APPENDIX C

SUPPORTING DOCUMENTATION

STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
HIGHWAY BUILDING
1589 MAIL SERVICE CENTER
RALEIGH, NORTH CAROLINA 27699-1589

SUBJECT: Bridge No. 108 on SR 1001 (Lizzie Mill Rd.) over I-95

PREPARED BY:	CW	TIP NO.:
DATE:	5/17	I-5786
CHECKED BY:	WPA	COUNTY: Johnston
DATE:	5/17	

END BENTS SUMMARY

END BENT 1

Pile Type:	HP 12X53 Steel Piles	
Bottom of Cap Elevation:	189.19 ft	Provided by WEI
Anticipated Pile Length:	52 ft ±	Bottom of Cap - Anticipated Pile Refusal Depth
Average Pile Length:	55 ft ±	Anticipated Pile Lengths Rounded Up to Nearest 5 ft
Max Factored Load:	80 Tons/Pile	Provided by WEI, rounded up to nearest 5 tons
Required Ultimate Resistance:	180 Tons/Pile	AASHTO Resistance Factor = 0.45
Required Driving Resistance:	160 Tons/Pile	NCDOT Driving Resistance Factor = 0.6 for WEAP analysis with limited or no PDAs plus additional driving resistance due to downdrag

END BENT 2

Pile Type:	HP 12X53 Steel Piles	
Bottom of Cap Elevation:	189.45 ft	Provided by WEI
Anticipated Pile Length:	47 ft (Lt), 52 ft (Rt) ±	Bottom of Cap - Anticipated Pile Refusal Depth
Average Pile Length:	50 ft (Lt), 55 ft (Rt) ±	Anticipated Pile Lengths Rounded Up to Nearest 5 ft
Max Factored Load:	80 Tons/Pile	Provided by WEI, rounded up to nearest 5 tons
Required Ultimate Resistance:	180 Tons/Pile	AASHTO Resistance Factor = 0.45
Required Driving Resistance:	165 Tons/Pile	NCDOT Driving Resistance Factor = 0.6 for WEAP analysis with limited or no PDAs plus additional driving resistance due to downdrag

NOTES

See Notes on Sheet 2 of the Foundation Recommendations.

COMMENTS

See Comments on Sheet 3 of the Foundation Recommendations.

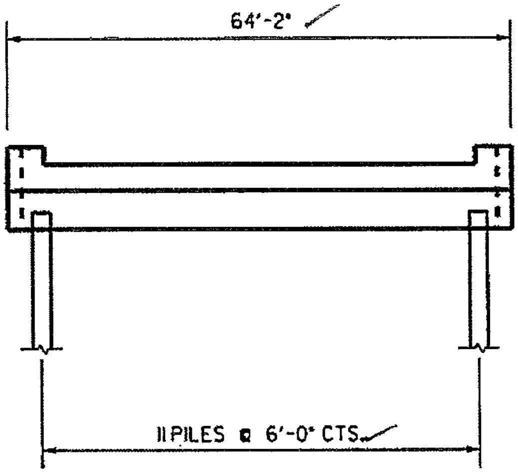
WETHERILL ENGINEERING
1223 ~~559~~ JONES FRANKLIN RD.
~~SUITE 164~~
RALEIGH NC 27606

SUBJECT FACTORED LOADS PROJECT I-5786
JOHNSTON COUNTY
PREPARED BY GMG DATE 5-2-17 STATION 17+77.11 -Y1-
CHECKED BY bcu DATE 5-2-17 STR NO 108 SHEET 1 OF 2

END BENT #1
SINGLE ROW OF PILES ✓
FIVE (5) BATTERED PILES ✓

PILE TYPE : HP 12 X 53 ✓

AVG BOC ELEV = 189.19 FT± ✓



FACTORED AXIAL PILE LOAD = 153.0 KIPS ✓

= 76.5T round up to 80T

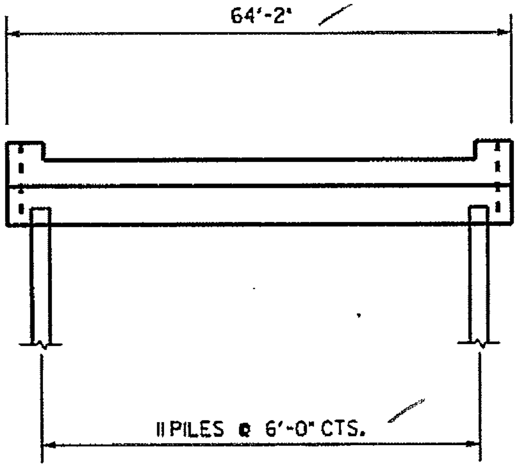
END BENT #1 SKETCH
NO SCALE

AASHTO Resistance Factor = 0.45. $\frac{80T}{0.45}$ = 177.8T, round up to 180T (360 Kips)
ultimate resistance req'd

END BENT #2
SINGLE ROW OF PILES ✓
FIVE (5) BATTERED PILES ✓

PILE TYPE : HP 12 X 53 ✓

BOC ELEV = 189.45 FT± ✓



FACTORED AXIAL PILE LOAD = 153.0 KIPS ✓

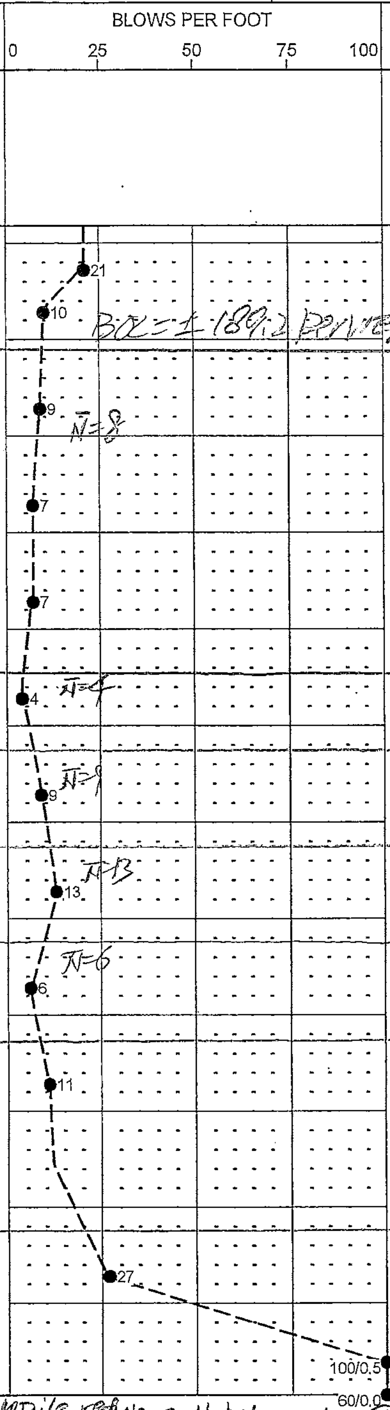
= 76.5T round up to 80T

END BENT #2 SKETCH
NO SCALE

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A
TIP I-5786
COUNTY JOHNSTON
GEOLOGIST J. Cranston
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95
GROUND WTR (ft)
BORING NO. EB1-A
STATION 16+45
OFFSET 8 ft LT
ALIGNMENT -Y1-
0 HR. NM
COLLAR ELEV. 195.9 ft
TOTAL DEPTH 60.7 ft
NORTHING 652,466
EASTING 2,224,854
24 HR. FIAD
DRILL RIG/HAMMER EFF./DATE F&R5785 CME-55 80% 02/11/2017
DRILL METHOD Mud Rotary
HAMMER TYPE Automatic
DRILLER D. Aiello
START DATE 04/10/17
COMP. DATE 04/10/17
SURFACE WATER DEPTH N/A

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	BLOW COUNT 0.5ft	BLOW COUNT 0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
194.6	1.3	8	11	10	21		M
192.4	3.5	4	4	6	10		M
187.4	8.5	1	4	5	9		M
182.4	13.5	2	2	5	7		M
177.4	18.5	2	3	4	7		M
172.4	23.5	1	2	2	4		M
167.4	28.5	3	4	5	9		M
162.4	33.5	4	5	8	13		M
157.4	38.5	1	3	3	6		W
152.4	43.5	3	5	6	11		Sat.
							Sat.
142.4	53.5	7	10	17	27		M
137.4	58.5	100/0.5			100/0.5		
135.2	60.7	60/0.0			60/0.0		



BOC = ± 189.2 DRIVEN

Depth 0

N=8

N=4

N=9

N=13

N=6

ELEV. (ft)	SOIL AND ROCK DESCRIPTION	DEPTH (ft)
195.9	GROUND SURFACE	0.0
194.6	**ASPHALT**	1.3
191.9	**ROADWAY EMBANKMENT** Dark Gray, Silty Fine SAND (A-2-4)	4.0
188.9	Brown-Gray, Clayey Fine to Coarse SAND (A-2-6)	7.0
183.9	Dark Gray, Silty Fine SAND (A-2-4) with Trace Wood Fragments	12.0
168.9	Dark to Light Gray, Fine Sandy CLAY (A-6) with Trace Wood Fragments	27.0
163.9	**COASTAL PLAIN** Light Gray, Clayey Fine SAND (A-2-6)	32.0
158.9	Orange-Brown-Gray, Fine Sandy Silty CLAY (A-7)	37.0
153.9	Brown-Light Gray, Clayey Fine to Coarse SAND (A-2-6) with Trace Mica	42.0
143.9	Light Brown and Gray, Fine to Coarse SAND (A-3) with Trace Gravel	52.0
138.9	Orange-Brown, Fine Sandy SILT (A-4)	57.0
135.2	**WEATHERED ROCK** Blue-Gray (META-ARGILLITE)	60.7
	Boring Terminated with Standard Penetration Test Refusal at Elevation 135.2 ft on Crystalline Rock (META-ARGILLITE)	

189.2

173.9

Assume pile refusal ~1' below WR @ EL 137.9

Pile length = BOC − Tip

= 189.2 − 137.9 = 51.3

∴ Anticipated Pile Length = 52'

Average Pile Length = 53'

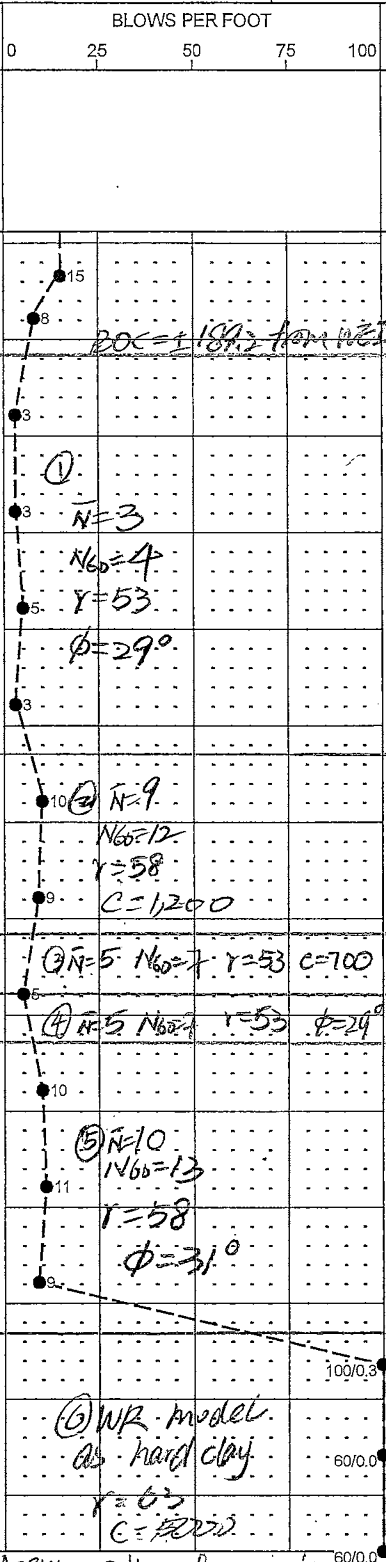
Only same pile length

Check downdrag @ EB1-B then

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** J. Cranston
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95			**GROUND WTR (ft)**
BORING NO. EB1-B	**STATION** 16+36	**OFFSET** 6 ft RT	**ALIGNMENT** -Y1- / **0 HR.** NM
COLLAR ELEV. 195.6 ft	**TOTAL DEPTH** 68.5 ft	**NORTHING** 652,456	**EASTING** 2,224,841 / **24 HR.** FIAD
DRILL RIG/HAMMER EFF./DATE F&R5785 CME-55 80% 02/11/2017	**DRILL METHOD** Mud Rotary		**HAMMER TYPE** Automatic
DRILLER D. Aiello	**START DATE** 04/14/17	**COMP. DATE** 04/14/17	**SURFACE WATER DEPTH** N/A

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
194.3	1.3	7	7	8	15		M
192.1	3.5	3	3	5	8	Depth 0	M
187.1	8.5	1	2	1	3		W
182.1	13.5	2	1	2	3		M
177.1	18.5	4	2	3	5		M
172.1	23.5	1	1	2	3	20.6	M
167.1	28.5	3	4	6	10		M
162.1	33.5	2	4	5	9	30.1	M
157.1	38.5	1	2	3	5	33.1 35.6	M
152.1	43.5	2	4	6	10		Sat.
147.1	48.5	5	6	5	11		Sat.
142.1	53.5	7	4	5	9	50.6	Sat.
137.1	58.5	100/0.3			100/0.3		
132.1	63.5	60/0.0			60/0.0		
127.1	68.5	60/0.0			60/0.0		



BOC = ± 189.2 from WEI

① N̄=3 N60=4 γ=53 ϕ=29°

② N̄=9 N60=12 γ=58 C=1,200

③ N̄=5 N60=7 γ=53 C=700

④ N̄=5 N60=7 γ=53 ϕ=29°

⑤ N̄=10 N60=13 γ=58 ϕ=31°

⑥ WR model as hard clay γ=63 C=~~[illegible]~~

ELEV. (ft)	SOIL AND ROCK DESCRIPTION	DEPTH (ft)
195.6	GROUND SURFACE	0.0
194.3	**ASPHALT**	1.3
	ROADWAY EMBANKMENT	
192.6	Dark Gray, Silty Fine SAND (A-2-4)	3.0
191.1	Brown-Black, Fine to Coarse SAND (A-3)	4.5
189.2	Gray, Clayey Fine SAND (A-2-6)	
188.6	Gray to Dark Gray, Fine Sandy CLAY (A-6) with Trace Organics	7.0
173.6	Gray, Fine Sandy Silty CLAY (A-7)	22.0
168.6	**COASTAL PLAIN** Brown-Gray, Fine Sandy Silty CLAY (A-7)	27.0
159.1		
156.1	Light Brown, Clayey Fine to Coarse SAND (A-2-6)	39.5
153.6	Light Brown, Fine to Coarse SAND (A-3) with Trace Gravel (48.5'-50.0')	42.0
143.6	Light Brown, GRAVEL (A-1-a) with Little Fine to Coarse Sand	52.0
138.6	**WEATHERED ROCK** Blue-Gray (META-ARGILLITE)	57.0
133.6	**CRYSTALLINE ROCK** (META-ARGILLITE)	62.0
127.1	Boring Terminated with Standard Penetration Test Refusal at Elevation 127.1 ft in Crystalline Rock (META-ARGILLITE)	68.5

Assume pile refuses ~1' into WR at El. 137.6'

Pile Length = BOC − TIP = 189.2 − 137.6 = 51.6'

∴ Anticipated Pile Length = 52'

Average Pile Length = 55'

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     =========================================================================

                      APILE for Windows, Version 2014.6.8

                       Serial Number :  293783516

                       A Program for Analyzing the Axial Capacity
                       and Short-term Settlement of Driven Piles
                                 under Axial Loading.
                     (c) Copyright ENSOFT, Inc., 1987-2014
                           All Rights Reserved

     =========================================================================


     This program is licensed to :

     Froehling & Robertson, Inc.
     Richmond, Virginia

     Path to file locations      : F:\Projects 66U\66U-0390 (WEI-I-5786 Bridges 108 & 111 Johnston
Co)\NON_CADD\Foundation Recs\Bridge 108 Lizzie Mill\APILE\
     Name of input data file     : End Bent 1.ap6d
     Name of output file         : End Bent 1.ap6o
     Name of plot output file    : End Bent 1.ap6p

     -------------------------------------------------------------------------
                          Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  May 17, 2017     Time:  14:11:22


1
          *********************
          * INPUT INFORMATION *
          *********************


          Bridge 108 End Bent 1

          DESIGNER   : C Wang

          JOB NUMBER : 66U-0390


          METHOD FOR UNIT LOAD TRANSFERS :

          - FHWA (Federal Highway Administration)
            Unfactored Unit Side Friction and Unit Side Resistance are used.



          COMPUTATION METHOD(S) FOR PILE CAPACITY :

          - FHWA (Federal Highway Administration)


          TYPE OF LOADING :
          - COMPRESSION


          PILE TYPE :

          H-Pile/Steel Pile
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DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.300E+08 PSI
- CROSS SECTION AREA        =     15.50 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     52.00 FT.
- PILE STICKUP LENGTH, PSL  =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     47.65 IN.
- TIP AREA OF PILE          =     15.50 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                  LATERAL    EFFECTIVE    FRICTION   BEARING
          SOIL    EARTH      UNIT         ANGLE      CAPACITY
 DEPTH    TYPE    PRESSURE   WEIGHT       DEGREES    FACTOR
  FT.                        LB/CF
   0.00   SAND      0.00        53.00        29.00      0.00
  20.60   SAND      0.00        53.00        29.00      0.00
  20.60   CLAY      0.00        58.00         0.00      0.00
  30.10   CLAY      0.00        58.00         0.00      0.00
  30.10   CLAY      0.00        53.00         0.00      0.00
  33.10   CLAY      0.00        53.00         0.00      0.00
  33.10   SAND      0.00        53.00        29.00      0.00
  35.60   SAND      0.00        53.00        29.00      0.00
  35.60   SAND      0.00        58.00        31.00      0.00
  50.60   SAND      0.00        58.00        31.00      0.00
  50.60   CLAY      0.00        63.00         0.00      0.00
  55.60   CLAY      0.00        63.00         0.00      0.00


MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF               KSF        KSF
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.20      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       1.20      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.70      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.70      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       5.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*       5.00      0.00   0.00       0.00      0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.


         LRFD FACTOR  LRFD FACTOR
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```
         ON UNIT     ON UNIT
 DEPTH   FRICTION    BEARING
  FT.
   0.00    1.000       1.000
  20.60    1.000       1.000
  20.60    1.000       1.000
  30.10    1.000       1.000
  30.10    1.000       1.000
  33.10    1.000       1.000
  33.10    1.000       1.000
  35.60    1.000       1.000
  35.60    1.000       1.000
  50.60    1.000       1.000
  50.60    1.000       1.000
  55.60    1.000       1.000


1
        **********************
        * COMPUTATION RESULT *
        **********************



                      **********************
                      *  FED. HWY. METHOD  *
                      **********************

   PILE        TOTAL SKIN      END       ULTIMATE
PENETRATION    FRICTION      BEARING     CAPACITY
    FT.           KIP          KIP          KIP
    0.00          0.0          0.0          0.0
    1.00          0.0          0.1          0.1
    2.00          0.1          0.2          0.3
    3.00          0.3          0.3          0.5
    4.00          0.5          0.3          0.8
    5.00          0.7          0.4          1.2
    6.00          1.1          0.5          1.6
    7.00          1.4          0.6          2.0
    8.00          1.9          0.7          2.6
    9.00          2.4          0.8          3.1
   10.00          2.9          0.8          3.8
   11.00          3.6          0.9          4.5
   12.00          4.2          1.0          5.3
   13.00          5.0          1.1          6.1
   14.00          5.8          1.2          7.0
   15.00          6.6          1.3          7.9
   16.00          7.5          1.3          8.9
   17.00          8.5          1.4          9.9
   18.00          9.5          1.4         11.0
   19.00         10.6          1.4         12.1
   20.00         11.8          1.4         13.2
   21.00         13.0          1.3         14.3
   22.00         16.0          1.2         17.2
   23.00         20.8          1.2         21.9
   24.00         25.5          1.2         26.7
   25.00         30.3          1.2         31.5
   26.00         35.1          1.2         36.2
   27.00         39.8          1.2         41.0
   28.00         44.6          1.2         45.8
   29.00         49.4          1.2         50.5
   30.00         54.1          1.0         55.2
   31.00         58.9          0.9         59.8
   32.00         62.7          0.8         63.4
   33.00         65.4          0.9         66.3
   34.00         68.2          1.1         69.3
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   35.00          70.7          1.4        72.1
   36.00          72.8          1.8        74.6
   37.00          75.2          2.0        77.2
   38.00          77.9          2.2        80.1
   39.00          80.6          2.2        82.8
   40.00          83.4          2.2        85.6
   41.00          86.3          2.2        88.5
   42.00          89.3          2.2        91.5
   43.00          92.3          2.2        94.5
   44.00          95.4          2.2        97.6
   45.00          98.6          2.2       100.8
   46.00         101.9          2.2       104.1
   47.00         105.2          2.2       107.4
   48.00         108.6          2.2       110.8
   49.00         112.1          2.2       114.3
   50.00         115.7          2.8       118.5
   51.00         119.3          3.5       122.8
   52.00         131.1          4.2       135.3
```

Skin friction @ 51' depth

