-soil-pile lateral effects (i.e., group effects) are negligible because piles are widely spaced (>6 times pile dimension).
5. P-multipliers are used to consider sloping ground effect for the contraction case (from reference 4).
6. Piles are HP14x117 oriented such that deck contraction and expansion cause bending about the pile weak axis.
7. No corrosion reduction on the pile section.
8. Piles are linear-elastic, non-yielding.
9. Displacements, axial load, and plastic moment applied to top of pile concurrently.
10. Only the pile elements are modeled in Lpile (i.e., abutment walls not modeled).
11. Contraction and expansion cases are modeled in LPile as illustrated schematically below:



ATTACHMENTS

SUMMARY OF LPILE RESULTS

Abutment	Case	Pile Top Lat. Displ. (in.)	Applied Moment at Top (lbs*in.)	Pile Axial Load (kip)	Pile Max. Shear (kip)	Pile Max. Moment (kip*ft)	Estimated Combined Stress (ksi)
1	Expansion	0.89	-3426000	396	238	286	66
	Contraction	1.34	-3384000	396	59	282	65
2	Expansion	0.89	-3426000	396	156	286	66
	Contraction	1.34	-3384000	396	84	282	65

With plastic moments applied and the lateral displacements prescribed, the maximum moment within the pile is equal to the plastic moment. The estimated combined stress (axial + bending) is in excess of fy.

Excerpts from References

14.4 DESIGN RECOMMENDATIONS FOR SERIES-II (COHESIONLESS SOILS)

14.4.1 SIMPLIFIED DESIGN PROCEDURE

Figure 14-2 presents a generalized soil slope profile created for cohesionless soils to obtain recommended p-multipliers (or reduction factors). These recommended p-multipliers are created to account for larger pile displacements (more conservative, higher reduction in load) and do not need to be modified for increasing pile displacements during design. These reduction factors are based on the distance from the slope crest and depth below the ground surface measured in pile diameters, D.

Recommended simplified design procedure to account for soil slope in cohesionless soils:

- Determine the designed pile size (diameter) being installed within proximity of the slope
- Identify cohesionless soil properties and corresponding free-field (level ground) *p-y* curves for the site
- Define the location and distance (in number of pile diameters) the pile will be located from the slope crest
- Using **Figure 14-2**, determine where the design pile will be located on the generalized slope shown in this figure
- Apply the corresponding *p*-multipliers from the figure to the free-field p-y curves to account for the presence of the slope
    - For piles located on the slope, apply a reduction factor of 0.3 for the top four pile diameters and 0.4 for the following six pile diameters
    - For piles located from the slope crest to four pile diameters back from the crest, apply a reduction factor of 0.5 for the top 4 pile diameters and 0.6 for the following 6 pile diameters
    - No reduction factor (*p*-multiplier of 1.0) is required below 10D
    - For piles located outside of this range no reduction factors are required

These recommendations are conservative due to the simplifications of this design procedure but present an efficient way to account for the reduction in lateral capacity due to proximity of a slope in cohesionless soils.

14.4.2 GENERALIZED CONCLUSIONS FOR SERIES-II

Based on the results of full-scale experiments and lateral load analyses, the main findings of this research study on the effect of soil slope on lateral capacity of piles in cohesionless soils are provided as the following:

- The effects of slope on lateral pile capacity are insignificant at displacements of less than 2.0 inches for piles located 2D and further from the crest.
- For pile located at 4D or greater from the slope crest, the effect of slope is insignificant for the analyzed ranges of soil displacements on p-y curves.
- Analytical, small scale, and computer models typically overestimate the effects of slope on lateral pile capacities and conservatively predict the ultimate resistance and initial soil stiffness.
- For all testing cases in the cohesionless material the lateral capacity was significantly higher than the 5 kips noted in the Caltrans BDS for 12-inch steel pipe piles for maximum allowable pile deflection of ¼-inch under Service Limit State Load according to Caltrans BDS Article 4.5.6.5.1.

The limitations of these conclusions and recommendations should always be considered when extrapolating for other design parameters that differ from the testing conditions in this study including slope angle, pile diameter, loading type, and pile type.

14.4.3 OTHER OBSERVATIONS FROM SERIES-II TESTING

The following sections present observations made during full-scale lateral load testing:

- Piles installed on a slope should not be considered to have similar lateral capacities as piles installed on the slope crest. In this study, the capacities and reduction factors were significantly different between these two cases.

- Ultimate capacity for load-displacement curves is reduced for piles closer than 8D
- The effects of reduced overburden pressure due to presence of soil slope has a larger impact on the reduction of lateral capacity in cohesionless soils
- The shear failure angle, Ω, of the passive wedge ranged between 24° and 39°. This angle increased with greater distances from the slope crest. A recommendation of 70% of ϕ is proposed for the shear failure angle in dense cohesionless material.
- LPILE 6.0 underestimates the initial stiffness and the lateral pile capacity in level ground conditions. The full-scale test results had an ultimate resistance of 20% more than predicted by LPILE 6.0. The lateral capacity for the 0D pile was relatively close and only underestimated the lateral capacity by about 10%.
- The predicted baseline API (1987) and Reese et al. (1974) p-y curves over predict the initial soil stiffness at displacements of less than 0.2 inches
- API (1987) and Reese et al. (1974) models significantly under predicted the back-calculated ultimate soil reaction at displacements greater than 0.25 inches.
- Mezazigh and Levacher (1998) reduction coefficients are considered conservative when applied to the baseline p-y curve and then compared to the near slope results.