```
AN ASTERISK WILL BE PLACED IN THE END-BEARING COLUMN
IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.
```

Driving resistance req'd

$\frac{80T}{0.6}$ = 133.5 round up to 135T (270 kips)

Piles likely refuse @ ~ 50'-51'! check Delmag D19-32,

% skin = $\frac{119.3}{270}$ ≈ 44%

WEAP Parameter Calculation

Bent #: **Bridge 108 EB1-B**

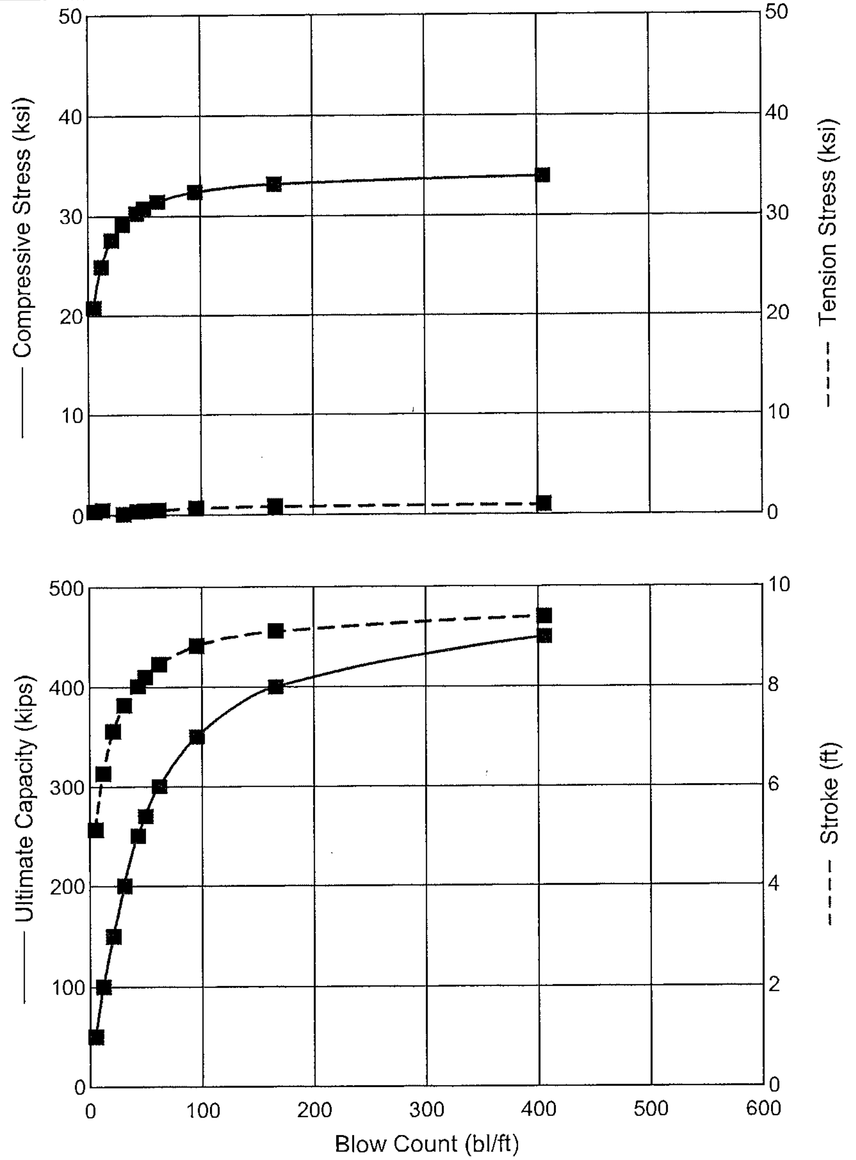
		Toe Quake	Shaft Quake
Pile Type:	HP 12X53	**0.10**	**0.10**

Subsurface Conditions: Loose/Soft or Submerged

Layer #	Top	Bottom	Navg	Soil Type	Shaft Damping	
1	189.2	168.6	3	Clay	0.30	
2	168.6	159.1	9	Clay	0.25	
3	159.1	156.1	5	Clay	0.30	
4	156.1	153.6	5	Sand	0.20	
5	153.6	138.6	10	Sand	0.20	
6						
7						
8						**Toe Damping**
					0.26	**0.10**

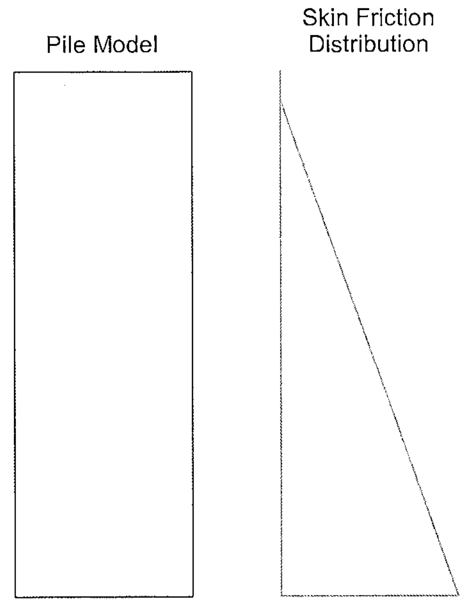
Froehling & Robertson, Inc.
Bridge 108 End Bent 1

17-May-2017
GRLWEAP Version 2010



DELMAG D 19-32

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1580 (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	0.260	sec/ft
Toe Damping	0.100	sec/ft
Pile Length	55.00	ft
Pile Penetration	52.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 44 %
(Proportional)

Froehling & Robertson, Inc.
Bridge 108 End Bent 1

17-May-2017
GRLWEAP Version 2010

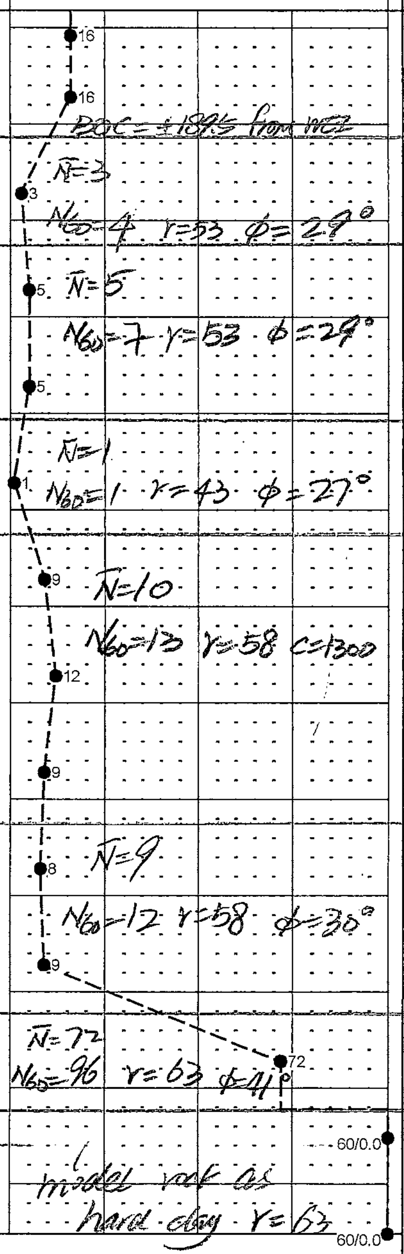
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
50.0	20.78	0.34	5.1	5.13	21.64
100.0	24.93	0.53	12.1	6.26	19.04
150.0	27.59	0.00	21.0	7.11	18.25
200.0	29.16	0.14	30.9	7.64	17.98
250.0	30.27	0.39	43.2	8.02	18.44
270.0	30.75 < 45 ✓	0.44	35 < ✓ 49.8 < 180 ✓	8.20	18.72
300.0	31.43	0.51	62.2	8.46	19.20
350.0	32.41	0.72	95.8	8.83	19.76
400.0	33.18	0.87	166.2	9.12	20.28
450.0	33.93	0.98	405.7	9.40	20.81

Delmag D19-32 or equivalent hammer
should be able to drive the piles @ EB1.

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** D. Racey	**GROUND WTR (ft)**
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95				
BORING NO. EB2-B	**STATION** 19+09	**OFFSET** 6 ft RT	**ALIGNMENT** -Y1-	**0 HR.** NM
COLLAR ELEV. 195.9 ft	**TOTAL DEPTH** 63.5 ft	**NORTHING** 652,367	**EASTING** 2,225,099	**24 HR.** FIAD
DRILL RIG/HAMMER EFF./DATE F&R4637 CME-75 81% 07/18/2015		**DRILL METHOD** Mud Rotary	**HAMMER TYPE** Automatic	
DRILLER S. Sequist	**START DATE** 04/14/17	**COMP. DATE** 04/14/17	**SURFACE WATER DEPTH** N/A	

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
195.6	0.3	27	7	9	16		W
192.4	3.5	30	11	5	16	Depth 0'	W
187.4	8.5	3	2	1	3	①	W
						5.6'	
182.4	13.5	4	3	2	5	②	W
177.4	18.5	1	2	3	5	14.6	W
172.4	23.5	WOR	WOH	1	1	③	Sat.
						20.6	
167.4	28.5	2	3	6	9		M
162.4	33.5	3	5	7	12	④	M
157.4	38.5	4	4	5	9	35.6	M
152.4	43.5	2	4	4	8	⑤	Sat.
147.4	48.5	2	3	6	9	45.6	W
142.4	53.5	26	36	36	72	⑥	M
						50.6	
137.4	58.5	60/0.0			60/0.0	⑦	
132.4	63.5	60/0.0			60/0.0		



BOC = ±189.5 from WEZ

$\bar{N}=3$

$N_{60}=4$ γ=53 φ=29°

$\bar{N}=5$

$N_{60}=7$ γ=53 φ=29°

$\bar{N}=1$

$N_{60}=1$ γ=43 φ=27°

$\bar{N}=10$

$N_{60}=13$ γ=58 C=1300

$\bar{N}=9$

$N_{60}=12$ γ=58 φ=30°

$\bar{N}=72$

$N_{60}=96$ γ=63 φ=41°

model rock as hard clay γ=63

C=100,000

ELEV. (ft)	SOIL AND ROCK DESCRIPTION	DEPTH (ft)
195.9	GROUND SURFACE	0.0
195.3	**ASPHALT**	0.6
	ROADWAY EMBANKMENT Gray-Tan, Clayey Silty Fine SAND (A-2-4)	
191.5	Red-Tan, Silty CLAY (A-7)	4.4
188.9 189.5'	Gray, Clayey Fine SAND (A-2-6)	7.0
183.9	Gray and Black, Fine Sandy CLAY (A-6) with Trace Organics (Roots)	12.0
174.9		
168.9	**COASTAL PLAIN** Gray-Tan-Red, Silty CLAY (A-7)	27.0
153.9	Gray-Tan, Fine to Coarse SAND (A-3), with Trace Gravel	42.0
148.9	Tan, Clayey Fine to Coarse SAND (A-2-6)	47.0
143.9	**RESIDUAL** Gray, Silty Fine SAND (A-2-4)	52.0
138.9	**CRYSTALLINE ROCK** Gray (META-ARGILLITE)	57.0
132.4	Boring Terminated with Standard Penetration Test Refusal at Elevation 132.4 ft in Crystalline Rock (META-ARGILLITE)	63.5

Assume pile refuses ~1' into CR. at EL 137.9

Pile Length = BOC − TIP = 189.5' − 137.9' = 51.6'

∴ Anticipated Pile Length = 52'

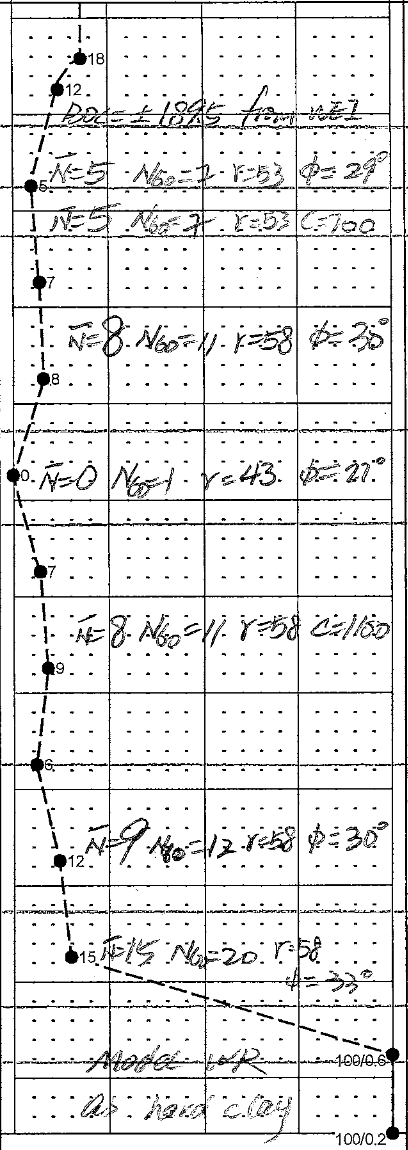
Average Pile Length = 55'

SHEET 10/2

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** J. Cranston
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95			**GROUND WTR (ft)**
BORING NO. EB2-A	**STATION** 19+19	**OFFSET** 6 ft LT	**ALIGNMENT** -Y1-
			0 HR. NM
COLLAR ELEV. 195.8 ft	**TOTAL DEPTH** 58.7 ft	**NORTHING** 652,375	**EASTING** 2,225,112
			24 HR. FIAD
DRILL RIG/HAMMER EFF./DATE F&R5785 CME-55 80% 02/11/2017	**DRILL METHOD** Mud Rotary		**HAMMER TYPE** Automatic
DRILLER D. Aiello	**START DATE** 04/11/17	**COMP. DATE** 04/11/17	**SURFACE WATER DEPTH** N/A

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
193.9	1.9	5	8	10	18		M
192.3	3.5	3	6	6	12	Depth 0	M
187.3	8.5	2	2	3	5	① 3.2 ② 5.7	W
182.3	13.5	2	3	4	7		W
177.3	18.5	4	4	4	8	③ 15.7	W
172.3	23.5	WOH	WOH	WOH	0	④ 20.7	M
167.3	28.5	2	3	4	7		M
162.3	33.5	3	4	5	9	⑤	M
157.3	38.5	3	3	3	6	33.2	M
152.3	43.5	4	5	7	12	⑥ 40.7	Sat.
147.3	48.5	5	7	8	15	⑦ 45.7	M
142.3	53.5	7	55	45/0.1	100/0.6	⑧	
137.3	58.5	100/0.2			100/0.2		



BOC = ± 189.5 from WEI

N̄=5 N_{60}=7 γ=53 ϕ=29°

N̄=5 N_{60}=7 γ=53 C=700

N̄=8 N_{60}=11 γ=58 ϕ=30°

N̄=0 N_{60}=1 γ=43 ϕ=27°

N̄=8 N_{60}=11 γ=58 C=1100

N̄=9 N_{60}=12 γ=58 ϕ=30°

N̄=15 N_{60}=20 γ=58 ϕ=33°

Model WR as hard clay

γ=63, C=6000

Assume pile refuses ~ 1' into WR at EL 142.8'

Pile Length = BOC − TIP = 189.5' − 142.8' = 46.7'

∴ Anticipated Pile Length = 47'

Average Pile Length = 50'

SOIL AND ROCK DESCRIPTION

ELEV. (ft)		DEPTH (ft)
195.8	GROUND SURFACE	0.0
	ASPHALT	
193.9		1.9
	ROADWAY EMBANKMENT	
192.4	Dark Gray, Silty Fine SAND (A-2-4)	3.4
191.8	Red-Brown, Fine Sandy CLAY (A-6)	4.0
189.5'	Gray, Clayey Fine SAND (A-2-6)	
186.3		9.5
	Gray, Fine Sandy Silty CLAY (A-7)	
183.8		12.0
	Dark Gray, Fine Sandy CLAY (A-6) with Trace Organics	
173.8		
168.8		27.0
	COASTAL PLAIN	
	Red-Brown-Gray, Fine Sandy CLAY (A-7)	
156.3		39.5
	Brown-Light Gray, Clayey Fine SAND (A-2-6) with Trace Mica	
153.8		42.0
	Light Brown, Fine to Coarse SAND (A-3) with Trace Rock Fragments	
148.8		47.0
	Red-Brown, Fine Sandy CLAY (A-6) with Trace Rock Fragments	
143.8		52.0
	WEATHERED ROCK (META-ARGILLITE)	
137.1		58.7
	Boring Terminated at Elevation 137.1 ft in Weathered Rock (META-ARGILLITE)	

```
=========================================================================

                 APILE for Windows, Version 2014.6.8

                  Serial Number :  293783516

                  A Program for Analyzing the Axial Capacity
                  and Short-term Settlement of Driven Piles
                            under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2014
                      All Rights Reserved

=========================================================================


This program is licensed to :

Froehling & Robertson, Inc.
Richmond, Virginia

Path to file locations      : F:\Projects 66U\66U-0390 (WEI-I-5786 Bridges 108 & 111 Johnston
Co)\NON_CADD\Foundation Recs\Bridge 108 Lizzie Mill\APILE\
Name of input data file     : End Bent 2 Lt.ap6d
Name of output file         : End Bent 2 Lt.ap6o
Name of plot output file    : End Bent 2 Lt.ap6p

-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  May 17, 2017     Time:  15:10:48


1
     *********************
     * INPUT INFORMATION *
     *********************



     Bridge 108 End Bent 2 Lt

     DESIGNER   : C Wang

     JOB NUMBER : 66U-0390


     METHOD FOR UNIT LOAD TRANSFERS :

     - FHWA (Federal Highway Administration)
       Unfactored Unit Side Friction and Unit Side Resistance are used.



     COMPUTATION METHOD(S) FOR PILE CAPACITY :

     - FHWA (Federal Highway Administration)


     TYPE OF LOADING :
     - COMPRESSION


     PILE TYPE :

     H-Pile/Steel Pile
```

```
DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY      = 0.300E+08 PSI
- CROSS SECTION AREA         =     15.50 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL      =     47.00 FT.
- PILE STICKUP LENGTH, PSL   =      0.00 FT.
- ZERO FRICTION LENGTH, ZFL  =      0.00 FT.
- PERIMETER OF PILE          =     47.65 IN.
- TIP AREA OF PILE           =     15.50 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION        =      1.00 FT.



SOIL INFORMATIONS :



                 LATERAL    EFFECTIVE    FRICTION   BEARING
         SOIL    EARTH      UNIT         ANGLE      CAPACITY
 DEPTH   TYPE    PRESSURE   WEIGHT       DEGREES    FACTOR
  FT.                       LB/CF
   0.00  SAND      0.00        53.00        29.00      0.00
   3.20  SAND      0.00        53.00        29.00      0.00
   3.20  CLAY      0.00        53.00         0.00      0.00
   5.70  CLAY      0.00        53.00         0.00      0.00
   5.70  SAND      0.00        58.00        30.00      0.00
  15.70  SAND      0.00        58.00        30.00      0.00
  15.70  SAND      0.00        43.00        27.00      0.00
  20.70  SAND      0.00        43.00        27.00      0.00
  20.70  CLAY      0.00        58.00         0.00      0.00
  33.20  CLAY      0.00        58.00         0.00      0.00
  33.20  SAND      0.00        58.00        30.00      0.00
  40.70  SAND      0.00        58.00        30.00      0.00
  40.70  SAND      0.00        58.00        33.00      0.00
  45.70  SAND      0.00        58.00        33.00      0.00
  45.70  CLAY      0.00        63.00         0.00      0.00
  50.00  CLAY      0.00        63.00         0.00      0.00



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR     BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH  COUNT  FRICTION   BEARING
  KSF       KSF       KSF        KSF              KSF        KSF
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.70      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.70      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      1.10      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      1.10      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      5.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      5.00      0.00   0.00       0.00      0.00
```

```
* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.



        LRFD FACTOR  LRFD FACTOR
         ON UNIT      ON UNIT
 DEPTH   FRICTION     BEARING
  FT.
   0.00    1.000       1.000
   3.20    1.000       1.000
   3.20    1.000       1.000
   5.70    1.000       1.000
   5.70    1.000       1.000
  15.70    1.000       1.000
  15.70    1.000       1.000
  20.70    1.000       1.000
  20.70    1.000       1.000
  33.20    1.000       1.000
  33.20    1.000       1.000
  40.70    1.000       1.000
  40.70    1.000       1.000
  45.70    1.000       1.000
  45.70    1.000       1.000
  50.00    1.000       1.000



1
**********************
* COMPUTATION RESULT *
**********************



                  **********************
                  *  FED. HWY. METHOD  *
                  **********************

   PILE       TOTAL SKIN      END       ULTIMATE
PENETRATION   FRICTION      BEARING     CAPACITY
    FT.          KIP          KIP         KIP
    0.00          0.0          0.0          0.0
    1.00          0.0          0.1          0.1
    2.00          0.1          0.2          0.3
    3.00          0.3          0.3          0.6
    4.00          0.5          0.5          0.9
    5.00          2.0          0.6          2.6
    6.00          4.8          0.7          5.4
    7.00          6.4          0.7          7.1
    8.00          6.9          0.8          7.7
    9.00          7.4          0.9          8.3
   10.00          8.0          1.0          9.0
   11.00          8.7          1.1          9.8
   12.00          9.5          1.2         10.7
   13.00         10.3          1.3         11.6
   14.00         11.2          1.4         12.6
   15.00         12.1          1.3         13.5
   16.00         13.2          1.2         14.4
   17.00         14.2          1.1         15.3
   18.00         15.1          1.1         16.2
   19.00         16.1          1.1         17.2
   20.00         17.1          1.1         18.2
   21.00         18.1          1.1         19.3
   22.00         20.9          1.1         22.0
```

```
  23.00            25.2            1.1          26.3
  24.00            29.6            1.1          30.7
  25.00            34.0            1.1          35.0
  26.00            38.3            1.1          39.4
  27.00            42.7            1.1          43.8
  28.00            47.1            1.1          48.1
  29.00            51.4            1.1          52.5
  30.00            55.8            1.1          56.9
  31.00            60.2            1.1          61.2
  32.00            64.5            1.1          65.6
  33.00            68.9            1.2          70.1
  34.00            73.3            1.2          74.5
  35.00            76.6            1.3          78.0
  36.00            78.9            1.4          80.4
  37.00            81.3            1.4          82.7
  38.00            83.8            1.4          85.2
  39.00            86.3            1.4          87.7
  40.00            88.9            2.4          91.3
  41.00            91.5            3.4          95.0
  42.00            94.7            4.4          99.1
  43.00            98.3            5.4         103.7
  44.00           102.0            5.4         107.4
  45.00           105.8            5.3         111.1
  46.00           109.7            5.1         114.9
  47.00           121.2            5.0         126.2
```

AN ASTERISK WILL BE PLACED IN THE END-BEARING COLUMN
IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

Shorter pile may control
the stress of the pile while driven
So, check the hammer @ EB2 (Lt)

Piles likely refuse @ around 46'~47'
depth

Skin friction @ 46' = 109.7

% Skin = $\frac{109.7}{270} \approx 41\%$

WEAP Parameter Calculation

Bent #: **Bridge 108 EB2-A**

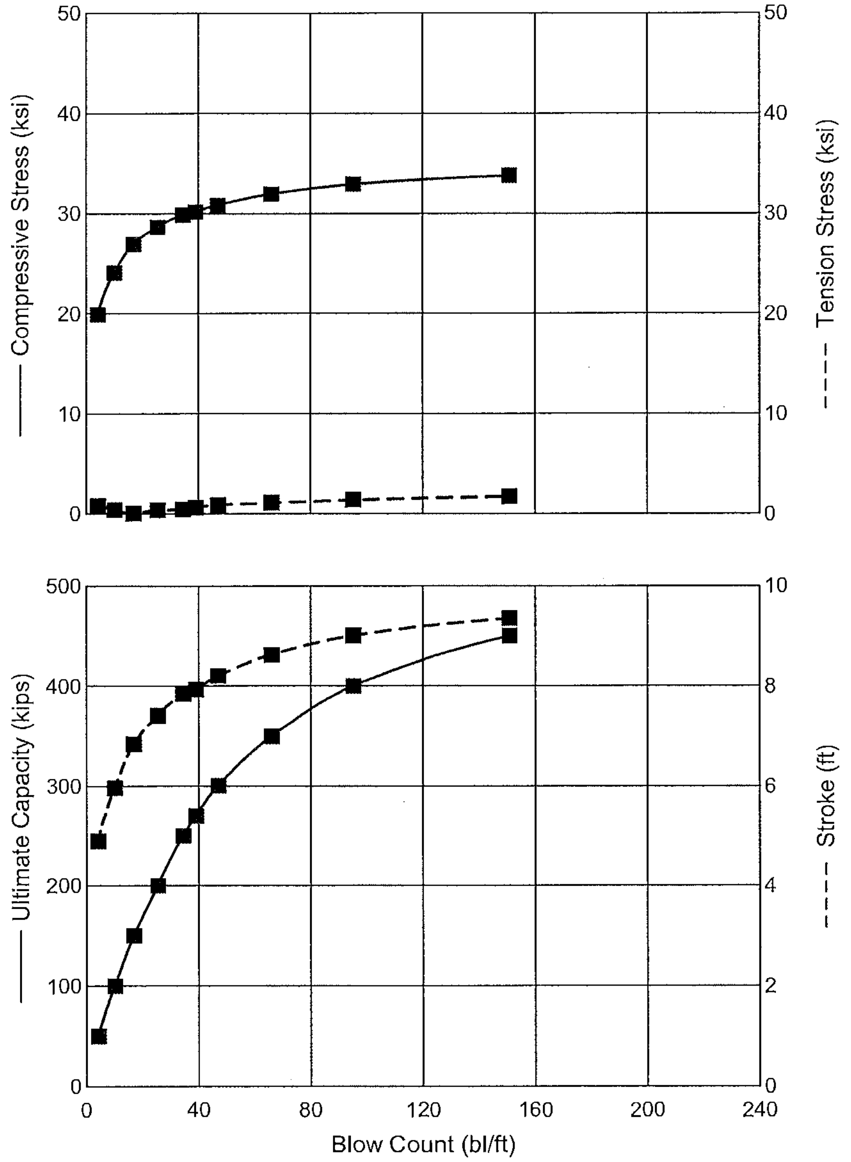
		Toe Quake	Shaft Quake
Pile Type:	HP 12X53	**0.10**	**0.10**

Subsurface Conditions: Loose/Soft or Submerged

Layer #	Top	Bottom	Navg	Soil Type	Shaft Damping	
1	189.5	186.3	5	Sand	0.20	
2	186.3	183.8	5	Clay	0.30	
3	183.8	173.8	8	Sand	0.20	
4	173.8	168.8	0	Sand	0.20	
5	168.8	156.3	8	Clay	0.25	
6	143.9	148.8	9	Sand	0.20	
7	148.8	143.8	15	Sand	0.18	
8						**Toe Damping**
					0.16	**0.10**

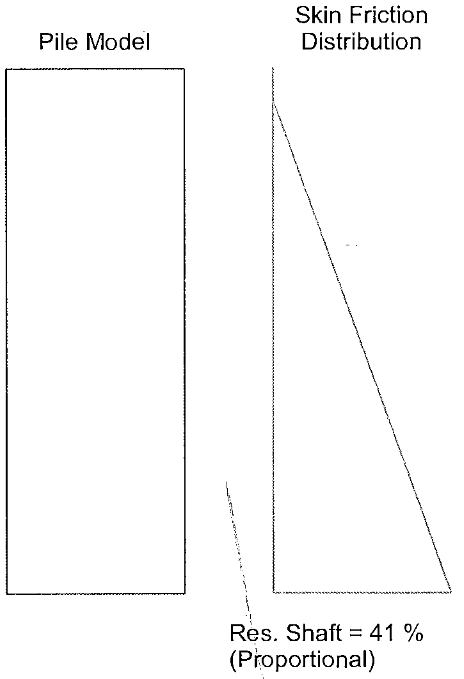
Froehling & Robertson, Inc.
Bridge 108 End Bent 2 Lt

17-May-2017
GRLWEAP Version 2010



DELMAG D 19-32

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1580 (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	0.160	sec/ft
Toe Damping	0.100	sec/ft
Pile Length	50.00	ft
Pile Penetration	47.00	ft
Pile Top Area	15.50	in2



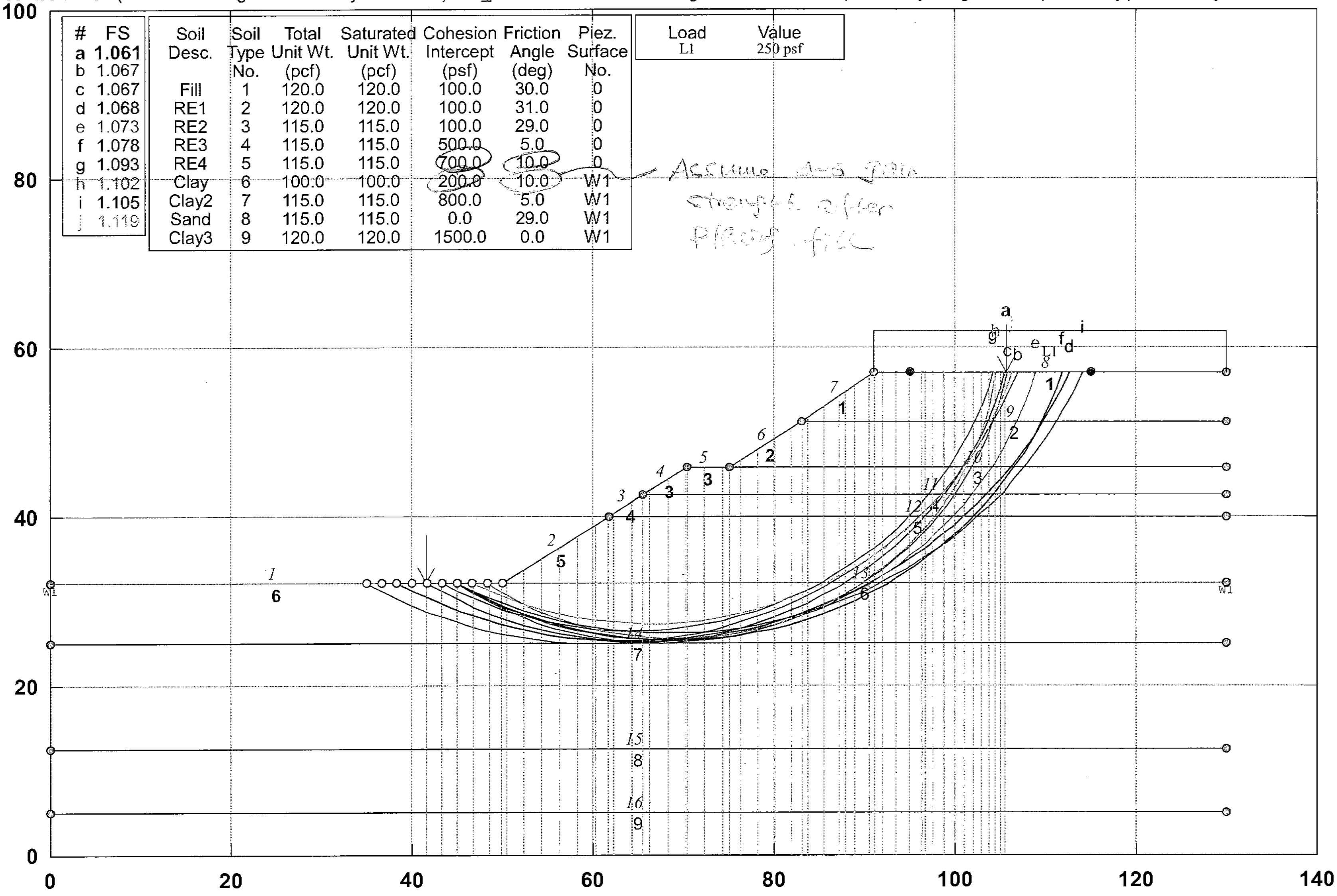
Froehling & Robertson, Inc. 17-May-2017
Bridge 108 End Bent 2 Lt GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
50.0	19.90	0.82	4.3	4.89	22.42
100.0	24.07	0.41	10.2	5.96	19.60
150.0	26.90	0.04	17.0	6.84	18.67
200.0	28.62	0.40	25.6	7.41	18.22
250.0	29.83	0.47	34.7	7.84	18.35
270.0	30.13 <45 ✓	0.63	✓ 30< 39.2 <180 ✓	7.93	18.44
300.0	30.83	0.90	47.1	8.20	18.78
350.0	31.95	1.11	66.2	8.63	19.44
400.0	32.93	1.40	95.3	9.01	20.24
450.0	33.78	1.72	151.0	9.35	20.96

Delmag D19 or equivalent hammer
should be able to drive the piles @ EB2

66U-0390 Bridge 108 Slope stability

f:\projects 66u\66u-0390 (wei-i-5786 bridges 108 & 111 johnston co)\non_cadd\foundation recs\bridge 108 lizzie mill\slope stability\bridge 108 slope stability.pl2 Run By: RES 5/15/2017 11:42AM



#	FS
a	1.061
b	1.067
c	1.067
d	1.068
e	1.073
f	1.078
g	1.093
h	1.102
i	1.105
j	1.119

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	120.0	120.0	100.0	30.0	0
RE1	2	120.0	120.0	100.0	31.0	0
RE2	3	115.0	115.0	100.0	29.0	0
RE3	4	115.0	115.0	500.0	5.0	0
RE4	5	115.0	115.0	700.0	10.0	0
Clay	6	100.0	100.0	200.0	10.0	W1
Clay2	7	115.0	115.0	800.0	5.0	W1
Sand	8	115.0	115.0	0.0	29.0	W1
Clay3	9	120.0	120.0	1500.0	0.0	W1

Load	Value
L1	250 psf

Assume 4-5 gain
strength after
proof fill

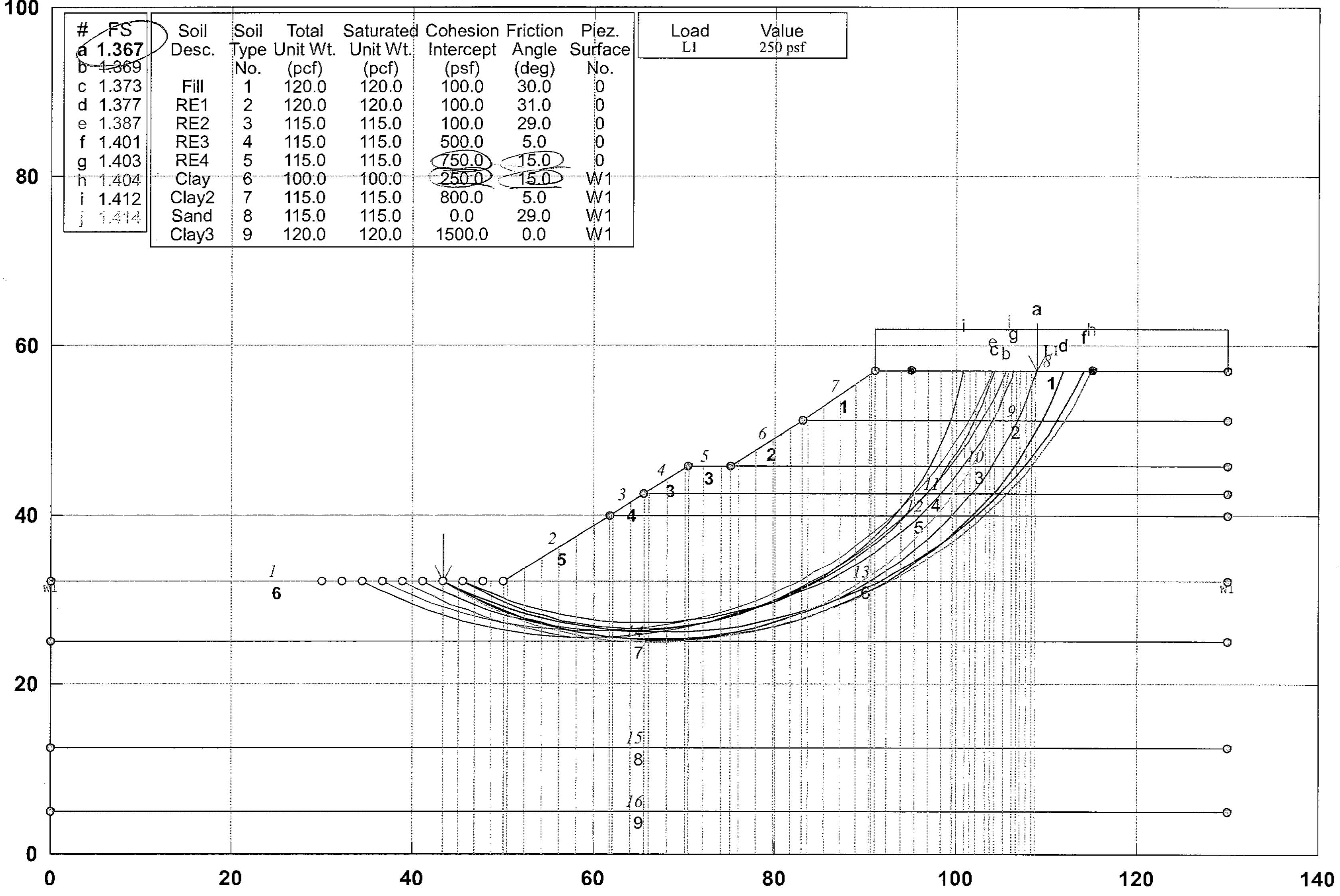
GSTABL7 v.2 FSmin=1.061

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

66U-0390 Bridge 108 Slope stability

f:\projects 66u\66u-0390 (wei-i-5786 bridges 108 & 111 johnston co)\non_cadd\foundation recs\bridge 108 lizzie mill\slope stability\bridge 108 slope stability.pl2   Run By: RES   5/15/2017   11:47AM



#	FS
a	1.367
b	1.369
c	1.373
d	1.377
e	1.387
f	1.401
g	1.403
h	1.404
i	1.412
j	1.414

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	120.0	120.0	100.0	30.0	0
RE1	2	120.0	120.0	100.0	31.0	0
RE2	3	115.0	115.0	100.0	29.0	0
RE3	4	115.0	115.0	500.0	5.0	0
RE4	5	115.0	115.0	750.0	15.0	0
Clay	6	100.0	100.0	250.0	15.0	W1
Clay2	7	115.0	115.0	800.0	5.0	W1
Sand	8	115.0	115.0	0.0	29.0	W1
Clay3	9	120.0	120.0	1500.0	0.0	W1

Load	Value
L1	250 psf

GSTABL7 v.2 FSmin=1.367

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

SINCE 1881

FROEHLING & ROBERTSON, INC.

Engineering • Environmental • Geotechnical

SHEET NO. 1 OF 1

JOB 66U-0380

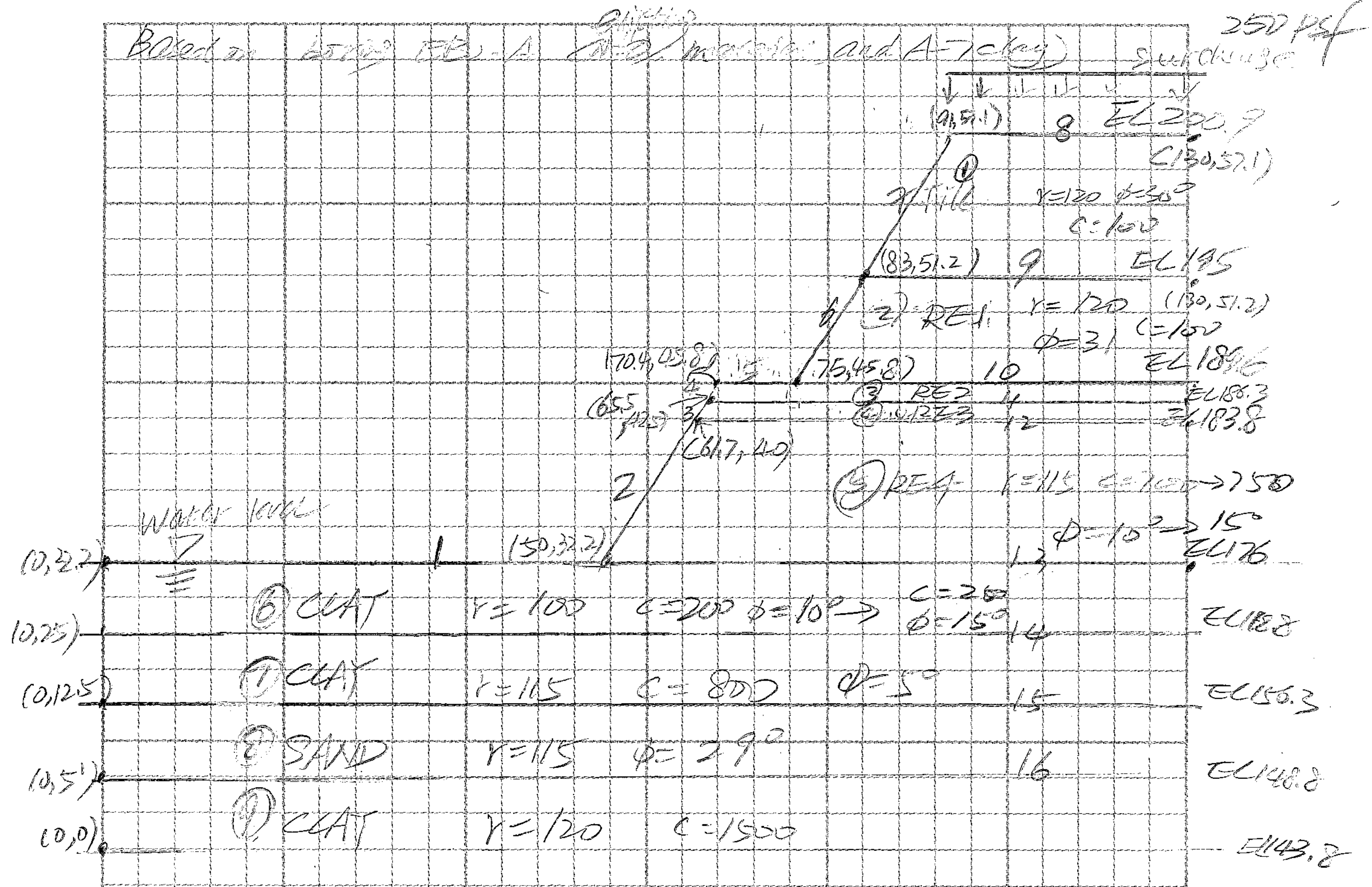
DATE 5/12/17

COMPUTATIONS FOR bridge 28 slope stability

BY C. Wang

CHKD

Based on boring FB-A (A-2 (shifting) material and A-7 clay)

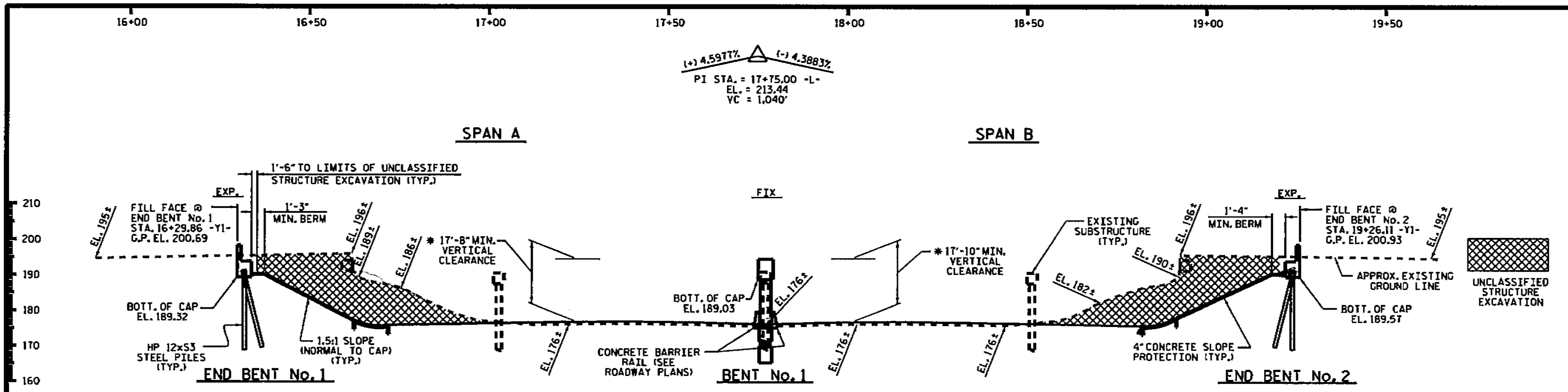


③ RE2 γ=115 ϕ=29° c=100

④ RE3 γ=115 c=500 ϕ=5°

[illegible] assume sdy clay [illegible]

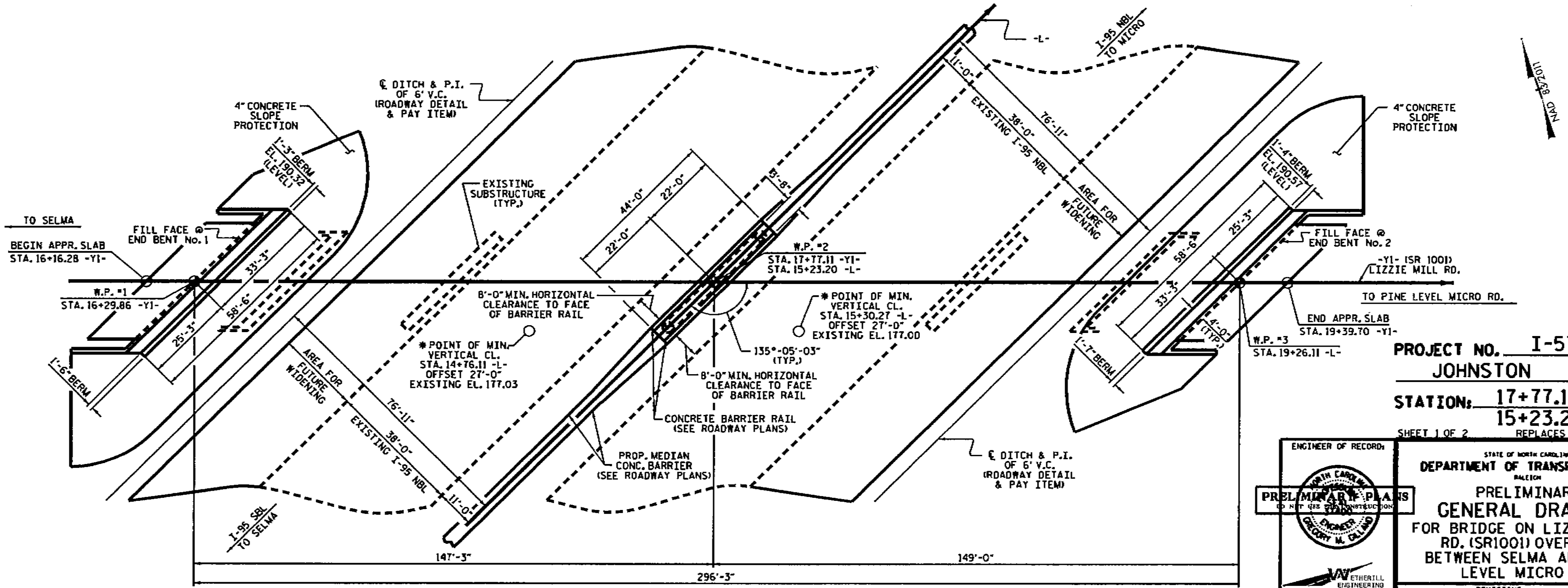
Form No. 502
Revised 05/2008



SECTION ALONG -Y1-

SECTIONS AT END BENTS AND BENT ARE TAKEN AT RIGHT ANGLES

*NOTE: MINIMUM VERTICAL CLEARANCE SHOWN INCLUDES AN 8" OVERLAY ON I-95. ELEVATIONS SHOWN IN THE PLAN VIEW DO NOT INCLUDE THE 8" OVERLAY.



PLAN

PILES AND DRILLED PIERS NOT SHOWN FOR CLARITY

PROJECT NO. I-5786

JOHNSTON COUNTY

STATION: 17+77.11 -Y1-
15+23.20 -L-

SHEET 1 OF 2
REPLACES BRIDGE No. 108

ENGINEER OF RECORD:

PRELIMINARY PLANS
DO NOT USE FOR CONSTRUCTION

GREGORY M. GILLAND

WETHERILL ENGINEERING
1223 Jones Franklin Rd.
Raleigh, N.C. 27606
Bus: 919 851 8077
Fax: 919 851 8107
LICENSE NO. F-0377

STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
RALEIGH

PRELIMINARY
GENERAL DRAWING
FOR BRIDGE ON LIZZIE MILL RD. (SR1001) OVER I-95 BETWEEN SELMA AND PINE LEVEL MICRO RD.

REVISIONS						SHEET NO.
NO.	BY:	DATE:	NO.	BY:	DATE:	
1			3			TOTAL SHEETS
2			4			

DOCUMENT NOT CONSIDERED FINAL UNLESS ALL SIGNATURES COMPLETED

DRAWN BY : D. HODGE DATE : 4/17
CHECKED BY : B.C. HUNT DATE : 4/17

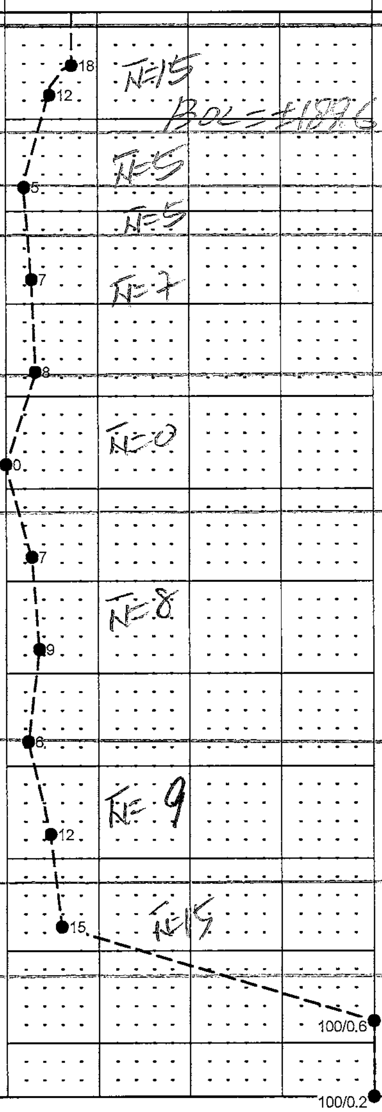
$FILE$
$DATE$
$TIME$

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** J. Cranston
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95			**GROUND WTR (ft)**
BORING NO. EB2-A	**STATION** 19+19	**OFFSET** 6 ft LT	**ALIGNMENT** -Y1-
COLLAR ELEV. 195.8 ft	**TOTAL DEPTH** 58.7 ft	**NORTHING** 652,375	**EASTING** 2,225,112
DRILL RIG/HAMMER EFF./DATE F&R5785 CME-55 80% 02/11/2017		**DRILL METHOD** Mud Rotary	**HAMMER TYPE** Automatic
DRILLER D. Aiello	**START DATE** 04/11/17	**COMP. DATE** 04/11/17	**SURFACE WATER DEPTH** N/A

0 HR. NM
24 HR. FIAD

ELEV (ft)	DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	BLOW COUNT 0.5ft	BLOW COUNT 0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
200							①	
195							51.2	
	193.9	1.9	5	8	10	18	②	M
190	192.3	3.5	3	6	6	12	45.8	M
185	187.3	8.5	2	2	3	5	③ 42.5 ④ 40	W
180	182.3	13.5	2	3	4	7	⑤	W
175	177.3	18.5	4	4	4	8	37.2	W
170	172.3	23.5	WOH	WOH	WOH	0	⑥ 25'	M
165	167.3	28.5	2	3	4	7		M
160	162.3	33.5	3	4	5	9	⑦	M
155	157.3	38.5	3	3	3	6	12.5	M
150	152.3	43.5	4	5	7	12	⑧ 5'	Sat.
145	147.3	48.5	5	7	8	15	⑨ 0'	M
140	142.3	53.5	7	55	45/0.1	100/0.6		
	137.3	58.5	100/0.2			100/0.2		



ELEV. (ft)	SOIL AND ROCK DESCRIPTION	DEPTH (ft)
195.8	EL195 GROUND SURFACE	0.0
	ASPHALT	
193.9		1.9
	ROADWAY EMBANKMENT Dark Gray, Silty Fine SAND (A-2-4)	
192.4		3.4
191.8	Red-Brown, Fine Sandy CLAY (A-6)	4.0
189.6	Gray, Clayey Fine SAND (A-2-6)	
186.3	Gray, Fine Sandy Silty CLAY (A-7)	9.5
183.8	Dark Gray, Fine Sandy CLAY (A-6) with Trace Organics	12.0
	EL176	
168.8	**COASTAL PLAIN** Red-Brown-Gray, Fine Sandy CLAY (A-7)	27.0
156.3	Brown-Light Gray, Clayey Fine SAND (A-2-6) with Trace Mica	39.5
153.8	Light Brown, Fine to Coarse SAND (A-3) with Trace Rock Fragments	42.0
148.8	Red-Brown, Fine Sandy CLAY (A-6) with Trace Rock Fragments	47.0
143.8	**WEATHERED ROCK** (META-ARGILLITE)	52.0
137.1	Boring Terminated at Elevation 137.1 ft in Weathered Rock (META-ARGILLITE)	58.7

STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
HIGHWAY BUILDING
1589 MAIL SERVICE CENTER
RALEIGH, NORTH CAROLINA 27699-1589

SUBJECT: Bridge No. 108 on SR 1001 (Lizzie Mill Rd.) over I-95

PREPARED BY:	CW	TIP NO.:
DATE:	5/17	I-5786
CHECKED BY:	WPA	COUNTY: Johnston
DATE:	5/17	

INTERIOR BENT SUMMARY

Foundation Type:	48-inch drilled piers	
Bottom of Cap Elevation:	189.16 ft	Provided by WEI
Top of Pier Elevation:	175.16 ft	Provided by WEI
Max Factored Load:	690 Tons/Pier	Provided by WEI, rounded up to nearest 5 tons
Required factored Resistance:	715 Tons/Pier	Per NCDOT axial spreadsheet rounded up to the nearest 5 tons
Required Tip Resistance:	95 tons/ft^2	Per NCDOT axial spreadsheet
Point of Fixity (POF) Elevation:	154.0 ft	"LPILE" program calculations
Tip No Higher Than Elevation:	132.0 ft (Lt & Ctr)	Axial Capacity Calculations
	134.0 ft (Rt)	Axial Capacity Calculations

NOTES

See Notes on Sheet 2 of the Foundation Recommendations

COMMENTS

See Comments on Sheet 3 of the Foundation Recommendations

STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
HIGHWAY BUILDING
PO BOX 25201
RALEIGH, NORTH CAROLINA 27611

SUBJECT: Bridge No. 108 on SR 1001 (Lizzie Mill Rd.) over I-95		WBS Element No.:
		N/A
PREPARED BY:	CW	COUNTY:
DATE:	May-17	Johnston
CHECKED BY:	WPA	TIP # : I-5786
DATE:	May-17	

QUANTITY OF DRILLED PIER IN SOIL & NOT IN SOIL CALCULATION SHEET

INTERIOR BENTS (Elevs. In feet)

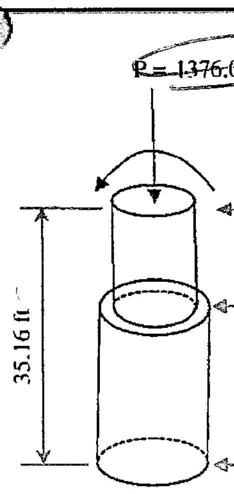
	B1-A	B1-B
APPROX. TOP OF SHAFT ELEV. =	175.2	175.2
TOP OF WEATHERED ROCK (WR) ELEV. =	139.4	145.3
TOP OF CRYSTALLINE ROCK (CR) ELEV. =	139.4	133.8
TIP NO HIGHER THAN ELEV. =	131.9	133.8
NUMBER OF SHAFTS PER BORING:	**2.0**	**1.0**
QUANTITY OF PIERS BELOW TOP OF SHAFT:	86.5	41.4
QUANTITY OF PIERS IN WR - Total:	0.0	11.5
QUANTITY OF PIERS IN WR - Soil (1/2 of total):	0.0	5.8
QUANTITY OF PIERS IN WR - Not in Soil (1/2 of total):	0.0	5.8
QUANTITY OF PIERS IN CR - Not in Soil:	15.0	0.0
TOTAL QUANTITY OF PIERS NOT IN SOIL:	15.0	5.8
ROUND UP TO NEAREST WHOLE NUMBER:	15	6
TOTAL QTY OF PIERS NOT IN SOIL (lf)	**21**	
TOTAL QUANTITY OF PIERS IN SOIL:	71.5	35.6
ROUND UP TO NEAREST WHOLE NUMBER:	72.0	36.0
TOTAL QTY OF PIERS IN SOIL (lf)	**108**	

WETHERILL ENGINEERING, INC.
1223 JONES FRANKLIN RD.
RALEIGH, NC 27606
(919) 851-8077 * (919) 851-8107 (FAX)

STR. NO.: 108 TIP NO.: I-5786 COUNTY: JOHNSTON
SUBJECT: CONTROLLING COLUMN LOADS (STRENGTH)
PREPARED BY: GMG DATE: 4/28/17 STATION:
CHECKED BY: DATE: 5/2/17 SHEET 1 OF 2

bmg 5/2/17

Col #	1		2		3							
Node #	3	3a	6	6a	9	9a						


~~P = 1376.0 kips~~ = 688T round up to 690T

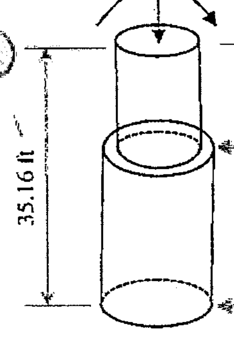
M = 128.9 k-ft L

V = 5.8 kips L

Bottom of Col. Elev. = 175.16 ft

Preliminary POF Elev. = 154.00 ft

MAXIMUM AXIAL LOAD WITH LONGITUDINAL SHEAR AND MOMENT


P = 833.0 kips 833,000 lbs

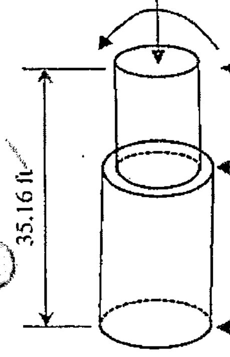
M = 64.5 k-ft = 774,000 lbs.in R

V = 30.2 kips R

Bottom of Col. Elev. = 175.16 ft

Preliminary POF Elev. = 154.00 ft

MAXIMUM LONGITUDINAL SHEAR WITH AXIAL LOAD AND LONGITUDINAL MOMENT


P = 809.3 kips

M = 490.8 k-ft L

V = 28.7 kips L

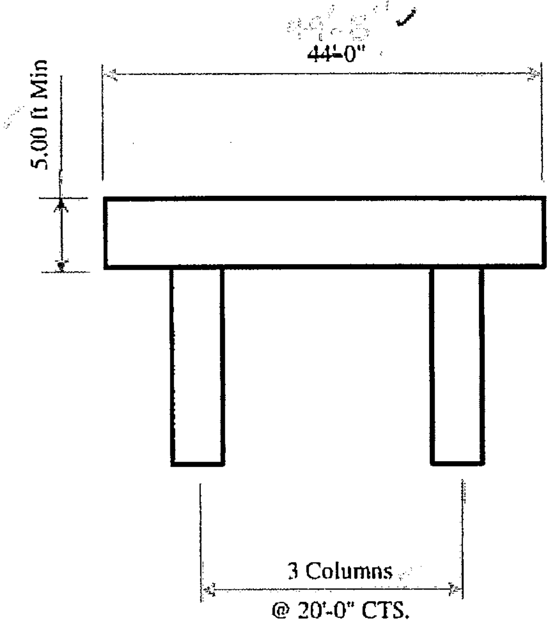
Bottom of Col. Elev. = 175.16 ft

Preliminary POF Elev. = 154.00 ft

MAXIMUM TRANSVERSE SHEAR WITH AXIAL LOAD AND TRANSVERSE MOMENT

NOTES:

Column Φ = 42 in = 3' - 6"
Drilled Shaft Φ = 48 in = 4' - 0"

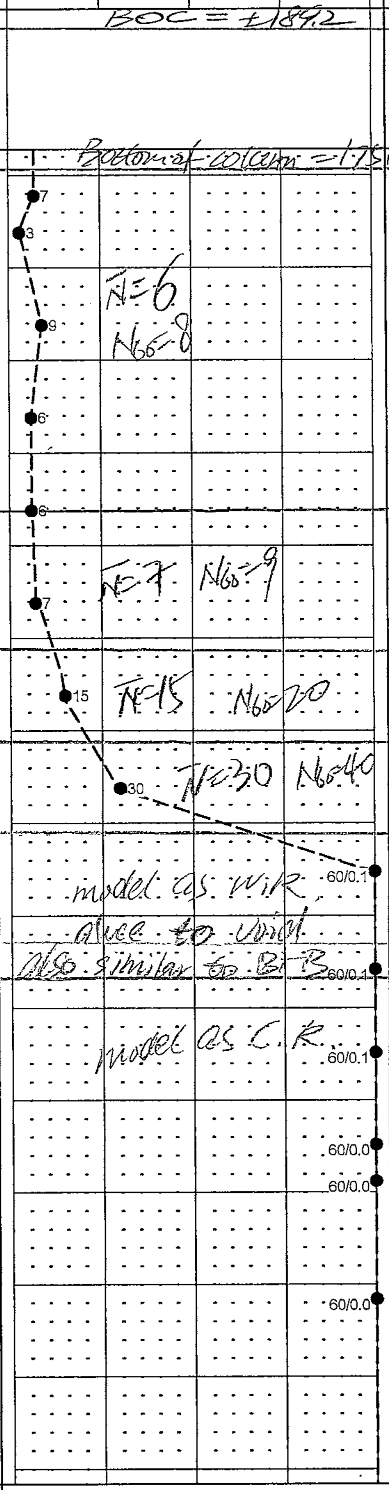


Bent No.	Bottom of Cap
1	~~189.16 ft~~

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** J. Cranston
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95			**GROUND WTR (ft)**
BORING NO. B1-A	**STATION** 17+85	**OFFSET** 7 ft LT	**ALIGNMENT** -Y1- / **0 HR.** NM
COLLAR ELEV. 176.4 ft	**TOTAL DEPTH** 72.2 ft	**NORTHING** 652,420	**EASTING** 2,224,986 / **24 HR.** FIAD
DRILL RIG/HAMMER EFF./DATE F&R5785 CME-55 80% 02/11/2017		**DRILL METHOD** Mud Rotary	**HAMMER TYPE** Automatic
DRILLER D. Aiello	**START DATE** 04/12/17	**COMP. DATE** 04/13/17	**SURFACE WATER DEPTH** N/A

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
174.9	1.5	23	4	3	7		M
172.9	3.5	1	1	2	3		M
167.9	8.5	2	4	5	9	①	M
162.9	13.5	2	2	4	6		M
157.9	18.5	2	2	4	6		Sat.
152.9	23.5	4	3	4	7	②	Sat.
147.9	28.5	4	6	9	15	③	Sat.
142.9	33.5	40	16	14	30	④	M
137.9	38.5	60/0.1			60/0.1	⑤	
132.9	43.5	60/0.1			60/0.1		
127.9	48.5	60/0.1			60/0.1	⑥	
122.9	53.5	60/0.0			60/0.0		
120.7	55.7	60/0.0			60/0.0		
114.2	62.2	60/0.0			60/0.0	RS-1	
						RS-2	



BOC = ±189.2

Bottom of column = 175.2'

N=6 N60=8

N=7 N60=9

N=15 N60=20

N=30 N60=40

model as W.R. due to void also similar to B1-B

model as C.R.

ELEV. (ft)	SOIL AND ROCK DESCRIPTION	DEPTH (ft)
189.2		
176.4	GROUND SURFACE	0.0
174.9	**ASPHALT**	1.5
	ROADWAY EMBANKMENT Dark Gray, Fine Sandy Silty CLAY (A-7) with Trace Organics and Trace Gravel	
169.4	**COASTAL PLAIN** Light Brown-Gray, Fine Sandy Silty CLAY (A-7)	7.0
156.9	Light Brown, Fine to Coarse SAND (A-3) with Trace Gravel	19.5
149.4	Light Brown, GRAVEL (A-1-a) with Little Fine to Coarse Sand	27.0
144.4	**RESIDUAL** Orange-Brown, Fine Sandy SILT (A-4)	32.0
139.4	**CRYSTALLINE ROCK** Blue-Gray (META-ARGILLITE)	37.0
133.4		43.0
131.9		44.5
	Void Encountered from 43.0'-44.5'	
120.7		55.7
114.2	Blue-Gray (META-ARGILLITE)	62.2
104.2	Boring Terminated at Elevation 104.2 ft in Crystalline Rock (META-ARGILLITE)	72.2

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING UNIT
Drilled Pier Axial Resistance Worksheet

PROJECT: I-5786 COUNTY Johnston
DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 left
DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
CHECKED BY: DATE: STR. NO.: PAGE: 1 OF 9

Elevations

Bottom of Cap (BOC) Elevation = 189.20 ft
Top of Pier/Bottom of Column Elevation = 175.20 ft
Natural Ground / Finished Grade Elevation = 176.00 ft
Groundwater Table (GWT) Elevation = 0.00 ft
Design Scour (DSE) Elevation = 176.00 ft
Amount of Contraction Scour (from BSR) = 0.00 ft
Is Permanent Casing Required? ○ Yes / Maybe ● No
Bottom of Permanent Casing Elevation = N/A ft
Drilled Pier Tip Elevation = 131.90 ft

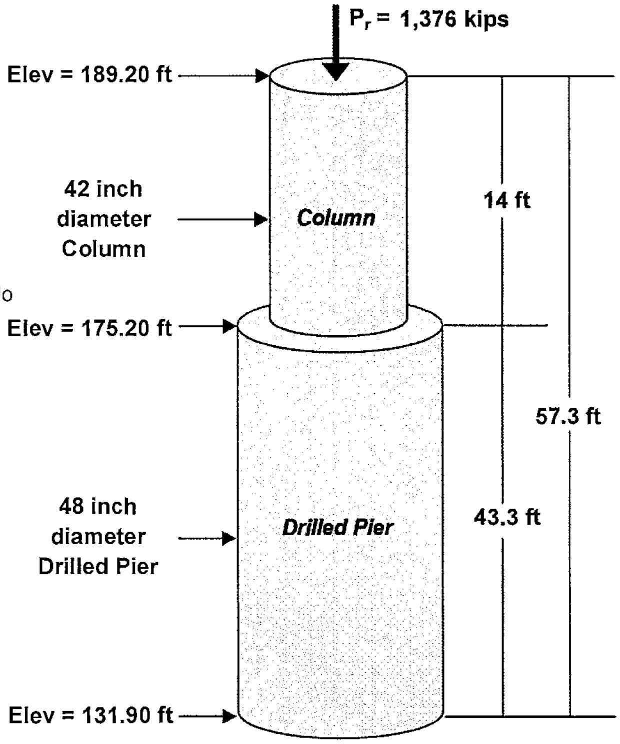


Figure shows typical drilled pier

Drilled Pier Information

Maximum Factored Axial Load (P_r) = 1,376.0 kips
Number of Drilled Piers per Bent = 2
Diameter of Column (d_{Column}) = 42 in
Diameter of Drilled Pier (d_{DP}) = 48 in
Unit Weight of Concrete (γ_c) = 0.150 kcf
Compressive Strength of Concrete (f'_c) = 4.500 ksi

Subsurface Information and Soil/Rock Layer Properties

internally calculate $N1_{60}$ values at midpoint of each layer ☑

Subsurface Boring Name / ID No. = B1-A
SPT Hammer Energy Efficiency Rating (ER) = 80 %
Top of Boring (Collar) Elevation = 176.40 ft
Depth to Groundwater Table (for actual boring) = 0.00 ft

Calculate GSI using RQD values : ☐
(Use if GSI is not shown on boring)

Layer No.	Material Description	Layer Elevations		Total γ (kcf)	N (bpf)	N_{60} (bpf)	$N1_{60}$ (bpf)	RQD (%)	GSI [2]	q_u (ksf)	E_i (ksi)	v
		Top[1] (ft)	Bottom (ft)									
1	Cohesive Soil (Clay)	175.20	156.90	0.115	6	8	11					
2	Cohesionless Soil (Sand)	156.90	149.40	0.115	7	9	10					
3	Cohesionless Soil (Gravel)	149.40	144.40	0.120	15	20	22					
4	Cohesionless Soil (Silty Sand)	144.40	139.40	0.125	30	40	41					
5	Weathered Rock	139.40	131.90	0.130	100	133	128					
6	Hard Rock	131.90	131.90	0.130	100		N/A	38	35	346		
7												
8												
TIP [3]	Hard Rock	131.90	123.90	0.130	150		N/A	57	35	753	830	0.200

Notes

1. Resistance from subsurface layers above the Bottom of Column Elevation, Drilled Pier Design Scour Elevation, and Permanent Casing Elevation will be ignored.
2. Hard rock layers with poor or very poor quality rock mass (GSI < 30) will be modeled as weathered rock.
3. Input the subsurface information for the soil / rock at the base of the drilled pier to a distance of 2 pier diameters below the base of the drilled pier.

DISCLAIMER: *The application of this spreadsheet is the responsibility of the user. It is imperative that the user understands the potential accuracy limitations and examines the reasonableness of the results with engineering knowledge and experience. There are no expressed or implied warranties.*

NORTH CAROLINA DEPARTMENT
OF TRANSPORTATION
GEOTECHNICAL ENGINEERING UNIT
Drilled Pier Axial Resistance Worksheet

PROJECT: I-5786 COUNTY Johnston
DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 left
DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
CHECKED BY: DATE: STR. NO.: PAGE: 1A OF 9

Correcting SPT Values for Hammer Efficiency and Overburden Pressure

SPT-N Value Corrected for Hammer Efficiency, (N_{60})

$N_{60} = (ER/60\%)(N)$ *AASHTO Eqn. 10.4.6.2.4-2*

N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used. If ER is not known, use 80% for automatic hammers and 60% for drop hammers.
N = uncorrected SPT blow count (blows/ft)

SPT-N Value Corrected for Overburden Pressure, (N1)

$N1 = (C_N)(N)$ *AASHTO Eqn. 10.4.6.2.4-1*

N1 = SPT blow count corrected for overburden pressure (blows/ft)
C_N = correction factor = $[0.77\log_{10}(40/\sigma'_v) < 2.0$

$\sigma'_v = \sigma_v - \mu$ = effective vertical stress at the depth of the SPT-N value (ksf)

σ_v = total vertical stress at the depth of the SPT-N value (ksf)
μ = total pore water pressure at the depth of the SPT-N value (ksf)

N = uncorrected SPT blow count (blows/ft)

SPT-N Value Corrected for both Overburden Pressure and Hammer Efficiency, ($N1_{60}$)

$N1_{60} = (C_N)(N)$ *AASHTO Eqn. 10.4.6.2.4-3*

Summary of Corrected N Values for Boring

Top of Boring (Collar) Elevation = 176.4 ft
Depth to Groundwater Table = 0.0 ft
Hammer Efficiency (ER) = 80 %
Unit Weight of Water = 0.0624 kcf

Layer No.	Layer Elevations		σ_v at top (ksf)	Δz (ft)	Total γ (kcf)	σ_v at bottom (ksf)	σ_v at midpoint (ksf)	z_{water} (ft)	μ at midpoint (ksf)	σ'_{vo} at midpoint (ksf)	N (bpf)	N_{60} (bpf)	C_N	$N1_{60}$ (bpf)
	Top (ft)	Bottom (ft)												
1	175.20	156.90	0.144	18.30	0.115	2.249	1.196	10.35	0.646	0.55	6	8	1.43	11
2	156.90	149.40	2.249	7.50	0.115	3.111	2.680	23.25	1.451	1.229	7	9	1.16	10
3	149.40	144.40	3.111	5.00	0.120	3.711	3.411	29.50	1.841	1.57	15	20	1.08	22
4	144.40	139.40	3.711	5.00	0.125	4.336	4.024	34.50	2.153	1.871	30	40	1.02	41
5	139.40	131.90	4.336	7.50	0.130	5.311	4.824	40.75	2.543	2.281	100	133	0.96	128
6	131.90	131.90	5.311		0.130	5.311	5.311	44.50	2.777	2.534	N/A		2	N/A
7														
8														
TIP	131.90	123.90	5.311	8.00	0.130	6.351	5.831	48.50	3.026	2.805	N/A		2	N/A

Selecting Design Properties for Hard Rock

1. q_u values for rock should be based on AASHTO Table 10.4.6.4-1 (which uses Point Load Index Testing) or actual values from Uniaxial Compressive Strength Testing. If neither of these options is available, the NCDOT Rock Core Database may be used to estimate compressive strength.

2. E_i and v values for rock should be based on AASHTO Tables C10.4.6.5-1, and 2 if lab test data is not available

Unconfined Compressive Strength from Point Load Strength Index for Hard Rock AASHTO Table C10.4.6.4-1

Parameter			Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0

Summary of Elastic Moduli for Intact Rock, E_i (modified by Kulhawy, 1978) AASHTO Table C10.4.6.5-1

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (ksi $\times 10^3$)			Standard Deviation (ksi $\times 10^3$)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Summary of Poisson's Ratio for Intact Rock, v (modified by Kulhawy, 1978) AASHTO Table C10.4.6.5-2