14.5 COMPARISON BETWEEN COHESIVE & COHESIONLESS RESULTS

During this study seven non-battered piles were tested in each soil type, cohesive and cohesionless. The cohesionless load-displacement curves had higher ultimate capacities for all load tests (baseline through 0D) when compared to the cohesive results. The initial stiffness at lower displacements was also greater for the cohesionless piles. These curves show a larger effect from slope (when compared to baseline) on the 2D and 4D piles capacities in cohesive soils. A greater effect on capacities was seen for the 0D and -4D piles in the cohesionless soils.

The shapes and the effects of slope on the *p-y* curves differed between Series-I and Series-II. For displacements less than 0.25 inches, the slope a small to insignificant effect on the lateral pile response in cohesive soils. The effect also increased with increased soil displacement (i.e. a larger reduction in capacity with displacement) for cohesive soils. This was not the case for Series-II, as the lateral capacities were affected at all soil displacements. When compared to respective baseline tests, the results from the cohesionless series had a larger reduction in lateral capacities. The recommended *p*-multipliers for the cohesionless ranged from 0.3 to 0.6, while the cohesive recommended *p*-multipliers ranged from 0.5 to 0.7. The presence of a slope, and consequently a reduction in overburden pressure for the soil resisting lateral movement has a greater impact on cohesionless soils. This is, most likely, due to the absence of cohesion, wherein Series-I the presence of the test slope has less of an effect on the resistance due to apparent cohesion between soil grains.

14.6 BATTERED PILE TEST CONCLUSIONS (SERIES-II)

Pile P-4 with a -14° batter angle had the highest stiffness and capacity of all piles tested in this study. Pile P-3 (+14° positive batter) had the lowest capacity of the tested battered piles. The load displacement results from pile P-5 (+26°) do not fit the predicted trend. The LPILE predicted load-displacement curves from pile P-3 (+14°) and P-4 (-14°) follow the trend of the full-scale results, but LPILE is conservative in estimating the initial stiffness and ultimate lateral resistances. The full-scale results from Pile P-5 (+26°) had a significantly greater stiffness and capacity than the LPILE prediction, where it was predicted to have the lowest overall load. An analysis of the load displacement data from the +26° battered pile showed that the testing equipment was likely near its limitations to laterally load a pile with this high batter angle. The unexpected stiffness and load from the full-scale test are likely due to unintended axial loading. Overall, LPILE is a conservative method to predicted lateral capacity of battered piles in cohesionless soils. The load ratio model used in LPILE battered pile predictions compares well with the ratios obtained for full-scale lateral load tests.

14.7 RECOMMENDATIONS FOR FUTURE RESEARCH

- Soil slope effects for different pile diameter can be considered in a controlled environment, such as using physical model testing. The soil properties and slope geometry can therefore be controlled. The stiffness of the pile should remain constant for different pile diameters in order to achieve the same level of soil displacement for a proper comparison of *p-y* curves. The constant pile stiffness with varying pile diameter can be achieved by selecting different pile thickness or using different materials.

- Three-dimensional finite element modeling, which can model construction sequences and some aspects observed during the testing, such as gapping and cracking, as well as accounting for softening due to soil dilatency should be conducted to understand if these aspects have significant contribution to the effects of slope on the pile response. Results from full-scale lateral loading tests can be used to calibrate the 3-D model, and therefore the analysis for slope effects can be reasonably extrapolated to use for different slope geometry, soil type, pile type and different distance between pile-slope crest.

- The effects of slope for pile groups may be different than that for a single pile and should be investigated.

- Though *p-y* curves have been developed based on the results of the full-scale lateral pile loading tests for a case of long, flexible piles, they have been used in design to predict the lateral response for rigid pile as well. However, the implementation of *p-y* curves for short, rigid piles has not been verified with the results from full-scale tests. Research on the effects of pile length on the pile response using full-scale testing should be conducted to verify if they existing *p-y* curves are appropriate for the case of rigid pile.

- The effects of loading type such as cyclic loading, sustained loading and dynamic loading should be investigated. In addition, the effects of axial loads on the lateral pile response also require further study. The effects of varying slope angle on should also be examined.