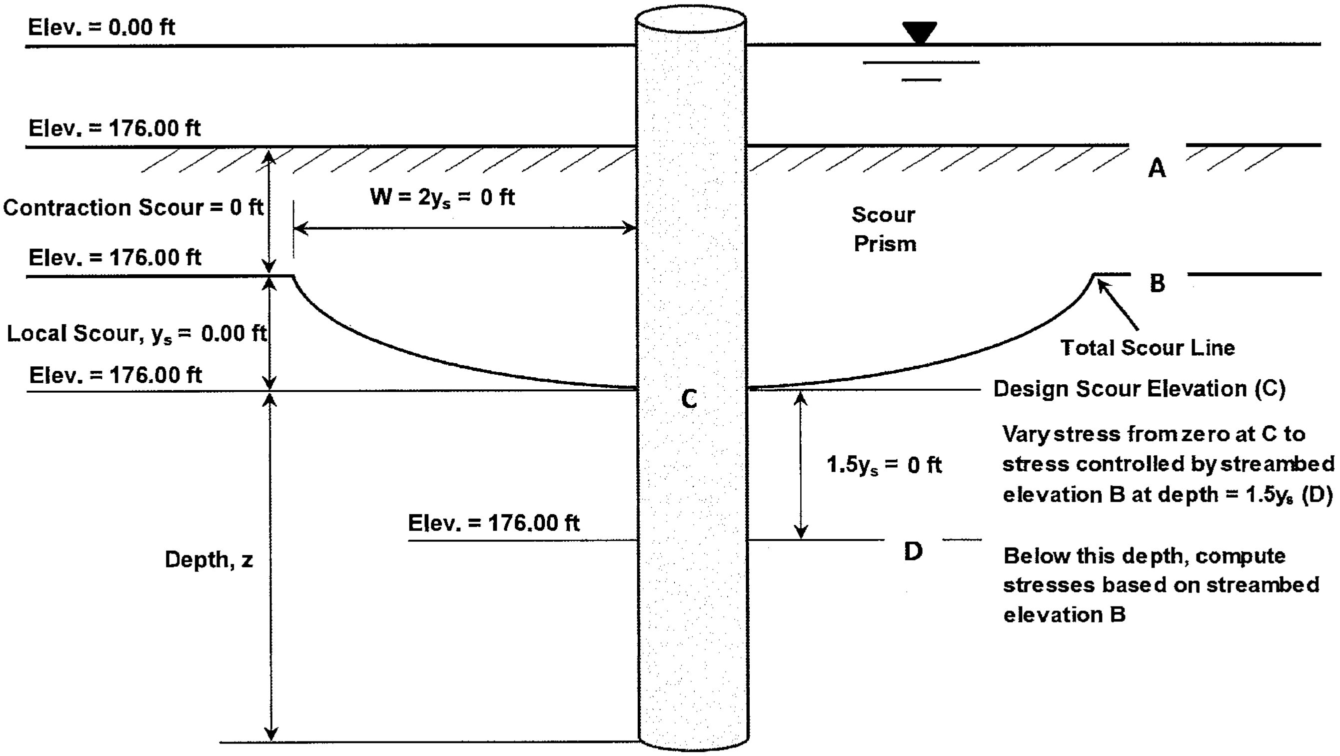
Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, v			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

Calculating Design Stresses for Drilled Piers based on Scour Prism used in FHWA GEC 010

For analysis purposes, lower ground line to the contraction scour elevation (CSE) to account for contraction scour reported in the bridge survey report.

- If the CSE is lower than or equal to the design scour elevation (DSE), consider all scour as contraction scour and lower t ground line to the design scour elevation (DSE).
- If the CSE is higher than the DSE, consider the difference between the CSE and the DSE as local scour.

Groundwater Elevation = 0.00 ft
Original Pre-Scour Streambed Elevation (Point A) = 176.00 ft *= Natural Ground / Finished Grade Elevation*
Amount of Contraction Scour = 0.00 ft
Streambed Elevation after General Scour (Point B) = 176.00 ft *= Point A - Contraction Scour ≥ Design Scour Elevation*
Amount of Local Scour (y_s) = 0.00 ft
p of the embedded length of the drilled pier (Point C) = 176.00 ft *= Design Scour Elevation*
1.5(y_s) = 0.00 ft
evation corresponding to a depth of 1.5(y_s), (Point D) = 176.00 ft *= Point C - 1.5y_s*



Adapted from FHWA GEC 010 Figure 13.18: Illustration of Scour Prism and Effects on Drilled Pier

Per FHWA GEC 010 page 13-46, vertical stress along any depth of the drilled pier can be estimated as follows;

1) At the top of the embedded drilled pier (Point C) the vertical stress is equal to zero.
2) At a depth of 1.5ys (Point D) or greater, assume the vertical stress is controlled by the streambed elevation (Point B).
3) Assume a linear variation in vertical stress from 0 at Point C to the vertical stress value controlled by the streambed at Point

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Soil Layer Profile and Effective Vertical Stress controlled by the streambed elevation (Point B)

- Assume the streambed elevation is equal to the contraction scour elevation (Elevation 176.00 ft).

Layer No.	Top (ft)	Midpoint (ft)	Bottom (ft)	σ_{v_top} (ksf)	$\mu_{\_top}$ (ksf)	σ'_{v_top} (ksf)	Δz (ft)	γ (kcf)	σ_{v_bottom} (ksf)	$\mu_{\_bottom}$ (ksf)	σ'_{v_bottom} (ksf)
0	176.00	175.60	175.20	0.000	0.000	0.000	0.80	0.120	0.096	0.000	0.096
1	175.20	166.05	156.90	0.096	0.000	0.096	18.30	0.115	2.201	0.000	2.201
2	156.90	153.15	149.40	2.201	0.000	2.201	7.50	0.115	3.063	0.000	3.063
3	149.40	146.90	144.40	3.063	0.000	3.063	5.00	0.120	3.663	0.000	3.663
4	144.40	141.90	139.40	3.663	0.000	3.663	5.00	0.125	4.288	0.000	4.288
5	139.40	135.65	131.90	4.288	0.000	4.288	7.50	0.130	5.263	0.000	5.263
6	131.90	131.90	131.90	5.263	0.000	5.263		0.130	5.263	0.000	5.263
7											
8											

Variation in Vertical Stress from Point C to Point D

- Assume the top of the embedded drilled pier is equal to the design scour elevation.
- Vertical stress at elevation 176 ft (Point C) = 0 ksf
- Assume a linear variation in vertical stress from 0 ksf at elevation 176.00 ft (Point C) to a stress value controlled by the elevation 176.00 ft (Point B) at the depth Point D, elevation 176.00 ft.
- PointD lies within Soil Layer No.0

Point D Elevation (ft)	Top of Layer 0 (ft)	σ_v at 176.00 ft	Depth below Layer 0 (ft)	γ for Layer 2	μ at Point D (ksf)	σ'_v at Point D (ksf)
176.00	176.00	0.000	0.00	0.120	0.000	0.000

Point	Elevation (ft)	z (ft)	σ'_v (ksf)	Equation for linear variation over a depth of $1.5y_s$
C	176.00	0.00	0.000	σ'_v (for z = 0 to 22.5 ft) = (0.0000)z
D	176.00	0.00	0.000	

- All stress calculations below elevation 176.00 ft (Point D) will be based on elevation 176.00 ft (Point B).

Summary of Design Stress at the Midpoint of each Soil Layer and at Tip of Drilled Pier

Layer	Top (ft)	Bottom (ft)	Midpoint (ft)	z (ft)	$\sigma_{v\_midpoint}$ (ksf)	μ (ksf)	$\sigma'_{v\_midpoint}$ (ksf)
1	175.20	156.90	166.05	9.95	1.148	0.000	1.148
2	156.90	149.40	153.15	22.85	2.632	0.000	2.632
3	149.40	144.40	146.90	29.10	3.363	0.000	3.363
4	144.40	139.40	141.90	34.10	3.976	0.000	3.976
5	139.40	131.90	135.65	40.35	4.776	0.000	4.776
6	131.90	131.90	131.90	44.10	5.263	0.000	5.263

Tip Elev. (ft)	z (ft)	σ_{v_bottom} (ksf)	μ (ksf)	σ'_{v_bottom} (ksf)
131.90	44.10	5.263	0.000	5.263

Side Resistance in Cohesive Soil (Clays with $S_U \le 5$ ksf)

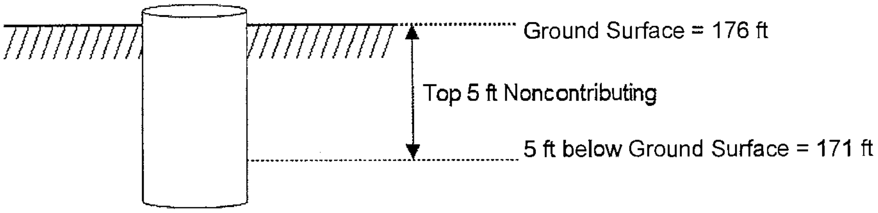
$R_s = (q_s)(A_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for soil layer (ksf)
$= (\alpha)(S_U)$ *AASHTO Eqn. 10.8.3.5.1b-1*

α = adhesion factor
= 0 between the ground surface and a depth of 5 ft
$= 0.55$ for $(S_U/p_a) \le 1.5$ *AASHTO Eqn. 10.8.3.5.1b-2*
$= 0.55 - 0.1(S_U/p_a - 1.5)$ for $1.5 \le (S_U/p_a) \le 2.5$ *AASHTO Eqn. 10.8.3.5.1b-3*

S_U = undrained shear strength (ksf)
$= 100(N1_{60})/1000$ *NCDOT Empirical Formula*

ρ_a = atmospheric pressure (2.12 ksf)



Based on AASHTO Figure 10.8.3.5.1b-1

A_s = area of drilled pier side resistance (ft^2)
$= (\pi)(B)(\Delta z)$

B = diameter of drilled pier (4 ft)
Δz = effective thickness of the soil layer (ft)

Layer No.	Layer Elevations		$N1_{60}$	S_U (ksf)	S_U/ρ_a	α	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
	Top (ft)	Bottom (ft)								
1	175.20	156.90	11	1.100	0.52	0.55	0.605	14.10	177.19	107

Total Side Resistance in Cohesive Soil = 107

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Side Resistance in Cohesionless Soil (Sand / Gravel with $N1_{60} \leq 100$)

$R_s = (q_s)(A_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for soil layer (ksf)
$= (\beta)(\sigma'_v)$ *AASHTO Eqn. 10.8.3.5.2b-1*

β = load transfer coefficient

$$= (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f$$ *AASHTO Eqn. 10.8.3.5.2b-2*

ϕ'_f = effective friction angle
$= 27.5 + 9.2\log(N1_{60}),\quad N1_{60} \leq 100$ *AASHTO Eqn. 10.8.3.5.2b-3*

$N1_{60}$ = SPT - N value corrected for hammer efficiency and overburden (limited to 100 bpf)

σ'_p = effective vertical preconsolidation stress

For Sands: $\frac{\sigma'_p}{\rho_a} \approx 0.47(N_{60})^m$ *AASHTO Eqn. 10.8.3.5.2b-4*

For Gravels: $\frac{\sigma'_p}{\rho_a} = 0.15(N_{60})$ *AASHTO Eqn. 10.8.3.5.2b-5*

m = 0.6 for clean sands; 0.8 for silty sands and sandy silts
N_{60} = SPT - N value corrected for hammer efficiency (limited to 100 bpf)
ρ_a = atmospheric pressure (2.12 ksf)

σ'_v = effective vertical stress at soil layer mid-depth as defined in FHWA GEC 010 pages 13-46

A_s = area of drilled pier side resistance (ft^2)
$= (\pi)(B)(\Delta z)$

B = diameter of drilled pier (4 ft)
Δz = effective thickness of the soil layer (ft)

Layer No.	Layer Elevations		Material Type	$N1_{60}$	ϕ' (deg)	m	N_{60}	σ'_p/ρ_a	σ'_v (ksf)	β	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
	Top (ft)	Bottom (ft)												
2	156.90	149.40	Sand	10	37	0.6	9	1.760	2.632	0.370	0.974	7.50	94.25	92
3	149.40	144.40	Gravel	22	40	0.6	20	3.000	3.363	0.451	1.517	5.00	62.83	95
4	144.40	139.40	Sand	41	42	0.8	40	8.990	3.976	0.850	3.379	5.00	62.83	212

Total Side Resistance in Cohesionless Soil = **399**

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Side Resistance in Weathered and Hard Rock

$R_s = (A_s)(q_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for weathered or hard rock layer (ksf)

For weathered rock layers or hard rock layers with a GSI < 30
= 8 ksf *NCDOT Policy*

For drilled piers socketed into hard rock

$$= \left(C \sqrt{\frac{q_u}{p_a}} \right) p_a$$ *AASHTO Eqn. 10.8.3.5.4b-1*

C = regression coefficent taken as 1.0 for normal rock sockets (see AASHTO C10.8.3.5.4b-1 for details)

For fractured rock that caves and cannot be drilled without artificial support

$$= \left(0.65\alpha_E \sqrt{\frac{q_u}{p_a}} \right) p_a$$ *AASHTO Eqn. 10.8.3.5.4b-2*

a_E = reduction factor to account for jointing in rock (from AASHTO Table 10.8.3.5.4b-1)

RQD (%)	Joint Modificaiton Factor, α_E	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

q_u = Uniaxial Compressive Strength of Intact Rock (ksf) $\leq f'_c$
f'_c = 28 day Compressive Strength of Concrete (4.5 ksi = 648 ksf)
p_a = atmospheric pressure (2.12 ksf)

A_s = area of drilled pier side resistance (ft^2)
= $(\pi)(B)(\Delta z)$

B = diameter of drilled pier (subtract 2 inches to account for possible reduction of drilled pier in rock)
= (48 inches - 2 inches) / 12 inches per ft = 3.83 ft

Δz = effective thickness of the soil layer (ft)

Layer No.	Rock Type	Layer Elevations		AASHTO Equation and Rock Joint Condition to use	RQD (%)	α_E	q_u (ksf)	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
		Top (ft)	Bottom (ft)								
5	Weathered Rock	139.40	131.90	N/A	N/A	N/A	N/A	8.000	7.50	90.32	723

Total Side Resistance in Weathered and Hard Rock = 723

Tip Resistance in Hard Rock

$R_p = (q_p)(A_p)$ *AASHTO Eqn. 10.8.3.5-2*

q_p = unit tip resistance (ksf)

If rock to a depth of 2B below drilled pier tip is intact or tightly jointed and the depth of socket > 1.5 D

$= 2.5q_u$ *AASHTO Eqn. 10.8.3.5.4c-1*

If the rock to a depth of 2D below the drilled pier tip is jointed with random orientation

$$= A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a$$ *AASHTO Eqn. 10.8.3.5.4c-2*

q_u = Uniaxial Compressive Strength of Intact Rock (ksf)

σ'_{vb} = vertical effective stress at the socket bearing elevation

$$A = \sigma'_{vb} + q_u \left[m_b \left(\frac{\sigma'_{vb}}{q_u} \right) + s \right]^a$$ *AASHTO Eqn. 10.8.3.5.4c-3*

$$s = exp\left(\frac{GSI - 100}{9} \right)$$ *AASHTO Eqn. 10.4.6.4-2*

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right)$$ *AASHTO Eqn. 10.4.6.4-3*

$$m_b = exp\left(\frac{GSI - 100}{28} \right) m_i$$ *AASHTO Eqn. 10.4.6.4-4*

m_i = constant for intact rock *AASHTO Table 10.4.6.4-1*

GSI = Global Strength Index

***Hard Rock Layers with an GSI less than 30 will be modeled as weathered rock.**

A_p = area of drilled pier tip resistance (ft^2)
$= (\pi)(B^2)/4$

B = diameter of drilled pier - 2 inches to account for possible reduction for drilled pier in rock (B = 3.83 ft)

Tip Elevation (ft)	AASHTO Equation used to calculate q_u	q_u (ksf)	GSI	m	m_b	s	a	A	q_p (ksf)	A_p (ft^2)	R_p (kips)
131.90	10.8.3.5.4c-2	753	35	13	1.2757	0.00073	0.51595	73.944	333	11.54	3,843

Tip Resistance in Hard Rock (continued)

Table 10.4.6.4-1—Values of the Constant mi by Rock Group

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Summary of Nominal and Factored Side Resistance

Cohesionless IGM	Nominal Side Resistance (kips)	Resistance Factor from AASHTO Table 10.5.5.2.4-1	Factored Side Resistance (kips)	Percentage of Side Resistance produced by Material Type
Cohesive Soil	107	0.45	48	8.7%
Cohesionless Soil	399	0.55	219	32.5%
CohesiveIGM	0	0.60	0	0.0%
Weathered Rock	723	0.60	434	58.8%
Hard Rock	0	0.55	0	0.0%
Total	**1,229**		**701**	**100%**

Note: *Side resistance in soil and weathered rock develops at a much greater displacement than hard rock. If the pier does not have a true rock socket, the side resistance from the hard rock will be ignored and nominal side resistance will be based on the total side resistance in soil and weathered rock.*

Total Nominal Side Resistance = 1,229 kips
Total Factored Side Resistance = 701 kips

Summary of Total Nominal and Factored Tip Resistance

Total Nominal Tip Resistance =	3,843	kips	*the drilled pier is bearing on Hard Rock*
Tip Resistance Factor =	0.50		*for Hard Rock, see AASHTO Table 10.5.5.2.4-1.*
Total Factored Tip Resistance =	1,922	kips	

Required Factored Resistance

$R_{req} = P_r + \gamma_{DC}(W_{Column} + W_{Pier}) - \gamma_{WA}W_{Water} - \gamma_{DC}W_{Soil/Rock} \geq P_r$ *Required Factored Resistance*

P_r = 1,376 kips *Maximum Factored Axial Load Reported by Structure Design*
γ_{DC} = 1.25 *Factor for Permanent Dead Loads, from AASHTO Table 3.4.1-2*
γ_{WA} = 1.00 *Factor for Water Loads, from AASHTO Table 3.4.1-1*

$W_{Column} = (A_{Column})(L_{Column})(\gamma_c)$ *Unfactored Weight of Column*

A_{Column} = 9.62 ft^2 *Area of Column*
L_{Column} = 14 ft *Length of Column*
γ_c = 0.150 kcf *Unit Weight of Concrete*

= 20 kips

$W_{Pier} = (A_{Pier})(L_{Pier})(\gamma_c)$ *Unfactored Weight of Drilled Pier*

A_{Pier} = 12.57 ft^2 *Area of Drilled Pier*
L_{Pier} = 43.3 ft *Length of Drilled Pier*
γ_c = 0.150 kcf *Unit Weight of Concrete*

= 82 kips

$W_{Water} = (A_{Pier})(z_w)(\gamma_w)$ *Unfactored Weight of Water Displaced by Drilled Pier*

A_{Pier} = 12.57 ft^2 *Area of Drilled Pier*
z_w = 0 ft *Depth from water surface to the drilled pier tip*
γ_w = 0.0624 kcf *Unit Weight of Water*

= 0 kips

$W_{Soil/Rock} = (A_{Pier})(\sigma'_{vo})$ *Unfactored Effective Weight of Soil / Rock that will be displaced*

A_{Pier} = 12.57 ft^2 *Area of Drilled Pier*
σ'_{vo} = 5.263 ksf *ffective vertical stress at drilled pier tip as defined in FHWA GEC 010 pages 13-46*

$W_{Soil/Rock}$ = 66 kips

R_{req} = 1,376 kips + 1.25(20 kips + 82 kips) -1.00(0 kips) -1.25(66 kips) = 1,421 kips => 710.5T round up to 715T req'd factored resistance

Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket

For Load Transfer of a drilled pier that is bearing on hard rock with no rock socket, the total displacement of the pier will be controlled by the rock layer below the base of the pier. The total displacement, (w_c), will be calculated using FHWA GEC 10 Equation D-17 and assumes the entire load is carried by the tip. Use the normalized load transfer values along with the total factored side resistance to calculate the factored side resistance developed in the soil and weathered rock layers at this displacement. The remaining factored resistance that is carried by the drilled pier tip must be less than or equal to the total factored tip resistance.

Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket (continued)

Calculate the total displacement, w_c, at the drilled pier tip (assume entire load is carried by the tip)

$$w_c = F_3\left(\frac{Q_c}{\pi E_r B}\right) - F_4 B$$ *FHWA-NHI-10-016 Eqn. D-17*

$Q_c = R_{req}$ = 1,421 kips *Required Factored Resistance*

B = 48 inches *Diameter of Drilled Pier*

L = 0.01 inches *Length of Pier in Hard Rock (use 0.01 inch when assuming entire load carried by tip)*

E_c = 3,824 ksi *Elastic Modulus of Concrete*

v_c = 0.25 *Poisson's Ration for Concrete*

E_{i_r} = 830 ksi *Elastic Modulus of Intact Rock around Drilled Pier Tip*
(Assume $E_{i\_r} = E_{i\_b}$ if the tip of the drilled pier is sitting on top of the hard rock layer)

E_r ($\le E_{i\_r}$) = 611 ksi *Elastic Modulus of Rock Mass around Drilled Pier Tip (AASHTO Eqn. 10.4.6.5-1)*
(Assume $E_r = E_b$ if the tip of the drilled pier is sitting on top of the hard rock layer)

v_r = 0.20 *Poisson's Ration of Rock Mass around Drilled Pier Tip*
(Assume $v_r = v_b$ if the tip of the drilled pier is sitting on top of the hard rock layer)

E_{i_b} = 830 ksi *Elastic Modulus of Intact Rock below Drilled Pier Tip*

E_b ($\le E_{i\_b}$) = 367 ksi *Elastic Modulus of Rock Mass below Drilled Pier Tip (FHWA-NHI-10-016 Table 3-9)*

v_b = 0.20 *Poisson's Ration of Rock Mass below Drilled Pier Tip*

ζ = 0.01 $= ln[5(1 - v_b)L/B]$ *(must be > 0) FHWA-NHI-10-016 Eqn. D-11*

a_1 = 22.10880939 $= (1 + v_r)\zeta + a_2$ *FHWA-NHI-10-016 Eqn. D-25*

a_2 = 22.09680939 $= \left[(1 - v_c)\left(\frac{E_r}{E_C}\right) + (1 + v_r)\right]\left(\frac{1}{2\tan\phi\tan\psi}\right)$ *FHWA-NHI-10-016 Eqn. D-26*

a_3 = 0.228287198 $= \left(\frac{v_c}{2\tan\psi}\right)\left(\frac{E_r}{E_c}\right)$ *FHWA-NHI-10-016 Eqn. D-27*

β = 68.5803 $= a_3\left(\frac{E_c}{E_r}\right)B$ *FHWA-NHI-10-016 Eqn. D-24*

α = 79701.13612 $= a_1\left(\frac{E_c}{E_r}\right)\left(\frac{B^2}{4}\right)$ *FHWA-NHI-10-016 Eqn. D-23*

λ_1 = 0.003137955 $= \frac{-\beta + (\beta^2 + 4\alpha)^{1/2}}{2\alpha}$ *FHWA-NHI-10-016 Eqn. D-22*

λ_2 = -0.003998423 $= \frac{-\beta - (\beta^2 + 4\alpha)^{1/2}}{2\alpha}$ *FHWA-NHI-10-016 Eqn. D-22*

D_3 = 1.690629196 $= \left[\pi(1 - v_b{}^2)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_2 B\right] exp[\lambda_2 D]$ *FHWA-NHI-10-016 Eqn. D-21*

D_4 = 9.264274904 $= \left[\pi(1 - v_b{}^2)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_1 B\right] exp[\lambda_1 D]$ *FHWA-NHI-10-016 Eqn. D-21*

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Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket (continued)

$C_3 = 0.223225282 \quad = \dfrac{D_3}{D_4 - D_3}$ *FHWA-NHI-10-016 Eqn. D-20*

$C_4 = 1.223225282 \quad = \dfrac{D_4}{D_4 - D_3}$ *FHWA-NHI-10-016 Eqn. D-20*

$F_3 = 5.020618276 \quad = a_1(\lambda_1 B C_3 - \lambda_2 B C_4) - 4a_3$ *FHWA-NHI-10-016 Eqn. D-18*

$F_4 = 7.52219E\text{-}08 \quad = \left[1 - a_1\left(\dfrac{\lambda_1 - \lambda_2}{D_4 - D_3}\right)B\right]a_2\left(\dfrac{c}{E_r}\right)$ *FHWA-NHI-10-016 Eqn. D-19*

w_c = 0.08 inches *Total Displacement at Drilled Pier Tip*

Calculate the developed side resistance in the soil / weathered rock layers at the displacement, w_c

The majority of the side resistance is produced by Weathered Rock , which is treated as a cohesive material for Load transfer. Use AASHTO Figure 10.8.2.2.2.1 to predict the normalized load transfer for side resistance.

Δz / D (%)	Normalized Side Transfer R_{sd} / R_s AASHTO Figure 10.8.2.2.2.1
0.0	0.00
0.3	0.83
0.6	0.95
1.0	0.93
1.3	0.91
1.6	0.88
2.0	0.83
5.0	0.55

Developed Side Resistance					
$\frac{\Delta z}{D}$	D (in)	Δz (in)	$\varphi_{qs}R_s$ (kips)	$\frac{R_{sd}}{R_s}$	$\varphi_{qs}R_{sd}$ (kips)
0.00%	48	0.00	701	0.00	**332**
0.17%	**48**	**0.08**	**701**	**0.47**	
0.30%	48	0.14	701	0.83	

Δz / D = total settlement / drilled pier diameter
R_{sd} / R_s = developed side resistance / total nominal side resistance
$\varphi_{qs}R_s$ = total factored side resistance
$\varphi_{qs}R_{sd}$ = developed factored side resistance
= $(R_{sd}/R_s)(\varphi_{qs}R_s)$

Calculate the remaining resistance that must be carried by the tip (must ≤ the total factored tip resistance)

Required Factored Resistance = 1,421 kips
Developed Factored Side Resistance = 332 kips
Required Factored Tip Resistance = 1,089 kips **≤ 1,922 kips OK**

Required Tip Resistance

q_{req} = required tip resistance (rounded up to the nearest 10 ksf or 5 tsf)

$$= \frac{\dfrac{R_{req} - \varphi_{qs}R_{sd}}{A_T}}{\varphi_{qp}} \le q_p$$

NCDOT policy

R_r = required factored geotechnical resistance (kips)
$\varphi_{qs}R_{sd}$ = factored developed side resistance (kips)
A_T = area of drilled pier tip (ft^2)
φ_{qp} = tip resistance factor
q_p = unit tip resistance (ksf)

R_{req} (kips)	$\varphi_{qs}R_{sd}$ (kips)	A_{Tip} (ft^2)	φ_{qp}	q_p (ksf)	q_{req} (ksf)
1,421	332	11.54	0.50	333	190

= 95 tsf
tip resistance req'd

SINCE 1881

FROEHLING & ROBERTSON, INC.
Engineering • Environmental • Geotechnical

SHEET NO. 1 OF 1

JOB Bridge 188 Bent 1 Lateral

DATE 5/12/17

COMPUTATIONS FOR

BY C. Wang CHKD

Boring B-1A

Depth (ft)		Elev.
0	BOC = ±189.2 per WE2	189.2
14	ϕ=42" Top of pier = ±175.2 "	175.2
32.3	ϕ=48" ① CLAY N_{60}=8 γ=53 c=600 k=100 ε=0.01	156.9
39.8	② SAND N_{60}=9 γ=53 ϕ=29° k=20	149.4
44.8	③ SAND N_{60}=20 γ=58 ϕ=33° k=60	144.4
49.8	④ SAND N_{60}=40 γ=63 ϕ=39° k=125	139.4
57.3	⑤ C.R. model as W.R. due to existing 1.5' void W.R. γ=63 c=5000 k=2000 ε=0.0004 and BHB info	131.9
69.2	⑥ C.R. γ=100 c=1000 psi	120

① Model longitudinal direction as free head.
Shear = 30,200 lbs, Moment = 774,000 lbs·in, Axial = 833,000 lbs

y = 0.74", 1st neg = 32.8 (EL. 156.4), max neg = 39.5 (EL. 149.7)

[illegible])

② Model transversal direction as fixed head.
Shear = 28,700 lbs, slope = 0, Axial = 809,300 lbs
y = 0.18" 1st neg = 33.8 (EL. 155.4), max neg = 40.6 (EL 148.6)
say POF = EL 154.0'
MinTip 1.5B below 1st neg = EL 152.4'
1.0B below max neg = EL 145.7'
MinTip EL = 145.0'

From axial, MinTip = EL 126.0'

∴ POF = EL 154.0'
MinTip = EL 126.0'

Form No. 502
Revised 05/2008

```
================================================================================

                  LPile Plus for Windows, Version 2013-07.001

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

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                              All Rights Reserved

================================================================================

This copy of LPile is licensed to:

Branch User
Froehling & Robertson, Inc.

Serial Number of Security Device:        293783516
Company Name Stored in Security Device:  Froehling & Robertson, Inc.


--------------------------------------------------------------------------------
                            Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:        F:\Projects 66U\66U-0390 (WEI-I-5786 Bridges 108 & 111 Johnston
Co)\NON_CADD\Foundation Recs\Bridge 108 Lizzie Mill\Lateral\
Name of input data file:       B1-A.lp7d
Name of output report file:    B1-A.lp7o
Name of plot output file:      B1-A.lp7p
Name of runtime messeage file: B1-A.lp7r


--------------------------------------------------------------------------------
                           Date and Time of Analysis
--------------------------------------------------------------------------------

               Date:  May 12, 2017     Time:   0:22:55


--------------------------------------------------------------------------------
                                Problem Title
--------------------------------------------------------------------------------

Project Name: Bridge 108 Lizzie Mill


Job Number: 66U-0390


Client: Wetherill Engineering


Engineer: C. Wang


Description: Bent 1 B1-A




--------------------------------------------------------------------------------
                               Program Options
--------------------------------------------------------------------------------

Engineering Units of Input Data and Computations:
 - Engineering units are US Customary Units: pounds, inches, feet

Analysis Control Options:
```

```
- Maximum number of iterations allowed                  =         500
- Deflection tolerance for convergence                  =   1.0000E-05 in
- Maximum allowable deflection                          =     100.0000 in
- Number of pile increments                             =         100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

Computational Options:
 - Use unfactored loads in computations
 - No computation of pile-head foundation stiffness matrix
 - Compute pile response under loading and nonlinear bending properties of pile
   (if nonlinear properties are specified)
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Input Data Options:
- Analysis does not use p-y modification factors (individual pile or shaft only)
- Analysis assumes zero shear resistance at the pile tip
- Analysis assumes no loading by soil movements acting on pile

Output Options:
- No p-y curves to be computed and reported for user-specified depths
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points)  = 1


--------------------------------------------------------------------------------
                    Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Total number of pile sections                           =          2

Total length of pile                                    =      52.00 ft

Depth of ground surface below top of pile               =      14.00 ft

Pile diameter values used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile.

Point         Depth              Pile
                X              Diameter
               ft                 in
-----       ---------        -----------
  1           0.00000         42.0000000
  2          14.000000        42.0000000
  3          14.000000        48.0000000
  4          52.000000        48.0000000


Input Structural Properties:
----------------------------

Pile Section No. 1:

   Section Type                                         =    Elastic Pile
   Cross-sectional Shape                                =        Circular
   Section Length                                       =     14.00000 ft
   Top Width                                            =     42.00000 in
   Bottom Width                                         =     42.00000 in
   Top Area                                             =   1385.44236 Sq. in
   Bottom Area                                          =   1385.44236 Sq. in
   Moment of Inertia at Top                             =      152745. in^4
   Moment of Inertia at Bottom                          =      152745. in^4
   Elastic Modulus                                      =     3122019. lbs/in^2
```

```
Pile Section No. 2:

   Section Type                                    =    Elastic Pile
   Cross-sectional Shape                           =        Circular
   Section Length                                  =     38.00000 ft
   Top Width                                       =     48.00000 in
   Bottom Width                                    =     48.00000 in
   Top Area                                        =   1809.55737 Sq. in
   Bottom Area                                     =   1809.55737 Sq. in
   Moment of Inertia at Top                        =      260576. in^4
   Moment of Inertia at Bottom                     =      260576. in^4
   Elastic Modulus                                 =     3823676. lbs/in^2


-------------------------------------------------------------------------------
                      Ground Slope and Pile Batter Angles
-------------------------------------------------------------------------------

Ground Slope Angle                                 =        0.000 degrees
                                                   =        0.000 radians

Pile Batter Angle                                  =        0.000 degrees
                                                   =        0.000 radians


-------------------------------------------------------------------------------
                       Soil and Rock Layering Information
-------------------------------------------------------------------------------

The soil profile is modelled using 6 layers

Layer 1 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer       =     14.00000 ft
   Distance from top of pile to bottom of layer    =     32.30000 ft
   Effective unit weight at top of layer           =     53.00000 pcf
   Effective unit weight at bottom of layer        =     53.00000 pcf
   Undrained cohesion at top of layer              =    600.00000 psf
   Undrained cohesion at bottom of layer           =    600.00000 psf
   Epsilon-50 at top of layer                      =      0.01000
   Epsilon-50 at bottom of layer                   =      0.01000
   Subgrade k at top of layer                      =    100.00000 pci
   Subgrade k at bottom of layer                   =    100.00000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer       =     32.30000 ft
   Distance from top of pile to bottom of layer    =     39.80000 ft
   Effective unit weight at top of layer           =     53.00000 pcf
   Effective unit weight at bottom of layer        =     53.00000 pcf
   Friction angle at top of layer                  =     29.00000 deg.
   Friction angle at bottom of layer               =     29.00000 deg.
   Subgrade k at top of layer                      =     20.00000 pci
   Subgrade k at bottom of layer                   =     20.00000 pci


Layer 3 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer       =     39.80000 ft
   Distance from top of pile to bottom of layer    =     44.80000 ft
   Effective unit weight at top of layer           =     58.00000 pcf
   Effective unit weight at bottom of layer        =     58.00000 pcf
   Friction angle at top of layer                  =     33.00000 deg.
   Friction angle at bottom of layer               =     33.00000 deg.
   Subgrade k at top of layer                      =     60.00000 pci
   Subgrade k at bottom of layer                   =     60.00000 pci
```

```
Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =     44.80000 ft
   Distance from top of pile to bottom of layer       =     49.80000 ft
   Effective unit weight at top of layer              =     63.00000 pcf
   Effective unit weight at bottom of layer           =     63.00000 pcf
   Friction angle at top of layer                     =     39.00000 deg.
   Friction angle at bottom of layer                  =     39.00000 deg.
   Subgrade k at top of layer                         =    125.00000 pci
   Subgrade k at bottom of layer                      =    125.00000 pci


Layer 5 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer          =     49.80000 ft
   Distance from top of pile to bottom of layer       =     57.30000 ft
   Effective unit weight at top of layer              =     63.00000 pcf
   Effective unit weight at bottom of layer           =     63.00000 pcf
   Undrained cohesion at top of layer                 =   5000.00000 psf
   Undrained cohesion at bottom of layer              =   5000.00000 psf
   Epsilon-50 at top of layer                         =      0.00400
   Epsilon-50 at bottom of layer                      =      0.00400
   Subgrade k at top of layer                         =   2000.00000 pci
   Subgrade k at bottom of layer                      =   2000.00000 pci


Layer 6 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer          =     57.30000 ft
   Distance from top of pile to bottom of layer       =     69.20000 ft
   Effective unit weight at top of layer              =    100.00000 pcf
   Effective unit weight at bottom of layer           =    100.00000 pcf
   Uniaxial compressive strength at top of layer      =   1000.00000 psi
   Uniaxial compressive strength at bottom of layer   =   1000.00000 psi


   (Depth of lowest soil layer extends   17.20 ft below pile tip)



--------------------------------------------------------------------------------
                           Summary of Soil Properties
--------------------------------------------------------------------------------

                          Layer                      Layer      Effective   Undrained     Angle of      Uniaxial
Strain
Layer                 Soil Type                      Depth      Unit Wt.    Cohesion      Friction        qu
Factor        kpy
 Num.         (p-y Curve Criteria)                    ft          pcf         psf           deg.          psi
Epsilon 50     pci
-----   ----------------------------------   ----------   ----------   ----------   ----------   ----------
----------   ----------
  1     Stiff Clay with Free Water            14.000       53.000      600.000         --           --
0.01000      100.000
                                               32.300       53.000      600.000         --           --
0.01000      100.000
  2     Sand (Reese, et al.)                  32.300       53.000        --           29.000         --           --
          20.000
                                               39.800       53.000        --           29.000         --           --
          20.000
  3     Sand (Reese, et al.)                  39.800       58.000        --           33.000         --           --
          60.000
                                               44.800       58.000        --           33.000         --           --
          60.000
  4     Sand (Reese, et al.)                  44.800       63.000        --           39.000         --           --
         125.000
                                               49.800       63.000        --           39.000         --           --
         125.000
  5     Stiff Clay with Free Water            49.800       63.000     5000.000         --           --
```

```
0.00400     2000.000
                                                57.300       63.000     5000.000         --            --
0.00400     2000.000
  6     Vuggy Limestone                         57.300      100.000        --            --         1000.000        --
         --
                                                69.200      100.000        --            --         1000.000        --
         --



--------------------------------------------------------------------------------
                               Loading Type
--------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



--------------------------------------------------------------------------------
               Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 2

Load    Load          Condition               Condition              Axial Thrust          Compute
 No.    Type              1                       2                   Force, lbs     Top y vs. Pile Length
-----   ----    --------------------    -----------------------    ----------------   ---------------------
   1      1     V =       30200. lbs    M =      774000. in-lbs          833000.             No
   2      2     V =       28700. lbs    S =        0.0000 in/in          809300.             No

V = perpendicular shear force applied to pile head
M = bending moment applied to pile head
y = lateral deflection relative to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applie to pile head
Axial thrust is assumed to be acting axially for all pile batter angles.