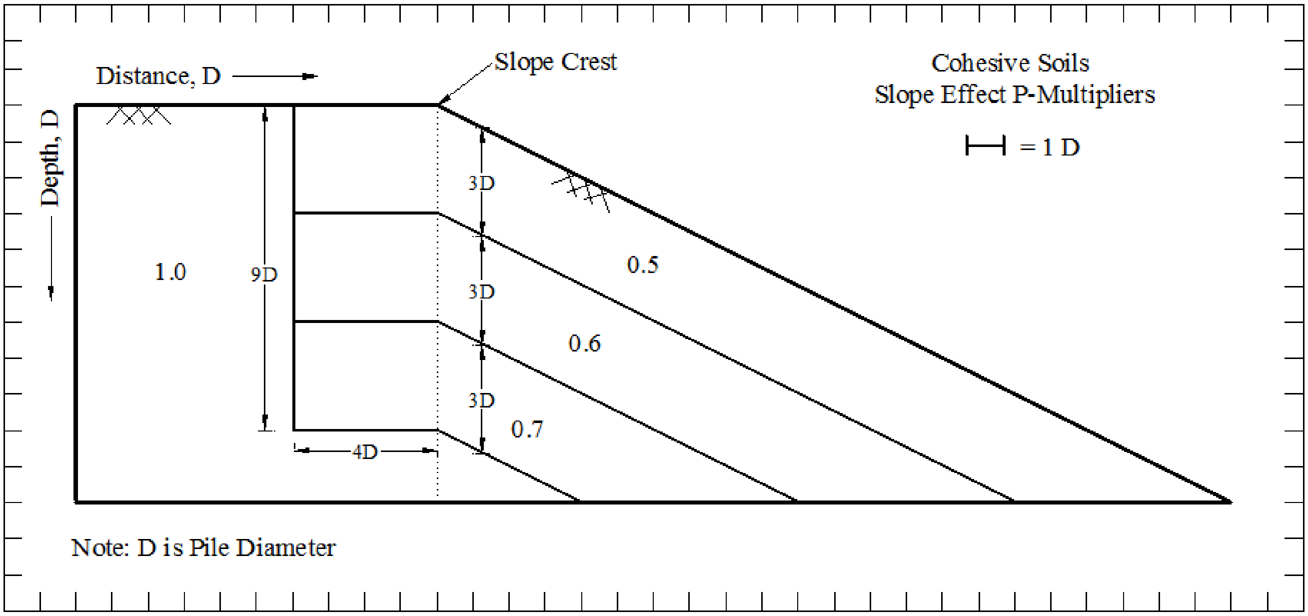


Figure 14-1 Recommended p-Multipliers for a Generalized Cohesive Slope

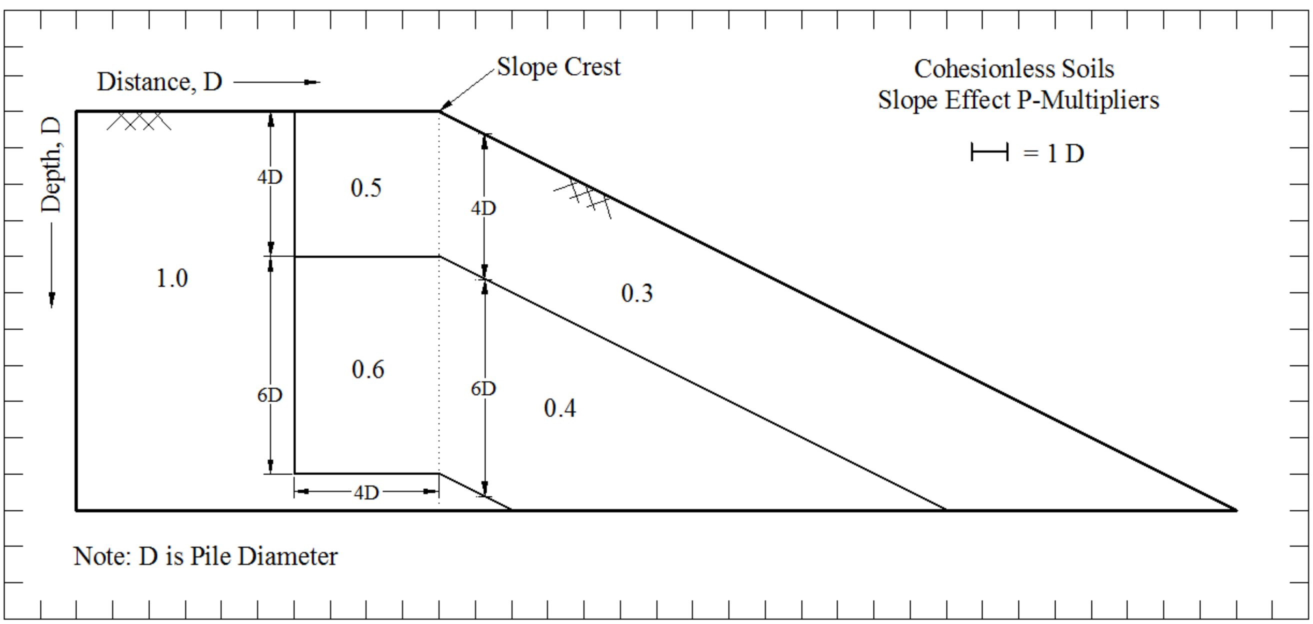


Figure 14-2 Recommended p-Multipliers for a Generalized Cohesionless Slope

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	06APR2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	JLL
Subject:	Lateral Analysis of Pier H-Piles	Checked by:	NAS

PROBLEM STATEMENT & OBJECTIVE

Determine the bending moment and shear in the Pier H-Pile foundations under the anticipated loading

EXECUTIVE SUMMARY

Under the loads provided, the pile lateral deflection under the SER loads is 0.1 in. Under the STR loads, the max. pile moment about the strong axis is 61.5 kip*ft and the max. pile moment about the weak axis is 42.3 kip*ft.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2017 Edition.
2. FHWA NHI-06-088, NHI Course No. 132012, Soils and Foundations, Reference Manual - Volume 1, December 2006.
3. Ensoft, Inc., LPile 2018, Technical Manual.

AVAILABLE INFORMATION

1. Recent boring log BB-BWS-203 and related lab testing results.
2. Site & Subsurface Exploration Location Plan & Interpretive Subsurface Profile for I-395 Route 9 Connection I-95 Brewer-Eddington Penobscot County, Sheets 2 and 3 from Bridge Plans, dated 19 February 2020.
3. Pier pile plan and loading from MaineDOT, by email, dated 02 April 2020.

ASSUMPTIONS

1. Elevation Units and Datum: feet, North American Vertical Datum of 1988 (NAVD88).
2. Single pile model using LPile. Piles are driven to bedrock.
3. Pile-soil-pile lateral effects (i.e., group effects) are negligible because piles are widely spaced (>6 times pile dimension).
4. Piles are HP14x117. No corrosion reduction on the pile section.
5. Piles are linear-elastic, non-yielding (i.e., no ceiling on moment generated within the pile).
6. Piles are "fixed" to the pile cap so top of model element restrained against rotation but allowed to translate.
7. Bottom of cap/top of pile at El. +111. Tip of pile at top of rock at approx. El. +67.
8. Model ground surface assumed at El. +111. Groundwater assumed at El. +107 (average between the levels assumed for the two abutments).
9. Lateral pile-soil-pile interaction (i.e., group effects) are based on an approximate center-to-center spacing of 5 ft (i.e., the spacing between the two rows). P-multipliers of 0.8 and 0.62 were used for loads causing bending about the strong and weak axis, respectively.
10. Pile section properties are as follows:

Section Type	Width (in.)	Depth (in.)	A (in.^2)	Ixx=Istrong (in.^4)	Iyy=Iweak (in.^4)	E (ksi)
HP14x117	14	14	34.4	1220	443	29000

11. Load cases are as follows:

    Pile Axial Load = 326 kips SER; 411 kips STR

    Lateral Loads = 13.8 kips (bending about strong axis); 10.4 kips (bending about weak axis).