--------------------------------------------------------------------------------
    Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:
-------------------
Moment-curvature properties were derived from elastic section properties


Pile Section No. 2:
-------------------
Moment-curvature properties were derived from elastic section properties




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

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head                           =      30200.0 lbs
Applied moment at pile head                        =     774000.0 in-lbs
Axial thrust load on pile head                     =     833000.0 lbs
```

```
   Depth     Deflect.    Bending     Shear        Slope       Total     Bending    Soil Res.  Soil Spr.   Distrib.
     X          y        Moment      Force          S         Stress   Stiffness      p         Es*h     Lat. Load
    feet     inches      in-lbs       lbs        radians      psi*      lb-in^2     lb/in      lb/inch    lb/inch
---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
      0.00     0.7384     774000.     30200.  -0.003259   707.6646  4.769E+11      0.000      0.000      0.000
     0.520     0.7181     979361.     30200.  -0.003247   735.8985  4.769E+11      0.000      0.000      0.000
     1.040     0.6979    1184655.     30200.  -0.003233   764.1232  4.769E+11      0.000      0.000      0.000
     1.560     0.6777    1389869.     30200.  -0.003216   792.3368  4.769E+11      0.000      0.000      0.000
     2.080     0.6577    1594988.     30200.  -0.003197   820.5374  4.769E+11      0.000      0.000      0.000
     2.600     0.6378    1799998.     30200.  -0.003175   848.7230  4.769E+11      0.000      0.000      0.000
     3.120     0.6181    2004887.     30200.  -0.003150   876.8919  4.769E+11      0.000      0.000      0.000
     3.640     0.5985    2209638.     30200.  -0.003122   905.0420  4.769E+11      0.000      0.000      0.000
     4.160     0.5791    2414240.     30200.  -0.003092   933.1714  4.769E+11      0.000      0.000      0.000
     4.680     0.5599    2618677.     30200.  -0.003059   961.2783  4.769E+11      0.000      0.000      0.000
     5.200     0.5410    2822937.     30200.  -0.003023   989.3607  4.769E+11      0.000      0.000      0.000
     5.720     0.5222    3027004.     30200.  -0.002985  1017.4167  4.769E+11      0.000      0.000      0.000
     6.240     0.5037    3230865.     30200.  -0.002944  1045.4444  4.769E+11      0.000      0.000      0.000
     6.760     0.4855    3434507.     30200.  -0.002901  1073.4418  4.769E+11      0.000      0.000      0.000
     7.280     0.4675    3637915.     30200.  -0.002854  1101.4072  4.769E+11      0.000      0.000      0.000
     7.800     0.4498    3841075.     30200.  -0.002805  1129.3385  4.769E+11      0.000      0.000      0.000
     8.320     0.4325    4043975.     30200.  -0.002754  1157.2339  4.769E+11      0.000      0.000      0.000
     8.840     0.4155    4246599.     30200.  -0.002700  1185.0915  4.769E+11      0.000      0.000      0.000
     9.360     0.3988    4448934.     30200.  -0.002643  1212.9094  4.769E+11      0.000      0.000      0.000
     9.880     0.3825    4650967.     30200.  -0.002583  1240.6857  4.769E+11      0.000      0.000      0.000
    10.400     0.3666    4852683.     30200.  -0.002521  1268.4185  4.769E+11      0.000      0.000      0.000
    10.920     0.3510    5054070.     30200.  -0.002456  1296.1059  4.769E+11      0.000      0.000      0.000
    11.440     0.3359    5255113.     30200.  -0.002389  1323.7460  4.769E+11      0.000      0.000      0.000
    11.960     0.3212    5455798.     30200.  -0.002319  1351.3370  4.769E+11      0.000      0.000      0.000
    12.480     0.3070    5656112.     30200.  -0.002246  1378.8770  4.769E+11      0.000      0.000      0.000
    13.000     0.2932    5856041.     30200.  -0.002171  1406.3641  4.769E+11      0.000      0.000      0.000
    13.520     0.2799    6055572.     30200.  -0.002093  1433.7965  4.769E+11      0.000      0.000      0.000
    14.040     0.2671    6254692.     30160.  -0.002033  1036.4129  9.964E+11   -12.8198   299.5200      0.000
    14.560     0.2545    6453108.     29641.  -0.001994  1054.6878  9.964E+11  -153.4427  3762.0311      0.000
    15.080     0.2422    6645340.     28584.  -0.001953  1072.3930  9.964E+11  -185.3276  4774.7868      0.000
    15.600     0.2301    6830139.     27335.  -0.001910  1089.4136  9.964E+11  -214.9467  5827.9544      0.000
    16.120     0.2184    7006346.     25909.  -0.001867  1105.6430  9.964E+11  -242.2800  6923.6664      0.000
    16.640     0.2068    7172892.     24319.  -0.001823  1120.9824  9.964E+11  -267.1718  8059.9926      0.000
    17.160     0.1956    7328801.     22587.  -0.001777  1135.3422  9.964E+11  -288.2180  9194.2342      0.000
    17.680     0.1847    7473248.     20734.  -0.001731  1148.6463  9.964E+11  -305.7057     10330.      0.000
    18.200     0.1740    7605549.     18776.  -0.001684  1160.8317  9.964E+11  -321.6724     11535.      0.000
    18.720     0.1637    7725078.     16724.  -0.001636  1171.8407  9.964E+11  -336.1161     12816.      0.000
    19.240     0.1536    7831267.     14586.  -0.001587  1181.6211  9.964E+11  -349.0359     14180.      0.000
    19.760     0.1438    7923611.     12373.  -0.001538  1190.1263  9.964E+11  -360.4314     15636.      0.000
    20.280     0.1344    8001662.     10093.  -0.001488  1197.3151  9.964E+11  -370.3028     17192.      0.000
    20.800     0.1253    8065035.  7755.9916  -0.001437  1203.1519  9.964E+11  -378.6508     18860.      0.000
    21.320     0.1165    8113401.  5371.9161  -0.001387  1207.6066  9.964E+11  -385.4760     20653.      0.000
    21.840     0.1080    8146493.  2949.9995  -0.001336  1210.6545  9.964E+11  -390.7793     22584.      0.000
    22.360     0.0998    8164104.   499.7374  -0.001285  1212.2766  9.964E+11  -394.5611     24671.      0.000
    22.880     0.0919    8166086. -1969.3752  -0.001234  1212.4591  9.964E+11  -396.8211     26933.      0.000
    23.400     0.0844    8152351. -4447.8374  -0.001183  1211.1941  9.964E+11  -397.5578     29394.      0.000
    23.920     0.0772    8122871. -6926.1333  -0.001132  1208.4788  9.964E+11  -396.7678     32079.      0.000
    24.440     0.0703    8077677. -9394.7181  -0.001081  1204.3163  9.964E+11  -394.4453     35024.      0.000
    24.960     0.0637    8016861.    -11844.  -0.001030  1198.7150  9.964E+11  -390.5805     38268.      0.000
    25.480     0.0574    7940576.    -14250.  -0.000980  1191.6889  9.964E+11  -380.4496     41348.      0.000
    26.000     0.0515    7849219.    -16560.  -0.000931  1183.2746  9.964E+11  -360.1522     43678.      0.000
    26.520     0.0458    7743583.    -18744.  -0.000882  1173.5451  9.964E+11  -339.7801     46297.      0.000
    27.040     0.0404    7624465.    -20800.  -0.000834  1162.5739  9.964E+11  -319.3028     49267.      0.000
    27.560     0.0354    7492665.    -22729.  -0.000787  1150.4346  9.964E+11  -298.6790     52669.      0.000
    28.080     0.0306    7348992.    -24527.  -0.000740  1137.2018  9.964E+11  -277.8528     56617.      0.000
    28.600     0.0261    7194260.    -26195.  -0.000695  1122.9505  9.964E+11  -256.7473     61272.      0.000
    29.120     0.0220    7029297.    -27730.  -0.000650  1107.7569  9.964E+11  -235.2549     66870.      0.000
    29.640     0.0180    6854946.    -29130.  -0.000607  1091.6984  9.964E+11  -213.2208     73782.      0.000
    30.160     0.0144    6672068.    -30389.  -0.000564  1074.8548  9.964E+11  -190.4118     82622.      0.000
    30.680     0.0110    6481560.    -31502.  -0.000523  1057.3082  9.964E+11  -166.4556     94515.      0.000
    31.200   0.007852    6284358.    -32461.  -0.000483  1039.1453  9.964E+11  -140.7012    111822.      0.000
    31.720   0.004959    6081474.    -33229.  -0.000444  1020.4590  9.964E+11  -105.4504    132687.      0.000
    32.240   0.002304    5874286.    -33715.  -0.000407  1001.3762  9.964E+11   -50.4374    136581.      0.000
1st 32.760  -0.000121    5664943.    -33872.  -0.000371   982.0949  9.964E+11     0.1774  9159.8204      0.000
neg 33.280  -0.002325    5455422.    -33860.  -0.000336   962.7973  9.964E+11     3.7026  9938.5724      0.000
```

```
        33.800  -0.004315   5245868.    -33825.  -0.000303   943.4966  9.964E+11     7.4116     10717.      0.000
        34.320  -0.006101   5036431.    -33767.  -0.000270   924.2068  9.964E+11    11.2398     11496.      0.000
        34.840  -0.007690   4827268.    -33685.  -0.000239   904.9421  9.964E+11    15.1265     12275.      0.000
        35.360  -0.009090   4618537.    -33578.  -0.000210   885.7173  9.964E+11    19.0152     13054.      0.000
        35.880    -0.0103   4410397.    -33447.  -0.000182   866.5468  9.964E+11    22.8532     13832.      0.000
        36.400    -0.0114   4203002.    -33293.  -0.000155   847.4450  9.964E+11    26.5920     14611.      0.000
        36.920    -0.0122   3996506.    -33116.  -0.000129   828.4260  9.964E+11    30.1872     15390.      0.000
        37.440    -0.0130   3791055.    -32917.  -0.000105   809.5032  9.964E+11    33.5981     16169.      0.000
        37.960    -0.0135   3586790.    -32697. -8.151E-05   790.6896  9.964E+11    36.7880     16947.      0.000
        38.480    -0.0140   3383839.    -32459. -5.968E-05   771.9972  9.964E+11    39.7242     17726.      0.000
        39.000    -0.0143   3182326.    -32203. -3.912E-05   753.4371  9.964E+11    42.3777     18505.      0.000
max     39.520    -0.0145   2982359.    -31931. -1.981E-05   735.0194  9.964E+11    44.7233     19284.      0.000
nep     40.040    -0.0145   2784036.    -31420. -1.758E-06   716.7532  9.964E+11   118.9178     51044.      0.000
        40.560    -0.0145   2590253.    -30662.  1.507E-05   698.9051  9.964E+11   123.9891     53380.      0.000
        41.080    -0.0143   2401214.    -29876.  3.070E-05   681.4939  9.964E+11   128.1240     55716.      0.000
        41.600    -0.0141   2217085.    -29066.  4.516E-05   664.5350  9.964E+11   131.2776     58053.      0.000
        42.120    -0.0138   2037995.    -28241.  5.849E-05   648.0402  9.964E+11   133.4141     60389.      0.000
        42.640    -0.0134   1864035.    -27405.  7.071E-05   632.0178  9.964E+11   134.5065     62725.      0.000
        43.160    -0.0129   1695250.    -26565.  8.185E-05   616.4722  9.964E+11   134.5363     65061.      0.000
        43.680    -0.0124   1531650.    -25729.  9.196E-05   601.4039  9.964E+11   133.4929     67398.      0.000
        44.200    -0.0118   1373197.    -24903.   0.000101   586.8099  9.964E+11   131.3733     69734.      0.000
        44.720    -0.0111   1219815.    -24093.   0.000109   572.6828  9.964E+11   128.1819     72070.      0.000
        45.240    -0.0104   1071384.    -23054.   0.000116   559.0118  9.964E+11   204.8472    122989.      0.000
        45.760  -0.009646    930894.    -21798.   0.000123   546.0723  9.964E+11   197.6493    127856.      0.000
        46.280  -0.008863    798071.    -20593.   0.000128   533.8387  9.964E+11   188.5124    132723.      0.000
        46.800  -0.008048    672561.    -19451.   0.000133   522.2789  9.964E+11   177.4658    137590.      0.000
        47.320  -0.007208    553940.    -18384.   0.000136   511.3534  9.964E+11   164.5485    142458.      0.000
        47.840  -0.006345    441708.    -17403.   0.000140   501.0164  9.964E+11   149.8085    147325.      0.000
        48.360  -0.005465    335295.    -16520.   0.000142   491.2154  9.964E+11   133.3020    152192.      0.000
        48.880  -0.004573    234061.    -15745.   0.000144   481.8914  9.964E+11   115.0934    157059.      0.000
        49.400  -0.003671    137301.    -15089.   0.000145   472.9795  9.964E+11    95.2547    161926.      0.000
        49.920  -0.002763     44245.    -11360.   0.000146   464.4087  9.964E+11  1099.7327   2483274.      0.000
        50.440  -0.001854 -5990.6580 -5118.5383   0.000146   460.8853  9.964E+11   900.8789   3031473.      0.000
        50.960  -0.000946    -21148.  -300.6499   0.000146   462.2814  9.964E+11   643.3161   4245411.      0.000
min     51.480 -3.757E-05    -11256.  1815.7602   0.000145   461.3703  9.964E+11    35.0204   5815978.      0.000
        52.000   0.000870      0.000      0.000   0.000145   460.3336  9.964E+11  -616.9949   2212739.      0.000
Tip
     * The above values of total stress are combined axial and bending stresses.


    Output Summary for Load Case No. 1:

    Pile-head deflection            =      0.7383815 inches
    Computed slope at pile head     =     -0.0032588 radians
    Maximum bending moment          =       8166086. inch-lbs
    Maximum shear force             =        -33872. lbs
    Depth of maximum bending moment =     22.8800000 feet below pile head
    Depth of maximum shear force    =     32.7600000 feet below pile head
    Number of iterations            =             13
    Number of zero deflection points =             2



    ------------------------------------------------------------------------------
                      Computed Values of Pile Loading and Deflection
                        for Lateral Loading for Load Case Number 2
    ------------------------------------------------------------------------------

    Pile-head conditions are Shear and Pile-head Rotation (Loading Type 2)

    Shear force at pile head                        =       28700.0 lbs
    Rotation of pile head                           =     0.000E+00 radians
    Axial load at pile head                         =      809300.0 lbs

    (Zero slope for this load indicates fixed-head conditions)

       Depth    Deflect.    Bending     Shear      Slope      Total    Bending   Soil Res.  Soil Spr.   Distrib.
         X         y        Moment      Force        S       Stress   Stiffness      p         Es*h     Lat. Load
        feet     inches     in-lbs       lbs      radians     psi*     lb-in^2     lb/in     lb/inch    lb/inch
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      0.00     0.1825  -4345842.     28700.      0.000  1181.6295  4.769E+11      0.000      0.000      0.000
     0.520     0.1823  -4166611.     28700. -5.569E-05  1156.9880  4.769E+11      0.000      0.000      0.000
     1.040     0.1818  -3987104.     28700.  -0.000109  1132.3086  4.769E+11      0.000      0.000      0.000
     1.560     0.1809  -3807333.     28700.  -0.000160  1107.5931  4.769E+11      0.000      0.000      0.000
     2.080     0.1798  -3627311.     28700.  -0.000209  1082.8429  4.769E+11      0.000      0.000      0.000
     2.600     0.1783  -3447050.     28700.  -0.000255  1058.0598  4.769E+11      0.000      0.000      0.000
     3.120     0.1766  -3266560.     28700.  -0.000299  1033.2454  4.769E+11      0.000      0.000      0.000
     3.640     0.1746  -3085855.     28700.  -0.000340  1008.4013  4.769E+11      0.000      0.000      0.000
     4.160     0.1724  -2904946.     28700.  -0.000380   983.5292  4.769E+11      0.000      0.000      0.000
     4.680     0.1699  -2723845.     28700.  -0.000416   958.6307  4.769E+11      0.000      0.000      0.000
     5.200     0.1672  -2542563.     28700.  -0.000451   933.7074  4.769E+11      0.000      0.000      0.000
     5.720     0.1642  -2361114.     28700.  -0.000483   908.7610  4.769E+11      0.000      0.000      0.000
     6.240     0.1611  -2179509.     28700.  -0.000513   883.7932  4.769E+11      0.000      0.000      0.000
     6.760     0.1578  -1997760.     28700.  -0.000540   858.8056  4.769E+11      0.000      0.000      0.000
     7.280     0.1544  -1815878.     28700.  -0.000565   833.7998  4.769E+11      0.000      0.000      0.000
     7.800     0.1508  -1633877.     28700.  -0.000588   808.7776  4.769E+11      0.000      0.000      0.000
     8.320     0.1471  -1451768.     28700.  -0.000608   783.7405  4.769E+11      0.000      0.000      0.000
     8.840     0.1432  -1269563.     28700.  -0.000626   758.6902  4.769E+11      0.000      0.000      0.000
     9.360     0.1393  -1087274.     28700.  -0.000641   733.6283  4.769E+11      0.000      0.000      0.000
     9.880     0.1352   -904913.     28700.  -0.000654   708.5566  4.769E+11      0.000      0.000      0.000
    10.400     0.1311   -722492.     28700.  -0.000665   683.4767  4.769E+11      0.000      0.000      0.000
    10.920     0.1269   -540024.     28700.  -0.000673   658.3902  4.769E+11      0.000      0.000      0.000
    11.440     0.1227   -357519.     28700.  -0.000679   633.2988  4.769E+11      0.000      0.000      0.000
    11.960     0.1184   -174992.     28700.  -0.000682   608.2041  4.769E+11      0.000      0.000      0.000
    12.480     0.1142  7547.6229     28700.  -0.000683   585.1832  4.769E+11      0.000      0.000      0.000
    13.000     0.1099    190086.     28700.  -0.000682   610.2794  4.769E+11      0.000      0.000      0.000
    13.520     0.1057    372613.     28700.  -0.000678   635.3739  4.769E+11      0.000      0.000      0.000
    14.040     0.1015    555114.     28685.  -0.000674   498.3644  9.964E+11    -4.8697   299.5200      0.000
    14.560     0.0973    737409.     28466.  -0.000670   515.1544  9.964E+11   -65.3558  4193.2800      0.000
    15.080     0.0931    917135.     27885.  -0.000665   531.7079  9.964E+11  -120.6423  8087.0400      0.000
    15.600     0.0890   1092135.     27069.  -0.000659   547.8260  9.964E+11  -140.9194  9885.0180      0.000
    16.120     0.0849   1261613.     26146.  -0.000651   563.4355  9.964E+11  -155.0442     11400.      0.000
    16.640     0.0808   1425014.     25137.  -0.000643   578.4853  9.964E+11  -168.2917     12992.      0.000
    17.160     0.0768   1581818.     24048.  -0.000634   592.9275  9.964E+11  -180.6505     14669.      0.000
    17.680     0.0729   1731537.     22885.  -0.000623   606.7172  9.964E+11  -192.1105     16439.      0.000
    18.200     0.0691   1873721.     21654.  -0.000612   619.8128  9.964E+11  -202.6628     18310.      0.000
    18.720     0.0653   2007955.     20359.  -0.000600   632.1762  9.964E+11  -212.2999     20292.      0.000
    19.240     0.0616   2133858.     19007.  -0.000587   643.7724  9.964E+11  -221.0153     22395.      0.000
    19.760     0.0580   2251089.     17604.  -0.000573   654.5697  9.964E+11  -228.8038     24632.      0.000
    20.280     0.0544   2359339.     16155.  -0.000559   664.5400  9.964E+11  -235.6610     27016.      0.000
    20.800     0.0510   2458338.     14665.  -0.000543   673.6582  9.964E+11  -241.5838     29563.      0.000
    21.320     0.0476   2547853.     13142.  -0.000528   681.9028  9.964E+11  -246.5700     32290.      0.000
    21.840     0.0444   2627687.     11591.  -0.000512   689.2558  9.964E+11  -250.6180     35217.      0.000
    22.360     0.0413   2697679.     10018.  -0.000495   695.7023  9.964E+11  -253.7271     38368.      0.000
    22.880     0.0382   2757706.  8427.6503  -0.000478   701.2310  9.964E+11  -255.8968     41768.      0.000
    23.400     0.0353   2807682.  6827.0153  -0.000460   705.8340  9.964E+11  -257.1272     45450.      0.000
    23.920     0.0325   2847557.  5221.6339  -0.000443   709.5066  9.964E+11  -257.4181     49449.      0.000
    24.440     0.0298   2877319.  3617.3701  -0.000425   712.2478  9.964E+11  -256.7690     53807.      0.000
    24.960     0.0272   2896992.  2020.0934  -0.000407   714.0597  9.964E+11  -255.1786     58577.      0.000
    25.480     0.0247   2906638.   445.3243  -0.000389   714.9481  9.964E+11  -249.5551     63040.      0.000
    26.000     0.0223   2906474. -1073.6517  -0.000370   714.9330  9.964E+11  -237.2962     66298.      0.000
    26.520     0.0201   2896978. -2516.0336  -0.000352   714.0585  9.964E+11  -225.0057     69920.      0.000
    27.040     0.0179   2878630. -3881.6003  -0.000334   712.3685  9.964E+11  -212.6759     73974.      0.000
    27.560     0.0159   2851910. -5170.0698  -0.000316   709.9075  9.964E+11  -200.2951     78548.      0.000
    28.080     0.0140   2817300. -6381.0699  -0.000298   706.7198  9.964E+11  -187.8460     83755.      0.000
    28.600     0.0122   2775287. -7514.0997  -0.000281   702.8503  9.964E+11  -175.3045     89748.      0.000
    29.120     0.0105   2726361. -8568.4759  -0.000264   698.3440  9.964E+11  -162.6366     96741.      0.000
    29.640   0.008899   2671015. -9543.2589  -0.000247   693.2464  9.964E+11  -149.7939    105039.      0.000
    30.160   0.007412   2609753.    -10437.  -0.000230   687.6039  9.964E+11  -136.7073    115098.      0.000
    30.680   0.006026   2543084.    -11240.  -0.000214   681.4636  9.964E+11  -120.6220    124900.      0.000
    31.200   0.004740   2471639.    -11922.  -0.000198   674.8832  9.964E+11   -97.8423    128794.      0.000
    31.720   0.003551   2396306.    -12462.  -0.000183   667.9447  9.964E+11   -75.5117    132687.      0.000
    32.240   0.002456   2317956.    -12866.  -0.000168   660.7285  9.964E+11   -53.7467    136581.      0.000
    32.760   0.001451   2237441.    -13040.  -0.000154   653.3127  9.964E+11    -2.1292  9159.8204      0.000
    33.280   0.000533   2156772.    -13049.  -0.000140   645.8828  9.964E+11    -0.8488  9938.5724      0.000
    33.800  -0.000300   2076002.    -13050.  -0.000127   638.4436  9.964E+11     0.5159     10717.      0.000
    34.320  -0.001053   1995186.    -13043.  -0.000114   631.0001  9.964E+11     1.9392     11496.      0.000
    34.840  -0.001727   1914382.    -13026.  -0.000102   623.5578  9.964E+11     3.3968     12275.      0.000
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1st neg

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      35.360  -0.002326   1833650.     -13000. -9.031E-05   616.1222  9.964E+11     4.8662     13054.        0.000
      35.880  -0.002854   1753050.     -12965. -7.908E-05   608.6986  9.964E+11     6.3263     13832.        0.000
      36.400  -0.003313   1672641.     -12921. -6.836E-05   601.2926  9.964E+11     7.7578     14611.        0.000
      36.920  -0.003707   1592480.     -12869. -5.813E-05   593.9095  9.964E+11     9.1426     15390.        0.000
      37.440  -0.004039   1512626.     -12808. -4.841E-05   586.5547  9.964E+11    10.4645     16169.        0.000
      37.960  -0.004311   1433131.     -12738. -3.918E-05   579.2329  9.964E+11    11.7087     16947.        0.000
      38.480  -0.004528   1354046.     -12662. -3.046E-05   571.9489  9.964E+11    12.8617     17726.        0.000
      39.000  -0.004691   1275420.     -12578. -2.222E-05   564.7071  9.964E+11    13.9119     18505.        0.000
      39.520  -0.004805   1197295.     -12488. -1.448E-05   557.5115  9.964E+11    14.8489     19284.        0.000
max   40.040  -0.004872   1119710.     -12318. -7.224E-06   550.3657  9.964E+11    39.8529     51044.        0.000
      40.560  -0.004895   1043642.     -12063. -4.494E-07   543.3595  9.964E+11    41.8753     53380.        0.000
neg   41.080  -0.004878    969171.     -11796.  5.854E-06   536.5005  9.964E+11    43.5510     55716.        0.000
      41.600  -0.004822    896365.     -11520.  1.170E-05   529.7948  9.964E+11    44.8612     58053.        0.000
      42.120  -0.004732    825278.     -11238.  1.709E-05   523.2474  9.964E+11    45.7908     60389.        0.000
      42.640  -0.004609    755947.     -10950.  2.204E-05   516.8619  9.964E+11    46.3284     62725.        0.000
      43.160  -0.004457    688397.     -10661.  2.656E-05   510.6402  9.964E+11    46.4662     65061.        0.000
      43.680  -0.004277    622634.     -10372.  3.067E-05   504.5832  9.964E+11    46.1993     67398.        0.000
      44.200  -0.004074    558650.     -10085.  3.437E-05   498.6901  9.964E+11    45.5263     69734.        0.000
      44.720  -0.003848    496421. -9804.6439  3.767E-05   492.9586  9.964E+11    44.4487     72070.        0.000
      45.240  -0.003604    435908. -9444.3554  4.059E-05   487.3851  9.964E+11    71.0283    122989.        0.000
      45.760  -0.003342    378146. -9009.1044  4.314E-05   482.0650  9.964E+11    68.4752    127856.        0.000
      46.280  -0.003065    323038. -8592.0396  4.533E-05   476.9894  9.964E+11    65.1994    132723.        0.000
      46.800  -0.002776    270459. -8197.6305  4.719E-05   472.1467  9.964E+11    61.2138    137590.        0.000
      47.320  -0.002476    220255. -7830.2521  4.873E-05   467.5227  9.964E+11    56.5357    142458.        0.000
      47.840  -0.002168    172246. -7494.1578  4.996E-05   463.1009  9.964E+11    51.1868    147325.        0.000
      48.360  -0.001853    126224. -7193.4539  5.089E-05   458.8621  9.964E+11    45.1927    152192.        0.000
      48.880  -0.001533     81957. -6932.0745  5.154E-05   454.7850  9.964E+11    38.5828    157059.        0.000
      49.400  -0.001210     39191. -6713.7576  5.192E-05   450.8460  9.964E+11    31.3906    161926.        0.000
      49.920  -0.000885 -2354.8310 -4674.2389  5.204E-05   447.4533  9.964E+11   622.3013   4388243.        0.000
      50.440  -0.000560    -19669. -1187.7780  5.197E-05   449.0481  9.964E+11   495.1541   5515213.        0.000
      50.960  -0.000236    -17703.  1035.1121  5.185E-05   448.8670  9.964E+11   217.3107   5738103.        0.000
      51.480  8.690E-05 -7274.8983  1460.4266  5.177E-05   447.9065  9.964E+11   -80.9919   5815978.        0.000
min   52.000   0.000410      0.000      0.000  5.175E-05   447.2364  9.964E+11  -387.0935   2946927.        0.000

Tip * The above values of total stress are combined axial and bending stresses.


   Output Summary for Load Case No. 2:

   Pile-head deflection            =      0.1824859 inches
   Computed slope at pile head     =       0.000000 radians
   Maximum bending moment          =      -4345842. inch-lbs
   Maximum shear force             =         28700. lbs
   Depth of maximum bending moment =       0.000000 feet below pile head
   Depth of maximum shear force    =      0.5200000 feet below pile head
   Number of iterations            =             10
   Number of zero deflection points =             2



   --------------------------------------------------------------------------------
                           Summary of Pile Response(s)
   --------------------------------------------------------------------------------

   Definitions of Pile-head Loading Conditions:

   Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs
   Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
   Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian
   Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
   Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians


                 Pile-head      Pile-head                                   Maximum        Maximum

   Load   Load   Condition 1    Condition 2         Axial        Pile-head    Moment          Shear          Pile-head
```

```
Case  Type    V(lbs) or     in-lb, rad.,        Loading     Deflection       in Pile          in Pile         Rotation

 No.   No.    y(inches)     or in-lb/rad.        lbs          inches         in-lbs            lbs             radians

----  ----  --------------  --------------  -------------  -------------  -------------  -------------
-------------
  1     1   V =     30200.  M =    774000.        833000.    0.73838154       8166086.         -33872.
-0.00325883
  2     2   V =     28700.  S =      0.000        809300.    0.18248594      -4345842.          28700.
0.00000000

The analysis ended normally.
```

GEOTECHNICAL BORING REPORT
BORE LOG

WBS N/A | **TIP** I-5786 | **COUNTY** JOHNSTON | **GEOLOGIST** M. Arnold

SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95

BORING NO. B1-B | **STATION** 17+75 | **OFFSET** 9 ft RT | **ALIGNMENT** -Y1-

COLLAR ELEV. 175.8 ft | **TOTAL DEPTH** 54.6 ft | **NORTHING** 652,408 | **EASTING** 2,224,971

GROUND WTR (ft)
0 HR. NM
24 HR. FIAD

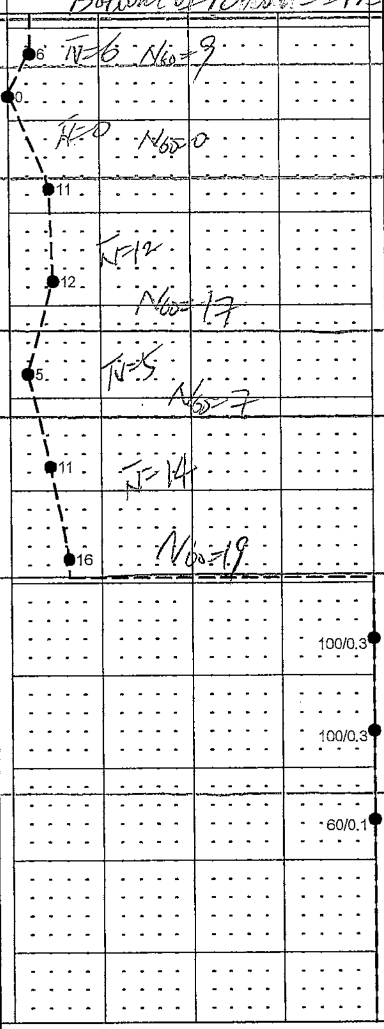
DRILL RIG/HAMMER EFF./DATE F&R3495 CME-55 85% 01/30/2017 | **DRILL METHOD** NW Casing w/ Advancer | **HAMMER TYPE** Automatic

DRILLER D. Tignor | **START DATE** 04/27/17 | **COMP. DATE** 04/28/17 | **SURFACE WATER DEPTH** N/A

BOC = ±189.2

Bottom of 10' Berm = ±175.2

DRIVE ELEV (ft)	DEPTH (ft)	BLOW COUNT 0.5ft	0.5ft	0.5ft	BLOWS PER FOOT	SAMP. NO.	MOI
174.6	1.2	3	3	3	6 N=6 N60=9	1	M
172.3	3.5	WOH	WOH	WOH	0 N=0 N60=0	2	W
167.3	8.5	24	4	7	11		W
162.3	13.5	3	6	6	12 N=12 N60=17	3	M
157.3	18.5	3	2	3	5 N=5 N60=7	4	W
152.3	23.5	3	5	6	11 N=14	5	Sat.
147.3	28.5	4	7	9	16 N60=19		W
142.3	33.5	100/0.3			100/0.3	6	
137.3	38.5	100/0.3			100/0.3		
132.3	43.5	60/0.1			60/0.1	7 RS-3	
						RS-4	



SOIL AND ROCK DESCRIPTION

ELEV. (ft)		DEPTH (ft)
189.2		
175.8	GROUND SURFACE	0.0
174.6	**ASPHALT**	1.2
173	**ROADWAY EMBANKMENT** ~~Black-Dark Gray~~, Silty Fine Sandy CLAY (A-7) with Trace Organics (Wood Fragments)	
168.8	Gray-Brown, Clayey GRAVEL (A-1-b) with Some Fine to Coarse Sand	7.0
166.8	**COASTAL PLAIN** Red-Gray-Brown, Fine Sandy Silty CLAY (A-7)	9.0
158.8		
153.8	Orange-Tan, Silty Fine to Coarse SAND (A-2-4) with Trace Gravel	22.0
148.8	Orange-Tan, Clayey Fine to Coarse SAND (A-2-6) with Trace Gravel	27.0
145.3	**WEATHERED ROCK** Black-Orange-Tan and Gray (META-ARGILLITE)	30.5
133.8		42.0
132.2	**CRYSTALLINE ROCK** Blue-Gray (META-ARGILLITE)	43.6
121.2	Boring Terminated at Elevation 121.2 ft in Crystalline Rock (META-ARGILLITE) Start Coring at 43.6'	54.6

GEOTECHNICAL BORING REPORT
CORE LOG

WBS N/A	**TIP** I-5786	**COUNTY** JOHNSTON	**GEOLOGIST** M. Arnold
SITE DESCRIPTION Bridge No. 108 on SR 1001 (Lizzie Mill Road) over I-95			**GROUND WTR (ft)**
BORING NO. B1-B	**STATION** 17+75	**OFFSET** 9 ft RT	**ALIGNMENT** -Y1-	**0 HR.** NM
COLLAR ELEV. 175.8 ft	**TOTAL DEPTH** 54.6 ft	**NORTHING** 652,408	**EASTING** 2,224,971	**24 HR.** FIAD
DRILL RIG/HAMMER EFF./DATE F&R3495 CME-55 85% 01/30/2017		**DRILL METHOD** NW Casing w/ Advancer	**HAMMER TYPE** Automatic
DRILLER D. Tignor	**START DATE** 04/27/17	**COMP. DATE** 04/28/17	**SURFACE WATER DEPTH** N/A
CORE SIZE NQ	**TOTAL RUN** 11.0 ft		

ELEV (ft)	RUN ELEV (ft)	DEPTH (ft)	RUN (ft)	DRILL RATE (Min/ft)	RUN REC. (ft) %	RUN RQD (ft) %	SAMP. NO.	STRATA REC. (ft) %	STRATA RQD (ft) %	LOG	ELEV. (ft)	DESCRIPTION AND REMARKS	DEPTH (ft)
132.2												Begin Coring @ 43.6 ft	
	132.2 131.2	43.6 44.6	1.0	2:30/1.0	(1.0) 100%	(0.8) 80%		(10.4) 95%	(9.3) 85%		132.2	Gray, Very Slightly Weathered to Fresh, Medium Hard to Moderately Hard, META-ARGILLITE with Close to Moderately Close Fracture Spacing RS-3: 44.3-44.6, qu=3,347 psi,GSI=35-45 RS-4: 49.7-50.0, qu=4,608 psi, GSI=35-45	43.6
130			5.0	2:13/1.0 1:50/1.0 2:00/1.0 2:11/1.0 2:28/1.0	(4.4) 88%	(3.5) 70%	RS-3						
	126.2	49.6											
125			5.0	2:16/1.0 1:56/1.0 2:02/1.0 2:36/1.0 2:19/1.0	(5.0) 100%	(5.0) 100%	RS-4						
	121.2	54.6									121.2		54.6

Boring Terminated at Elevation 121.2 ft in Crystalline Rock
(META-ARGILLITE)

Start Coring at 43.6'

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING UNIT
Drilled Pier Axial Resistance Worksheet

PROJECT: I-5786 COUNTY: Johnston
DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 right
DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
CHECKED BY: DATE: STR. NO.: PAGE: 1 OF 9

Elevations

Bottom of Cap (BOC) Elevation = 189.20 ft
Top of Pier/Bottom of Column Elevation = 175.20 ft
Natural Ground / Finished Grade Elevation = 176.00 ft
Groundwater Table (GWT) Elevation = 0.00 ft
Design Scour (DSE) Elevation = 176.00 ft
Amount of Contraction Scour (from BSR) = 0.00 ft
Is Permanent Casing Required? ○ Yes / Maybe ● No
Bottom of Permanent Casing Elevation = N/A ft
Drilled Pier Tip Elevation = 133.80 ft

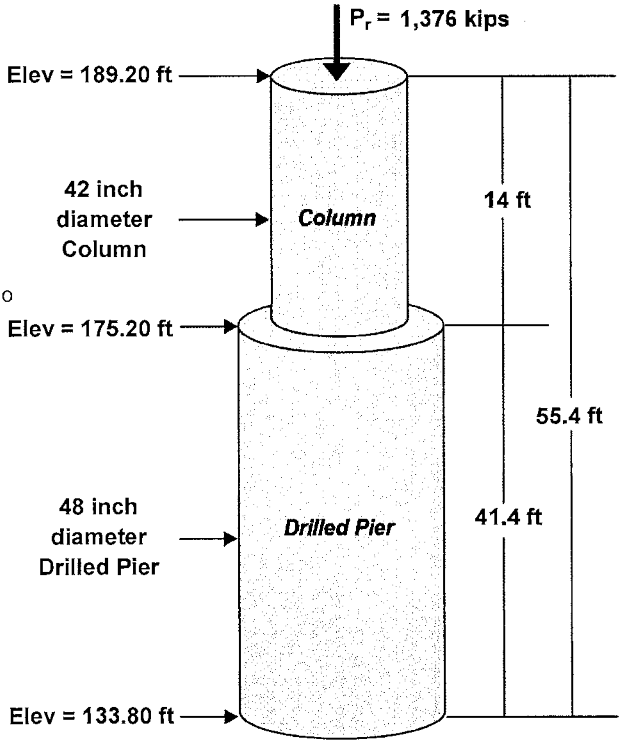


Figure shows typical drilled pier

Drilled Pier Information

Maximum Factored Axial Load (P_r) = 1,376.0 kips
Number of Drilled Piers per Bent = 2
Diameter of Column (d_{Column}) = 42 in
Diameter of Drilled Pier (d_{DP}) = 48 in
Unit Weight of Concrete (γ_c) = 0.150 kcf
Compressive Strength of Concrete (f'_c) = 4.500 ksi

Subsurface Information and Soil/Rock Layer Properties

internally calculate $N1_{60}$ values at midpoint of each layer ☑

Subsurface Boring Name / ID No. = B1-B
SPT Hammer Energy Efficiency Rating (ER) = 85 %
Top of Boring (Collar) Elevation = 175.80 ft
Depth to Groundwater Table (for actual boring) = 0.00 ft

Calculate GSI using RQD values : ☐
(Use if GSI is not shown on boring)

Layer No.	Material Description	Layer Elevations		Total γ (kcf)	N (bpf)	N_{60} (bpf)	$N1_{60}$ (bpf)	RQD (%)	(2) GSI	q_u (ksf)	E_i (ksi)	v
		Top(1) (ft)	Bottom (ft)									
1	Cohesive Soil (Clay)	175.20	173.00	0.115	6	9	18					
2	Cohesive Soil (Clay)	173.00	166.80	0.100	0	0	0					
3	Cohesive Soil (Clay)	166.80	158.80	0.120	12	17	24					
4	Cohesive Soil (Clay)	158.80	153.80	0.115	5	7	9					
5	Cohesionless Soil (Silty Sand)	153.80	145.30	0.120	14	20	23					
6	Weathered Rock	145.30	133.80	0.130	100	142	142					
7	Hard Rock	133.80	133.80	0.130	100		N/A	70	35	480		
8												
TIP (3)	Hard Rock	133.80	125.80	0.130	150		N/A	80	35	664	500	0.200

Notes

1. Resistance from subsurface layers above the Bottom of Column Elevation, Drilled Pier Design Scour Elevation, and Permanent Casing Elevation will be ignored.
2. Hard rock layers with poor or very poor quality rock mass (GSI < 30) will be modeled as weathered rock.
3. Input the subsurface information for the soil / rock at the base of the drilled pier to a distance of 2 pier diameters below the base of the drilled pier.

DISCLAIMER: *The application of this spreadsheet is the responsibility of the user. It is imperative that the user understands the potential accuracy limitations and examines the reasonableness of the results with engineering knowledge and experience. There are no expressed or implied warranties.*

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING UNIT
Drilled Pier Axial Resistance Worksheet

PROJECT: I-5786 COUNTY Johnston
DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 right
DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
CHECKED BY: ______ DATE: ______ STR. NO.: ____ PAGE: 1A OF 9

Correcting SPT Values for Hammer Efficiency and Overburden Pressure

SPT-N Value Corrected for Hammer Efficiency, (N_{60})

N_{60} = (ER/60%)(N) *AASHTO Eqn. 10.4.6.2.4-2*

N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used. If ER is not known, use 80% for automatic hammers and 60% for drop hammers.
N = uncorrected SPT blow count (blows/ft)

SPT-N Value Corrected for Overburden Pressure, (N1)

N1 = $(C_N)(N)$ *AASHTO Eqn. 10.4.6.2.4-1*

N1 = SPT blow count corrected for overburden pressure (blows/ft)
C_N = correction factor = $[0.77\log_{10}(40/\sigma'_v) < 2.0$
$\sigma'_v = \sigma_v - \mu$ = effective vertical stress at the depth of the SPT-N value (ksf)
σ_v = total vertical stress at the depth of the SPT-N value (ksf)
μ = total pore water pressure at the depth of the SPT-N value (ksf)
N = uncorrected SPT blow count (blows/ft)

SPT-N Value Corrected for both Overburden Pressure and Hammer Efficiency, ($N1_{60}$)

$N1_{60} = (C_N)(N)$ *AASHTO Eqn. 10.4.6.2.4-3*

Summary of Corrected N Values for Boring

Top of Boring (Collar) Elevation = 175.8 ft
Hammer Efficiency (ER) = 85 %
Depth to Groundwater Table = 0.0 ft
Unit Weight of Water = 0.0624 kcf

Layer No.	Layer Elevations		σ_v at top (ksf)	Δz (ft)	Total γ (kcf)	σ_v at bottom (ksf)	σ_v at midpoint (ksf)	z_{water} (ft)	μ at midpoint (ksf)	σ'_{v0} at midpoint (ksf)	N (bpf)	N_{60} (bpf)	C_N	$N1_{60}$ (bpf)
	Top (ft)	Bottom (ft)												
1	175.20	173.00	0.072	2.20	0.115	0.325	0.199	1.70	0.106	0.092	6	9	2	18
2	173.00	166.80	0.325	6.20	0.100	0.945	0.635	5.90	0.368	0.267	0	0	1.68	0
3	166.80	158.80	0.945	8.00	0.120	1.905	1.425	13.00	0.811	0.614	12	17	1.4	24
4	158.80	153.80	1.905	5.00	0.115	2.480	2.193	19.50	1.217	0.976	5	7	1.24	9
5	153.80	145.30	2.480	8.50	0.120	3.500	2.990	26.25	1.638	1.352	14	20	1.13	23
6	145.30	133.80	3.500	11.50	0.130	4.995	4.248	36.25	2.262	1.986	100	142	1	142
7	133.80	133.80	4.995		0.130	4.995	4.995	42.00	2.621	2.374	N/A		2	N/A
8														
TIP	133.80	125.80	4.995	8.00	0.130	6.035	5.515	46.00	2.870	2.645	N/A		2	N/A

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL ENGINEERING UNIT
Drilled Pier Axial Resistance Worksheet

PROJECT: I-5786 COUNTY Johnston
DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 right
DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
CHECKED BY: DATE: STR. NO.: PAGE: 1B OF 9

Selecting Design Properties for Hard Rock

1. q_u values for rock should be based on AASHTO Table 10.4.6.4-1 (which uses Point Load Index Testing) or actual values from Uniaxial Compressive Strength Testing. If neither of these options is available, the NCDOT Rock Core Database may be used to estimate compressive strength.

2. E_i and v values for rock should be based on AASHTO Tables C10.4.6.5-1, and 2 if lab test data is not available

Unconfined Compressive Strength from Point Load Strength Index for Hard Rock AASHTO Table C10.4.6.4-1

Parameter			Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0

Summary of Elastic Moduli for Intact Rock, E_i (modified by Kulhawy, 1978) AASHTO Table C10.4.6.5-1

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (ksi ×10³)			Standard Deviation (ksi × 10³)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Summary of Poisson's Ratio for Intact Rock, v (modified by Kulhawy, 1978) AASHTO Table C10.4.6.5-2

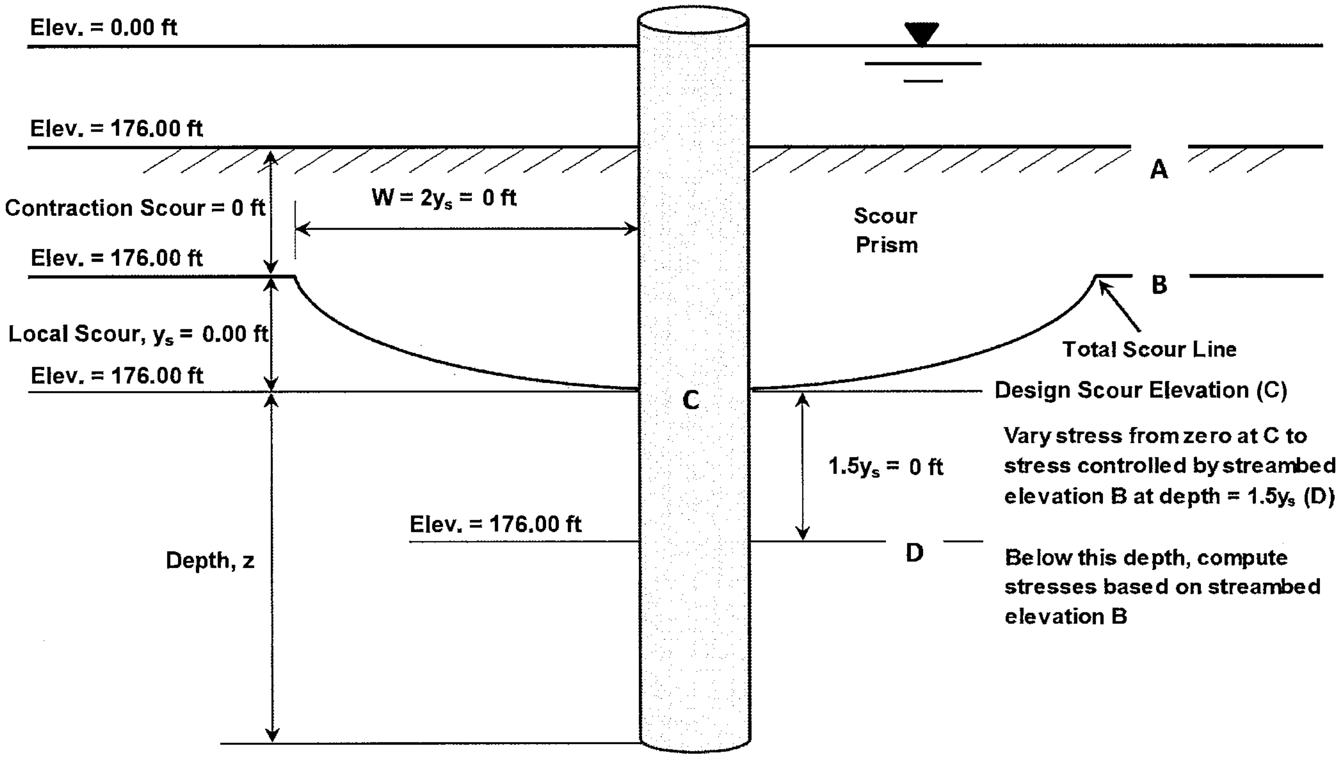
Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

Calculating Design Stresses for Drilled Piers based on Scour Prism used in FHWA GEC 010

For analysis purposes, lower ground line to the contraction scour elevation (CSE) to account for contraction scour reported in the bridge survey report.

- If the CSE is lower than or equal to the design scour elevation (DSE), consider all scour as contraction scour and lower t ground line to the design scour elevation (DSE).
- If the CSE is higher than the DSE, consider the difference between the CSE and the DSE as local scour.

Groundwater Elevation = 0.00 ft
Original Pre-Scour Streambed Elevation (Point A) = 176.00 ft *= Natural Ground / Finished Grade Elevation*
Amount of Contraction Scour = 0.00 ft
Streambed Elevation after General Scour (Point B) = 176.00 ft *= Point A - Contraction Scour ≥ Design Scour Elevation*
Amount of Local Scour (y_s) = 0.00 ft
p of the embedded length of the drilled pier (Point C) = 176.00 ft *= Design Scour Elevation*
1.5(y_s) = 0.00 ft
evation corresponding to a depth of 1.5(y_s), (Point D) = 176.00 ft *= Point C - 1.5y_s*



Adapted from FHWA GEC 010 Figure 13.18: Illustration of Scour Prism and Effects on Drilled Pier

Per FHWA GEC 010 page 13-46, vertical stress along any depth of the drilled pier can be estimated as follows;

1) At the top of the embedded drilled pier (Point C) the vertical stress is equal to zero.
2) At a depth of 1.5ys (Point D) or greater, assume the vertical stress is controlled by the streambed elevation (Point B).
3) Assume a linear variation in vertical stress from 0 at Point C to the vertical stress value controlled by the streambed at Point

OF TRANSPORTATION DESCRIPTION: Bridge 108 Lizzie Mill Rd, bent 1 right
GEOTECHNICAL ENGINEERING UNIT DESIGNED BY: CW DATE: 05/11/17 STATION: Bent 1
Drilled Pier Axial Resistance Worksheet CHECKED BY: DATE: STR. NO.: PAGE: 1D OF 9

Soil Layer Profile and Effective Vertical Stress controlled by the streambed elevation (Point B)

- Assume the streambed elevation is equal to the contraction scour elevation (Elevation 176.00 ft).

Layer No.	Top (ft)	Midpoint (ft)	Bottom (ft)	σ_{v_top} (ksf)	$\mu_{\_top}$ (ksf)	σ'_{v_top} (ksf)	Δz (ft)	γ (kcf)	σ_{v_bottom} (ksf)	$\mu_{\_bottom}$ (ksf)	σ'_{v_bottom} (ksf)
0	176.00	175.60	175.20	0.000	0.000	0.000	0.80	0.120	0.096	0.000	0.096
1	175.20	174.10	173.00	0.096	0.000	0.096	2.20	0.115	0.349	0.000	0.349
2	173.00	169.90	166.80	0.349	0.000	0.349	6.20	0.100	0.969	0.000	0.969
3	166.80	162.80	158.80	0.969	0.000	0.969	8.00	0.120	1.929	0.000	1.929
4	158.80	156.30	153.80	1.929	0.000	1.929	5.00	0.115	2.504	0.000	2.504
5	153.80	149.55	145.30	2.504	0.000	2.504	8.50	0.120	3.524	0.000	3.524
6	145.30	139.55	133.80	3.524	0.000	3.524	11.50	0.130	5.019	0.000	5.019
7	133.80	133.80	133.80	5.019	0.000	5.019		0.130	5.019	0.000	5.019
8											

Variation in Vertical Stress from Point C to Point D

- Assume the top of the embedded drilled pier is equal to the design scour elevation.
- Vertical stress at elevation 176 ft (Point C) = 0 ksf
- Assume a linear variation in vertical stress from 0 ksf at elevation 176.00 ft (Point C) to a stress value controlled by the elevation 176.00 ft (Point B) at the depth Point D, elevation 176.00 ft.
- PointD lies within Soil Layer No.0

Point D Elevation (ft)	Top of Layer 0 (ft)	σ_v at 176.00 ft	Depth below Layer 0 (ft)	γ for Layer 2	μ at Point D (ksf)	σ'_v at Point D (ksf)
176.00	176.00	0.000	0.00	0.120	0.000	0.000

Point	Elevation (ft)	z (ft)	σ'_v (ksf)	Equation for linear variation over a depth of $1.5y_s$
C	176.00	0.00	0.000	σ'_v (for z = 0 to 22.5 ft) = (0.0000)z
D	176.00	0.00	0.000	

- All stress calculations below elevation 176.00 ft (Point D) will be based on elevation 176.00 ft (Point B).

Summary of Design Stress at the Midpoint of each Soil Layer and at Tip of Drilled Pier

Layer	Top (ft)	Bottom (ft)	Midpoint (ft)	z (ft)	$\sigma_{v\_midpoint}$ (ksf)	μ (ksf)	$\sigma'_{v\_midpoint}$ (ksf)
1	175.20	173.00	174.10	1.90	0.223	0.000	0.223
2	173.00	166.80	169.90	6.10	0.659	0.000	0.659
3	166.80	158.80	162.80	13.20	1.449	0.000	1.449
4	158.80	153.80	156.30	19.70	2.217	0.000	2.217
5	153.80	145.30	149.55	26.45	3.014	0.000	3.014
6	145.30	133.80	139.55	36.45	4.272	0.000	4.272
7	133.80	133.80	133.80	42.20	5.019	0.000	5.019

Tip Elev. (ft)	z (ft)	σ_{v_bottom} (ksf)	μ (ksf)	σ'_{v_bottom} (ksf)
133.80	42.20	5.019	0.000	5.019

Side Resistance in Cohesive Soil (Clays with $S_U \leq 5$ ksf)

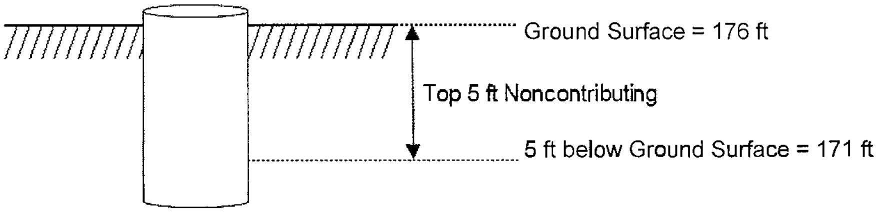
$R_s = (q_s)(A_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for soil layer (ksf)
$= (\alpha)(S_U)$ *AASHTO Eqn. 10.8.3.5.1b-1*

α = adhesion factor
= 0 between the ground surface and a depth of 5 ft
$= 0.55$ for $(S_U/p_a) \leq 1.5$ *AASHTO Eqn. 10.8.3.5.1b-2*
$= 0.55 - 0.1(S_U/p_a - 1.5)$ for $1.5 \leq (S_U/p_a) \leq 2.5$ *AASHTO Eqn. 10.8.3.5.1b-3*

S_U = undrained shear strength (ksf)
$= 100(N1_{60})/1000$ *NCDOT Empirical Formula*

ρ_a = atmospheric pressure (2.12 ksf)



Based on AASHTO Figure 10.8.3.5.1b-1

A_s = area of drilled pier side resistance (ft^2)
$= (\pi)(B)(\Delta z)$

B = diameter of drilled pier (4 ft)
Δz = effective thickness of the soil layer (ft)

Layer No.	Layer Elevations		$N1_{60}$	S_U (ksf)	S_U/ρ_a	α	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
	Top (ft)	Bottom (ft)								
1	175.20	173.00	18	1.800	0.85	0.00	0.000	0.00	0.00	0
2	173.00	166.80	0	0.000	0.00	0.55	0.000	4.20	52.78	0
3	166.80	158.80	24	2.400	1.13	0.55	1.320	8.00	100.53	133
4	158.80	153.80	9	0.900	0.42	0.55	0.495	5.00	62.83	31

Total Side Resistance in Cohesive Soil = 164

Side Resistance in Cohesionless Soil (Sand / Gravel with $N1_{60} \leq 100$)

$R_s = (q_s)(A_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for soil layer (ksf)
$= (\beta)(\sigma'_v)$ *AASHTO Eqn. 10.8.3.5.2b-1*

β = load transfer coefficient

$$= (1 - \sin \phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin \phi'_f} \tan \phi'_f$$ *AASHTO Eqn. 10.8.3.5.2b-2*

ϕ'_f = effective friction angle
$= 27.5 + 9.2 \log(N1_{60}), \quad N1_{60} \leq 100$ *AASHTO Eqn. 10.8.3.5.2b-3*

$N1_{60}$ = SPT - N value corrected for hammer efficiency and overburden (limited to 100 bpf)

σ'_p = effective vertical preconsolidation stress

For Sands: $\frac{\sigma'_p}{\rho_a} \approx 0.47(N_{60})^m$ *AASHTO Eqn. 10.8.3.5.2b-4*

For Gravels: $\frac{\sigma'_p}{\rho_a} = 0.15(N_{60})$ *AASHTO Eqn. 10.8.3.5.2b-5*

m = 0.6 for clean sands; 0.8 for silty sands and sandy silts
N_{60} = SPT - N value corrected for hammer efficiency (limited to 100 bpf)
ρ_a = atmospheric pressure (2.12 ksf)

σ'_v = effective vertical stress at soil layer mid-depth as defined in FHWA GEC 010 pages 13-46

A_s = area of drilled pier side resistance (ft^2)
$= (\pi)(B)(\Delta z)$

B = diameter of drilled pier (4 ft)
Δz = effective thickness of the soil layer (ft)

Layer No.	Layer Elevations		Material Type	$N1_{60}$	ϕ' (deg)	m	N_{60}	σ'_p/ρ_a	σ'_v (ksf)	β	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
	Top (ft)	Bottom (ft)												
5	153.80	145.30	Sand	23	40	0.8	20	5.160	3.014	0.686	2.068	8.50	106.81	221

Total Side Resistance in Cohesionless Soil = 221

Side Resistance in Weathered and Hard Rock

$R_s = (A_s)(q_s)$ *AASHTO Eqn. 10.8.3.5-3*

q_s = unit side resistance for weathered or hard rock layer (ksf)

For weathered rock layers or hard rock layers with a GSI < 30
= 8 ksf *NCDOT Policy*

For drilled piers socketed into hard rock

$$= \left(C \sqrt{\frac{q_u}{p_a}} \right) p_a$$ *AASHTO Eqn. 10.8.3.5.4b-1*

C = regression coefficent taken as 1.0 for normal rock sockets (see AASHTO C10.8.3.5.4b-1 for details)

For fractured rock that caves and cannot be drilled without artificial support

$$= \left(0.65\alpha_E \sqrt{\frac{q_u}{p_a}} \right) p_a$$ *AASHTO Eqn. 10.8.3.5.4b-2*

a_E = reduction factor to account for jointing in rock (from AASHTO Table 10.8.3.5.4b-1)

RQD (%)	Joint Modificaiton Factor, α_E	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

q_u = Uniaxial Compressive Strength of Intact Rock (ksf) $\leq f'_c$
f'_c = 28 day Compressive Strength of Concrete (4.5 ksi = 648 ksf)
p_a = atmospheric pressure (2.12 ksf)

A_s = area of drilled pier side resistance (ft^2)
= (π)(B)(Δz)

B = diameter of drilled pier (subtract 2 inches to account for possible reduction of drilled pier in rock)
= (48 inches - 2 inches) / 12 inches per ft = 3.83 ft

Δz = effective thickness of the soil layer (ft)

Layer No.	Rock Type	Layer Elevations		AASHTO Equation and Rock Joint Condition to use	RQD (%)	α_E	q_u (ksf)	q_s (ksf)	Δz (ft)	A_s (ft^2)	R_s (kips)
		Top (ft)	Bottom (ft)								
6	Weathered Rock	145.30	133.80	N/A	N/A	N/A	N/A	8.000	11.50	138.49	1108

Total Side Resistance in Weathered and Hard Rock = 1,108

Tip Resistance in Hard Rock

$R_p = (q_p)(A_p)$ *AASHTO Eqn. 10.8.3.5-2*

q_p = unit tip resistance (ksf)

If rock to a depth of 2B below drilled pier tip is intact or tightly jointed and the depth of socket > 1.5 D

$= 2.5q_u$ *AASHTO Eqn. 10.8.3.5.4c-1*

If the rock to a depth of 2D below the drilled pier tip is jointed with random orientation

$$= A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ *AASHTO Eqn. 10.8.3.5.4c-2*

q_u = Uniaxial Compressive Strength of Intact Rock (ksf)
σ'_{vb} = vertical effective stress at the socket bearing elevation

$$A = \sigma'_{vb} + q_u\left[m_b\left(\frac{\sigma'_{vb}}{q_u}\right) + s\right]^a$$ *AASHTO Eqn. 10.8.3.5.4c-3*

$$s = exp\left(\frac{GSI - 100}{9}\right)$$ *AASHTO Eqn. 10.4.6.4-2*

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ *AASHTO Eqn. 10.4.6.4-3*

$$m_b = exp\left(\frac{GSI - 100}{28}\right)m_i$$ *AASHTO Eqn. 10.4.6.4-4*

m_i = constant for intact rock *AASHTO Table 10.4.6.4-1*

GSI = Global Strength Index

***Hard Rock Layers with an GSI less than 30 will be modeled as weathered rock.**

A_p = area of drilled pier tip resistance (ft^2)
$= (\pi)(B^2)/4$

B = diameter of drilled pier - 2 inches to account for possible reduction for drilled pier in rock (B = 3.83 ft)

Tip Elevation (ft)	AASHTO Equation used to calculate q_u	q_u (ksf)	GSI	m	m_b	s	a	A	q_p (ksf)	A_p (ft^2)	R_p (kips)
133.80	10.8.3.5.4c-2	664	35	13	1.2757	0.00073	0.51595	67.8938	301	11.54	3,474

Tip Resistance in Hard Rock (continued)

Table 10.4.6.4-1—Values of the Constant mi by Rock Group

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Summary of Nominal and Factored Side Resistance

Cohesionless IGM	Nominal Side Resistance (kips)	Resistance Factor from AASHTO Table 10.5.5.2.4-1	Factored Side Resistance (kips)	Percentage of Side Resistance produced by Material Type
Cohesive Soil	164	0.45	74	11.0%
Cohesionless Soil	221	0.55	122	14.8%
CohesiveIGM	0	0.60	0	0.0%
Weathered Rock	1,108	0.60	665	74.2%
Hard Rock	0	0.55	0	0.0%
Total	**1,493**		**861**	**100%**

Note: *Side resistance in soil and weathered rock develops at a much greater displacement than hard rock. If the pier does not have a true rock socket, the side resistance from the hard rock will be ignored and nominal side resistance will be based on the total side resistance in soil and weathered rock.*

Total Nominal Side Resistance = 1,493 kips
Total Factored Side Resistance = 861 kips

Summary of Total Nominal and Factored Tip Resistance

Total Nominal Tip Resistance = 3,474 kips
Tip Resistance Factor = 0.50
Total Factored Tip Resistance = 1,737 kips

the drilled pier is bearing on Hard Rock
for Hard Rock, see AASHTO Table 10.5.5.2.4-1.

Required Factored Resistance

$R_{req} = P_r + \gamma_{DC}(W_{Column} + W_{Pier}) - \gamma_{WA}W_{Water} - \gamma_{DC}W_{Soil/Rock} \geq P_r$ — *Required Factored Resistance*

P_r = 1,376 kips — *Maximum Factored Axial Load Reported by Structure Design*
γ_{DC} = 1.25 — *Factor for Permanent Dead Loads, from AASHTO Table 3.4.1-2*
γ_{WA} = 1.00 — *Factor for Water Loads, from AASHTO Table 3.4.1-1*

$W_{Column} = (A_{Column})(L_{Column})(\gamma_c)$ — *Unfactored Weight of Column*
A_{Column} = 9.62 ft^2 — *Area of Column*
L_{Column} = 14 ft — *Length of Column*
γ_c = 0.150 kcf — *Unit Weight of Concrete*
= 20 kips

$W_{Pier} = (A_{Pier})(L_{Pier})(\gamma_c)$ — *Unfactored Weight of Drilled Pier*
A_{Pier} = 12.57 ft^2 — *Area of Drilled Pier*
L_{Pier} = 41.4 ft — *Length of Drilled Pier*
γ_c = 0.150 kcf — *Unit Weight of Concrete*
= 78 kips

$W_{Water} = (A_{Pier})(z_w)(\gamma_w)$ — *Unfactored Weight of Water Displaced by Drilled Pier*
A_{Pier} = 12.57 ft^2 — *Area of Drilled Pier*
z_w = 0 ft — *Depth from water surface to the drilled pier tip*
γ_w = 0.0624 kcf — *Unit Weight of Water*
= 0 kips

$W_{Soil/Rock} = (A_{Pier})(\sigma'_{vo})$ — *Unfactored Effective Weight of Soil / Rock that will be displaced*
A_{Pier} = 12.57 ft^2 — *Area of Drilled Pier*
σ'_{vo} = 5.019 ksf — *ffective vertical stress at drilled pier tip as defined in FHWA GEC 010 pages 13-46*
$W_{Soil/Rock}$ = 63 kips

R_{req} = 1,376 kips + 1.25(20 kips + 78 kips) -1.00(0 kips) -1.25(63 kips) = 1,420 kips =710T ~~Say~~ 715T to be equivalent w/ B-LA

Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket

For Load Transfer of a drilled pier that is bearing on hard rock with no rock socket, the total displacement of the pier will be controlled by the rock layer below the base of the pier. The total displacement, (w_c), will be calculated using FHWA GEC 10 Equation D-17 and assumes the entire load is carried by the tip. Use the normalized load transfer values along with the total factored side resistance to calculate the factored side resistance developed in the soil and weathered rock layers at this displacement. The remaining factored resistance that is carried by the drilled pier tip must be less than or equal to the total factored tip resistance.

Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket (continued)

Calculate the total displacement, w_c, at the drilled pier tip (assume entire load is carried by the tip)

$w_c = F_3\left(\frac{Q_c}{\pi E_r B}\right) - F_4 B$ *FHWA-NHI-10-016 Eqn. D-17*

$Q_c = R_{req}$ = 1,420 kips *Required Factored Resistance*

B = 48 inches *Diameter of Drilled Pier*

L = 0.01 inches *Length of Pier in Hard Rock (use 0.01 inch when assuming entire load carried by tip)*

E_c = 3,824 ksi *Elastic Modulus of Concrete*

v_c = 0.25 *Poisson's Ration for Concrete*

E_{i_r} = 500 ksi *Elastic Modulus of Intact Rock around Drilled Pier Tip*
(Assume $E_{i\_r} = E_{i\_b}$ if the tip of the drilled pier is sitting on top of the hard rock layer)

E_r ($\leq E_{i\_r}$) = 500 ksi *Elastic Modulus of Rock Mass around Drilled Pier Tip (AASHTO Eqn. 10.4.6.5-1)*
(Assume $E_r = E_b$ if the tip of the drilled pier is sitting on top of the hard rock layer)

v_r = 0.20 *Poisson's Ration of Rock Mass around Drilled Pier Tip*
(Assume $v_r = v_b$ if the tip of the drilled pier is sitting on top of the hard rock layer)

E_{i_b} = 500 ksi *Elastic Modulus of Intact Rock below Drilled Pier Tip*

E_b ($\leq E_{i\_b}$) = 345 ksi *Elastic Modulus of Rock Mass below Drilled Pier Tip (FHWA-NHI-10-016 Table 3-9)*

v_b = 0.20 *Poisson's Ration of Rock Mass below Drilled Pier Tip*

ζ = 0.01 $= ln[5(1 - v_b)L/B]$ *(must be > 0) FHWA-NHI-10-016 Eqn. D-11*

a_1 = 17.48347125 $= (1 + v_r)\zeta + a_2$ *FHWA-NHI-10-016 Eqn. D-25*

a_2 = 17.47147125 $= \left[(1 - v_c)\left(\frac{E_r}{E_C}\right) + (1 + v_r)\right]\left(\frac{1}{2\tan\phi\tan\psi}\right)$ *FHWA-NHI-10-016 Eqn. D-26*

a_3 = 0.186814401 $= \left(\frac{v_c}{2\tan\psi}\right)\left(\frac{E_r}{E_c}\right)$ *FHWA-NHI-10-016 Eqn. D-27*

β = 68.5803 $= a_3\left(\frac{E_c}{E_r}\right)B$ *FHWA-NHI-10-016 Eqn. D-24*

α = 77019.02674 $= a_1\left(\frac{E_c}{E_r}\right)\left(\frac{B^2}{4}\right)$ *FHWA-NHI-10-016 Eqn. D-23*

λ_1 = 0.003185489 $= \frac{-\beta + (\beta^2 + 4\alpha)^{1/2}}{2\alpha}$ *FHWA-NHI-10-016 Eqn. D-22*

λ_2 = -0.004075922 $= \frac{-\beta - (\beta^2 + 4\alpha)^{1/2}}{2\alpha}$ *FHWA-NHI-10-016 Eqn. D-22*

D_3 = 1.701991579 $= \left[\pi(1 - {v_b}^2)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_2 B\right] exp[\lambda_2 D]$ *FHWA-NHI-10-016 Eqn. D-21*

D_4 = 7.796133474 $= \left[\pi(1 - {v_b}^2)\left(\frac{E_r}{E_b}\right) + 4a_3 + a_1\lambda_1 B\right] exp[\lambda_1 D]$ *FHWA-NHI-10-016 Eqn. D-21*

Load Transfer and Developed Resistance for Drilled Piers in Hard Rock with no Rock Socket (continued)

C_3 = 0.279283221 $= \dfrac{D_3}{D_4 - D_3}$ *FHWA-NHI-10-016 Eqn. D-20*

C_4 = 1.279283221 $= \dfrac{D_4}{D_4 - D_3}$ *FHWA-NHI-10-016 Eqn. D-20*

F_3 = 4.375185967 $= a_1(\lambda_1 B C_3 - \lambda_2 B C_4) - 4a_3$ *FHWA-NHI-10-016 Eqn. D-18*

F_4 = 9.96259E-08 $= \left[1 - a_1\left(\dfrac{\lambda_1 - \lambda_2}{D_4 - D_3}\right)B\right] a_2\left(\dfrac{c}{E_r}\right)$ *FHWA-NHI-10-016 Eqn. D-19*

w_c = 0.08 inches *Total Displacement at Drilled Pier Tip*

Calculate the developed side resistance in the soil / weathered rock layers at the displacement, w_c

The majority of the side resistance is produced by Weathered Rock , which is treated as a cohesive material for Load transfer. Use AASHTO Figure 10.8.2.2.2.1 to predict the normalized load transfer for side resistance.

Δz / D (%)	Normalized Side Transfer R_{sd} / R_s AASHTO Figure 10.8.2.2.2.1
0.0	0.00
0.3	0.83
0.6	0.95
1.0	0.93
1.3	0.91
1.6	0.88
2.0	0.83
5.0	0.55

Developed Side Resistance					
$\frac{\Delta z}{D}$	D (in)	Δz (in)	$\varphi_{qs}R_s$ (kips)	$\frac{R_{sd}}{R_s}$	$\varphi_{qs}R_{sd}$ (kips)
0.00%	48	0.00	861	0.00	**408**
0.17%	**48**	**0.08**	**861**	**0.47**	
0.30%	48	0.14	861	0.83	

Δz / D = total settlement / drilled pier diameter

R_{sd} / R_s = developed side resistance / total nominal side resistance

$\varphi_{qs}R_s$ = total factored side resistance

$\varphi_{qs}R_{sd}$ = developed factored side resistance

= $(R_{sd}/R_s)(\varphi_{qs}R_s)$

Calculate the remaining resistance that must be carried by the tip (must ≤ the total factored tip resistance)

Required Factored Resistance = 1,420 kips

Developed Factored Side Resistance = 408 kips

Required Factored Tip Resistance = 1,012 kips **≤ 1,737 kips** **OK**

Required Tip Resistance

q_{req} = required tip resistance (rounded up to the nearest 10 ksf or 5 tsf)

$$= \frac{\dfrac{R_{req} - \varphi_{qs}R_{sd}}{A_T}}{\varphi_{qp}} \leq q_p$$

NCDOT policy

R_r = required factored geotechnical resistance (kips)

$\varphi_{qs}R_{sd}$ = factored developed side resistance (kips)

A_T = area of drilled pier tip (ft^2)

φ_{qp} = tip resistance factor

q_p = unit tip resistance (ksf)

R_{req} (kips)	$\varphi_{qs}R_{sd}$ (kips)	A_{Tip} (ft^2)	φ_{qp}	q_p (ksf)	q_{req} (ksf)
1,420	408	11.54	0.50	301	180

tip resistance req'd

= 90 tsf, say 95 to be equivalent w/ BHA.