12. Lateral soil model properties are as follows:

Layer	LPile Model	Cohesive/ Cohesionless	Top El. (ft)	Depth from top of Pile to Top of Layer (ft)	Total Unit Wt., γ_t (pcf)(note 3)	S_u or ϕ (psf, deg)	k or ε_{50} (pci, %)
1	Sand (Reese, et. al)	Cohesionless	111	0	125	32	225
2	Stiff Clay (w/out Free Water)	Cohesive	102	9	120	2000	0.7
3	Stiff Clay (w/out Free Water)	Cohesive	96	15	120	4000	0.5
4	Sand (Reese, et. al)	Cohesionless	75	36	130	40	125
5	-	Bedrock	67	44	-	-	-

Notes for the LPile Soil Parameters:

1. S_u = undrained shear strength, ϕ = soil internal friction angle, k = lateral subgrade modulus, ε_{50} = percent strain at 50% of shear strength
2. ϕ and k apply to Cohesionless type materials, S_u and ε_{50} apply to cohesive type materials.
3. Buoyant unit weights are used in LPile to calculate p-y curves for layers below the water table.

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	2 of 2
Client:	Maine Department of Transportation	Date:	06APR2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.00	Computed by:	JLL
Subject:	Lateral Analysis of Pier H-Piles	Checked by:	NAS

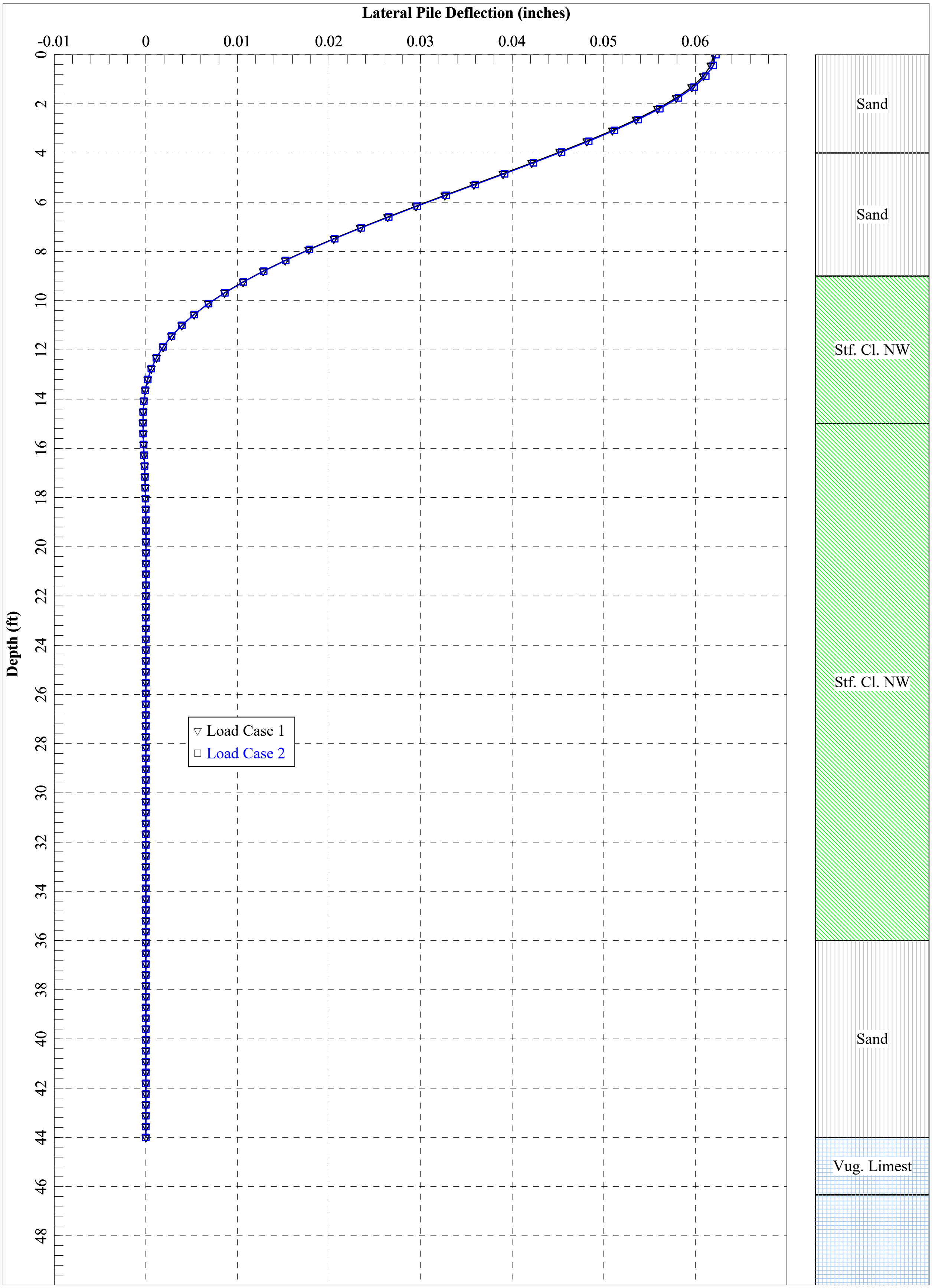
SUMMARY OF LPILE RESULTS

Case	Limit State	Bending Axis	Pile Head Condition	Axial Load (kip)	Lat. Load (kip)	Max. Mom. (kip*ft)	Top Displ. (in.)	Max. Stress (ksi)
1	SER	Strong	Fixed	326	13.8	61.3	0.06	13.8
2	STR	Strong	Fixed	411	13.8	61.5	0.06	16.2
3	SER	Weak	Fixed	326	10.4	42.0	0.10	18.0
4	STR	Weak	Fixed	411	10.4	42.3	0.10	20.5

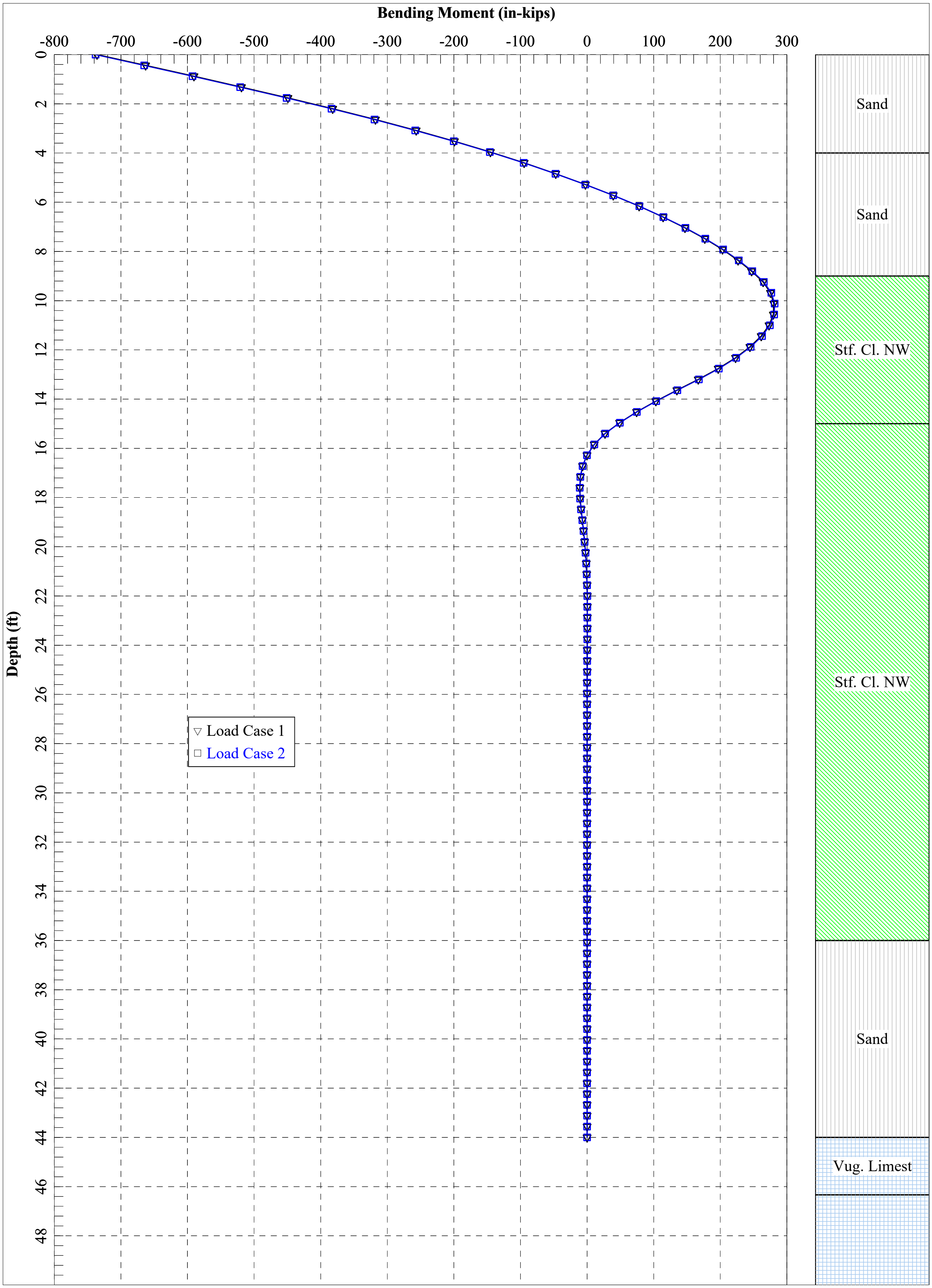
Under the loads provided, the pile lateral deflection under the SER loads is 0.1 in.
Under the STR loads, the max. pile moment about the strong axis is 61.5 kip*ft
and the max. pile moment about the weak axis is 42.3 kip*ft.

Pier LPile Results
Strong Axis Bending

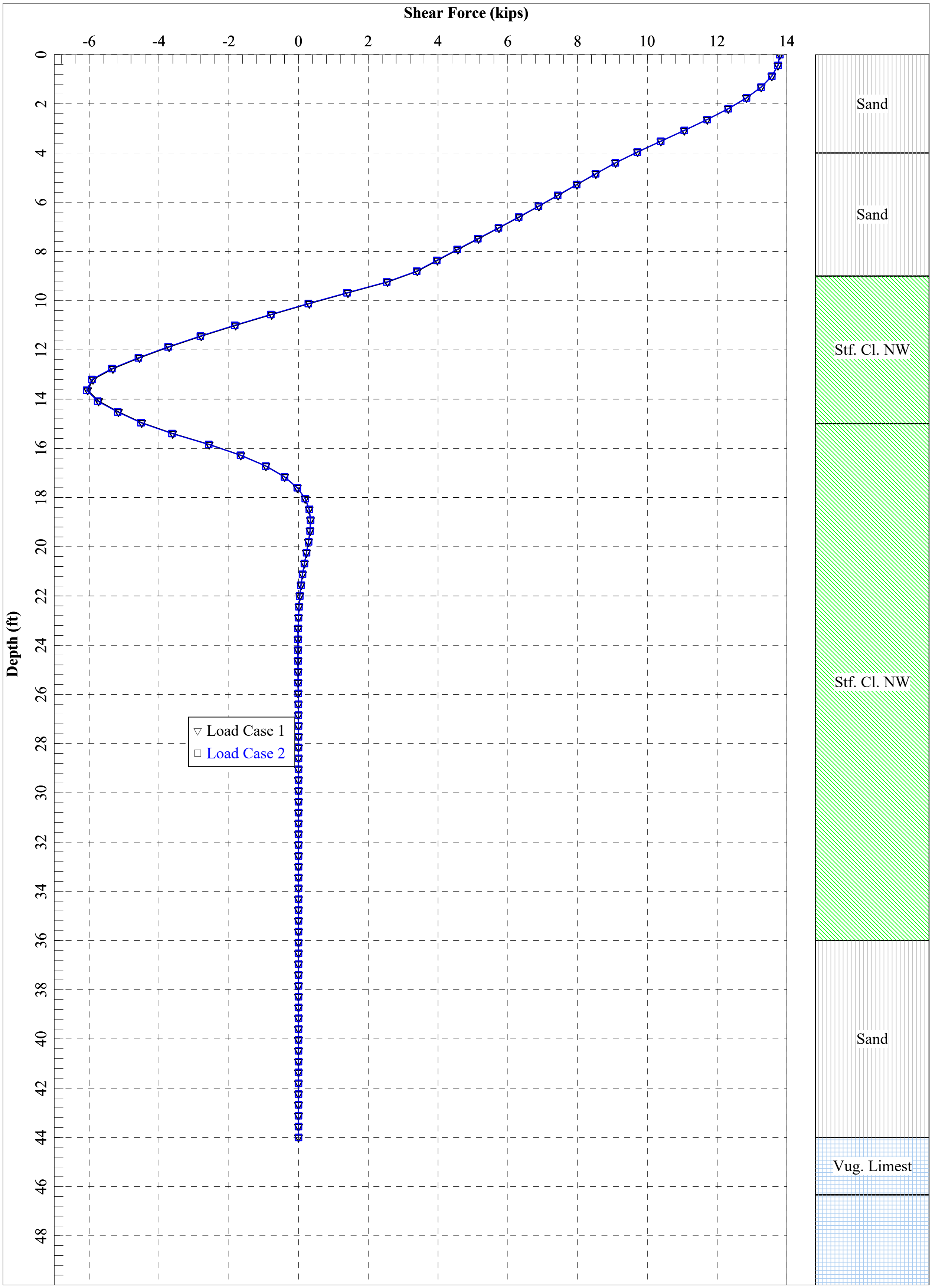
Strong Axis Bending



Strong Axis Bending

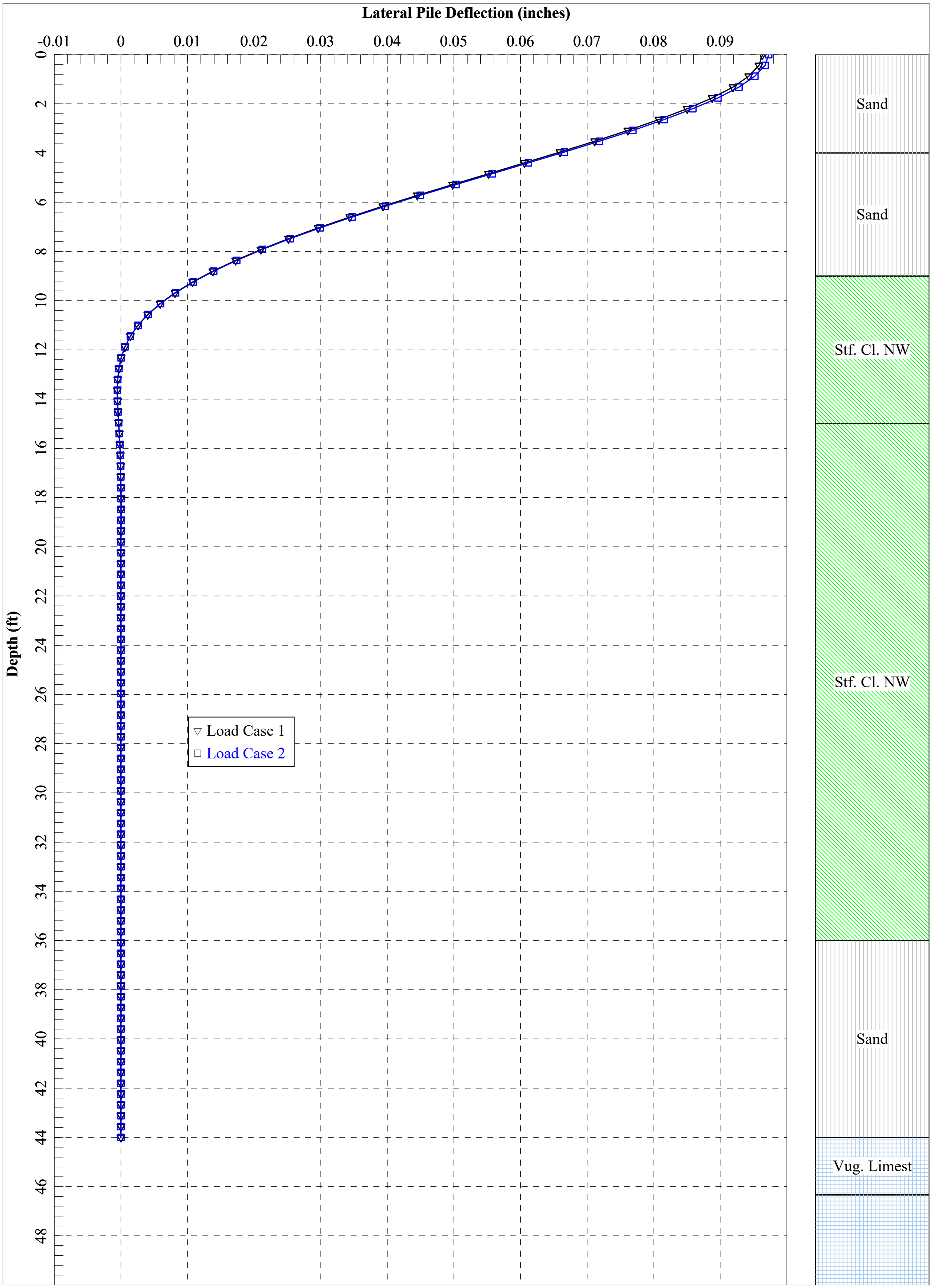


Strong Axis Bending

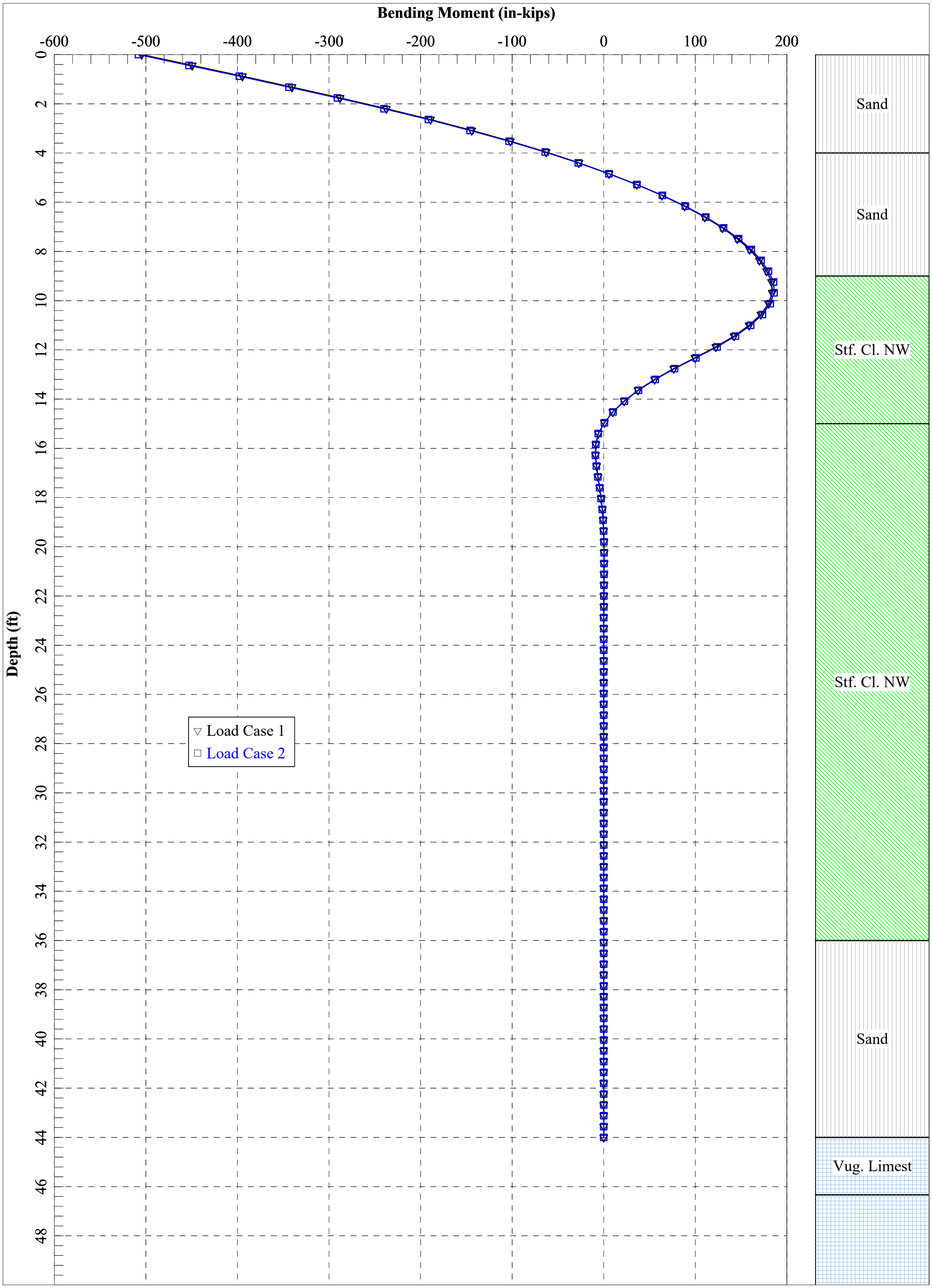


Pier LPile Results
Weak Axis Bending

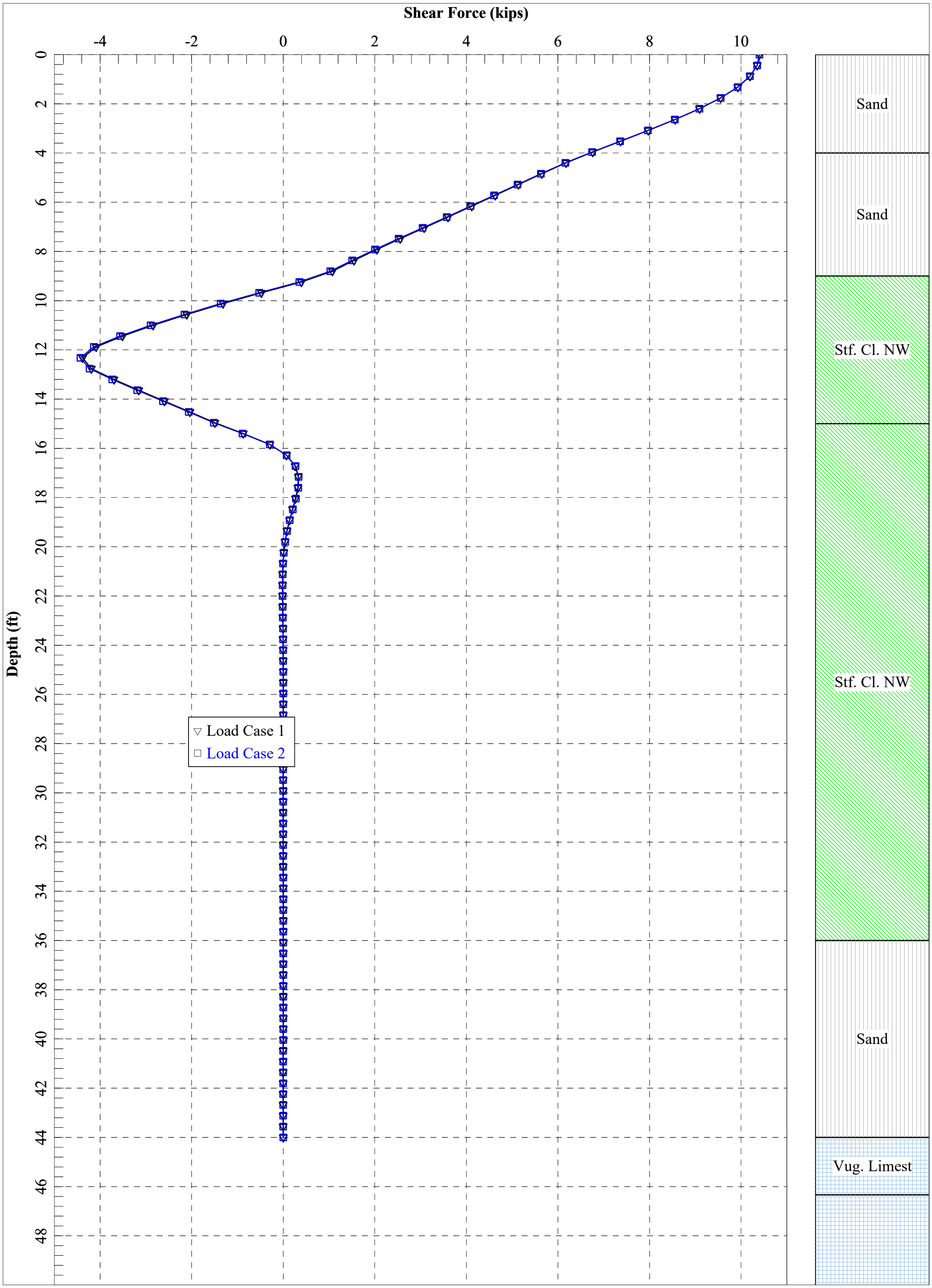
Weak Axis Bending



Weak Axis Bending



Weak Axis Bending



Frost Evaluation

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	10-Mar-2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.20	Computed by:	NAS
Subject:	Frost Susceptibility and Maximum Depth of Frost Penetration	Checked by:	BCS

OBJECTIVE:

Evaluate maximum depth of frost penetration based on soil and groundwater conditions, as well as geographic site location.

REFERENCES:

1. MaineDOT Bridge Design Guide, 2003 with interim revisions through 2017.
2. Haley & Aldrich boring logs BB-BWS-202, BB-BWS-103, and BB-BWS-206
3. 60% draft plan set prepared by MaineDOT dated 3/6/20.

EVALUTATION:

1. Gather relevant information from test borings performed near proposed bridge abutment locations:

STRUCTURE	STRUCTURE BEARING ELEVATION	TEST BORING NO./GS EL.	GROUND WATER	SAMPLE No. AND ELEVATION	AASHTO/ USCS	FINES CONTENT	MOISTURE CONTENT
ABUTMENT NO. 1	El. 131.1	BB-BWS-202 El. 143.8	El. 108.0 during drilling	Due to the elevation of the test boring, no samples are located in the vicinity of the proposed abutment bearing elevation.			
PIER	El. 111.0	BB-BWS-103 El. 107.1	El. 103.3 during drilling	1D El. 107.1 - 105.1	--/SM	--	--
				3D El. 102.1 - 100.1	A-4/CL-ML	52.4	13.1
ABUTMENT NO. 2	El. 121.5	BB-BWS-206 El. 134.6	El. 104.6 during drilling	Due to the elevation of the test boring, no samples are located in the vicinity of the proposed abutment bearing elevation.			

Note: Ground water elevations summarized above were determined in the field and may have been influenced by the drilling process. Ground water elevations may vary throughout the year due to seasonal variations and precipitation events.

2. The abutments will bear in existing and new embankment fill. Assume the embankment fill consists of granular material.

3. From MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1650 °F - days

4. Estimate range in frost penetration using MaineDOT Bridge Design Guide Table 5-1 and the design freezing index above.

5. For coarse grained soil at the abutments, from Table 5-1, frost penetration depths vary between approximately 5.1 ft (w=30%) to 7.2 ft (w=10%).

6. For fine grained soil at the pier, from Table 5-1, frost penetration depths vary between approximately 4.0 ft (w=30%) to 5.1 ft (w=10%).

Recommend 6.0 ft at the abutments and pier.

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	10-Mar-2020
Project:	Wilson Street Bridge Replacement - WIN No. 018915.20	Computed by:	NAS
Subject:	Frost Susceptibility and Maximum Depth of Frost Penetration	Checked by:	BCS

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0