```
===============================================================================

                LPile Plus for Windows, Version 2013-07.001

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

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===============================================================================

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Serial Number of Security Device:       293783516
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-------------------------------------------------------------------------------
                          Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:        F:\Projects 66U\66U-0390 (WEI-I-5786 Bridges 108 & 111 Johnston
Co)\NON_CADD\Foundation Recs\Bridge 108 Lizzie Mill\Lateral\
Name of input data file:       B1-B.lp7d
Name of output report file:    B1-B.lp7o
Name of plot output file:      B1-B.lp7p
Name of runtime messeage file: B1-B.lp7r


-------------------------------------------------------------------------------
                         Date and Time of Analysis
-------------------------------------------------------------------------------

              Date:  May 12, 2017     Time:   1:15:55


-------------------------------------------------------------------------------
                               Problem Title
-------------------------------------------------------------------------------

Project Name: Bridge 108 Lizzie Mill


Job Number: 66U-0390


Client: Wetherill Engineering


Engineer: C. Wang


Description: Bent 1 B1-B



-------------------------------------------------------------------------------
                              Program Options
-------------------------------------------------------------------------------

Engineering Units of Input Data and Computations:
 - Engineering units are US Customary Units: pounds, inches, feet

Analysis Control Options:
```

```
- Maximum number of iterations allowed             =         500
- Deflection tolerance for convergence             =   1.0000E-05 in
- Maximum allowable deflection                     =     100.0000 in
- Number of pile increments                        =         100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

Computational Options:
 - Use unfactored loads in computations
 - No computation of pile-head foundation stiffness matrix
 - Compute pile response under loading and nonlinear bending properties of pile
   (if nonlinear properties are specified)
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Input Data Options:
- Analysis does not use p-y modification factors (individual pile or shaft only)
- Analysis assumes zero shear resistance at the pile tip
- Analysis assumes no loading by soil movements acting on pile

Output Options:
- No p-y curves to be computed and reported for user-specified depths
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points)  = 1


-------------------------------------------------------------------------------
                    Pile Structural Properties and Geometry
-------------------------------------------------------------------------------

Total number of pile sections                       =          2

Total length of pile                                =      48.00 ft

Depth of ground surface below top of pile           =      14.00 ft

Pile diameter values used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile.

Point          Depth              Pile
                 X              Diameter
                 ft                in
-----          ---------        -----------
  1              0.00000         42.0000000
  2            14.000000         42.0000000
  3            14.000000         48.0000000
  4            48.000000         48.0000000


Input Structural Properties:
----------------------------

Pile Section No. 1:

   Section Type                                     =    Elastic Pile
   Cross-sectional Shape                            =        Circular
   Section Length                                   =     14.00000 ft
   Top Width                                        =     42.00000 in
   Bottom Width                                     =     42.00000 in
   Top Area                                         =   1385.44236 Sq. in
   Bottom Area                                      =   1385.44236 Sq. in
   Moment of Inertia at Top                         =       152745. in^4
   Moment of Inertia at Bottom                      =       152745. in^4
   Elastic Modulus                                  =      3122019. lbs/in^2
```

```
Pile Section No. 2:

   Section Type                                 =    Elastic Pile
   Cross-sectional Shape                        =        Circular
   Section Length                               =     34.00000 ft
   Top Width                                    =     48.00000 in
   Bottom Width                                 =     48.00000 in
   Top Area                                     =   1809.55737 Sq. in
   Bottom Area                                  =   1809.55737 Sq. in
   Moment of Inertia at Top                     =      260576. in^4
   Moment of Inertia at Bottom                  =      260576. in^4
   Elastic Modulus                              =     3823676. lbs/in^2


--------------------------------------------------------------------------------
                      Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------

Ground Slope Angle                              =        0.000 degrees
                                                =        0.000 radians

Pile Batter Angle                               =        0.000 degrees
                                                =        0.000 radians


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 7 layers

Layer 1 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer        =     14.00000 ft
   Distance from top of pile to bottom of layer     =     16.20000 ft
   Effective unit weight at top of layer            =     53.00000 pcf
   Effective unit weight at bottom of layer         =     53.00000 pcf
   Undrained cohesion at top of layer               =    900.00000 psf
   Undrained cohesion at bottom of layer            =    900.00000 psf
   Epsilon-50 at top of layer                       =      0.01000
   Epsilon-50 at bottom of layer                    =      0.01000
   Subgrade k at top of layer                       =    100.00000 pci
   Subgrade k at bottom of layer                    =    100.00000 pci


Layer 2 is soft clay, p-y criteria by Matlock, 1970

   Distance from top of pile to top of layer        =     16.20000 ft
   Distance from top of pile to bottom of layer     =     22.40000 ft
   Effective unit weight at top of layer            =     38.00000 pcf
   Effective unit weight at bottom of layer         =     38.00000 pcf
   Undrained cohesion at top of layer               =     50.00000 psf
   Undrained cohesion at bottom of layer            =     50.00000 psf
   Epsilon-50 at top of layer                       =      0.02000
   Epsilon-50 at bottom of layer                    =      0.02000


Layer 3 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer        =     22.40000 ft
   Distance from top of pile to bottom of layer     =     30.40000 ft
   Effective unit weight at top of layer            =     58.00000 pcf
   Effective unit weight at bottom of layer         =     58.00000 pcf
   Undrained cohesion at top of layer               =   1700.00000 psf
   Undrained cohesion at bottom of layer            =   1700.00000 psf
   Epsilon-50 at top of layer                       =      0.00700
   Epsilon-50 at bottom of layer                    =      0.00700
   Subgrade k at top of layer                       =    500.00000 pci
   Subgrade k at bottom of layer                    =    500.00000 pci
```

```
Layer 4 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer          =     30.40000 ft
   Distance from top of pile to bottom of layer       =     35.40000 ft
   Effective unit weight at top of layer              =     53.00000 pcf
   Effective unit weight at bottom of layer           =     53.00000 pcf
   Undrained cohesion at top of layer                 =    700.00000 psf
   Undrained cohesion at bottom of layer              =    700.00000 psf
   Epsilon-50 at top of layer                         =      0.01000
   Epsilon-50 at bottom of layer                      =      0.01000
   Subgrade k at top of layer                         =    100.00000 pci
   Subgrade k at bottom of layer                      =    100.00000 pci


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

   Distance from top of pile to top of layer          =     35.40000 ft
   Distance from top of pile to bottom of layer       =     43.90000 ft
   Effective unit weight at top of layer              =     58.00000 pcf
   Effective unit weight at bottom of layer           =     58.00000 pcf
   Friction angle at top of layer                     =     32.00000 deg.
   Friction angle at bottom of layer                  =     32.00000 deg.
   Subgrade k at top of layer                         =     60.00000 pci
   Subgrade k at bottom of layer                      =     60.00000 pci


Layer 6 is stiff clay with water-induced erosion

   Distance from top of pile to top of layer          =     43.90000 ft
   Distance from top of pile to bottom of layer       =     55.40000 ft
   Effective unit weight at top of layer              =     63.00000 pcf
   Effective unit weight at bottom of layer           =     63.00000 pcf
   Undrained cohesion at top of layer                 =   5000.00000 psf
   Undrained cohesion at bottom of layer              =   5000.00000 psf
   Epsilon-50 at top of layer                         =      0.00400
   Epsilon-50 at bottom of layer                      =      0.00400
   Subgrade k at top of layer                         =   2000.00000 pci
   Subgrade k at bottom of layer                      =   2000.00000 pci


Layer 7 is strong rock (vuggy limestone)

   Distance from top of pile to top of layer          =     55.40000 ft
   Distance from top of pile to bottom of layer       =     65.40000 ft
   Effective unit weight at top of layer              =    100.00000 pcf
   Effective unit weight at bottom of layer           =    100.00000 pcf
   Uniaxial compressive strength at top of layer      =   1000.00000 psi
   Uniaxial compressive strength at bottom of layer   =   1000.00000 psi


   (Depth of lowest soil layer extends   17.40 ft below pile tip)



--------------------------------------------------------------------------------
                           Summary of Soil Properties
--------------------------------------------------------------------------------

                       Layer                     Layer      Effective    Undrained    Angle of      Uniaxial
Strain
Layer                Soil Type                   Depth      Unit Wt.     Cohesion     Friction         qu
Factor        kpy
 Num.         (p-y Curve Criteria)                 ft          pcf          psf         deg.          psi
Epsilon 50      pci
-----  ---------------------------------   ----------   ----------   ----------   ----------   ----------
----------   ----------
  1     Stiff Clay with Free Water           14.000       53.000      900.000         --            --
```

```
0.01000       100.000
                                                  16.200        53.000       900.000          --            --

0.01000       100.000
  2     Soft Clay                                 16.200        38.000        50.000          --            --
0.02000        --
                                                  22.400        38.000        50.000          --            --

0.02000        --
  3     Stiff Clay with Free Water                22.400        58.000      1700.000          --            --
0.00700       500.000
                                                  30.400        58.000      1700.000          --            --

0.00700       500.000
  4     Stiff Clay with Free Water                30.400        53.000       700.000          --            --
0.01000       100.000
                                                  35.400        53.000       700.000          --            --

0.01000       100.000
  5     Sand (Reese, et al.)                      35.400        58.000         --           32.000          --            --
          60.000
                                                  43.900        58.000         --           32.000          --            --

          60.000
  6     Stiff Clay with Free Water                43.900        63.000      5000.000          --            --
0.00400      2000.000
                                                  55.400        63.000      5000.000          --            --

0.00400      2000.000
  7     Vuggy Limestone                           55.400       100.000         --             --          1000.000          --
         --
                                                  65.400       100.000         --             --          1000.000          --

         --


--------------------------------------------------------------------------------
                               Loading Type
--------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



--------------------------------------------------------------------------------
               Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 2

Load    Load        Condition              Condition             Axial Thrust         Compute
 No.    Type            1                      2                  Force, lbs     Top y vs. Pile Length
-----   ----   -------------------   ---------------------    ---------------    --------------------
   1      1    V =      30200. lbs   M =    774000. in-lbs           833000.              No
   2      2    V =      28700. lbs   S =     0.0000 in/in            809300.              No

V = perpendicular shear force applied to pile head
M = bending moment applied to pile head
y = lateral deflection relative to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applie to pile head
Axial thrust is assumed to be acting axially for all pile batter angles.


--------------------------------------------------------------------------------
    Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:
-------------------
Moment-curvature properties were derived from elastic section properties
```

```
Pile Section No. 2:
-------------------
Moment-curvature properties were derived from elastic section properties




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

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head                            =      30200.0 lbs
Applied moment at pile head                         =     774000.0 in-lbs
Axial thrust load on pile head                      =     833000.0 lbs

   Depth     Deflect.    Bending     Shear       Slope       Total     Bending    Soil Res.  Soil Spr.   Distrib.
     X          y        Moment      Force         S         Stress   Stiffness       p        Es*h     Lat. Load
    feet     inches      in-lbs       lbs       radians       psi*     lb-in^2      lb/in    lb/inch    lb/inch
---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
      0.00     0.7338    774000.     30200.  -0.003306   707.6646  4.769E+11      0.000      0.000      0.000
     0.480     0.7148    963794.     30200.  -0.003296   733.7583  4.769E+11      0.000      0.000      0.000
     0.960     0.6958   1153532.     30200.  -0.003283   759.8442  4.769E+11      0.000      0.000      0.000
     1.440     0.6770   1343203.     30200.  -0.003268   785.9210  4.769E+11      0.000      0.000      0.000
     1.920     0.6582   1532796.     30200.  -0.003251   811.9870  4.769E+11      0.000      0.000      0.000
     2.400     0.6395   1722301.     30200.  -0.003231   838.0409  4.769E+11      0.000      0.000      0.000
     2.880     0.6210   1911705.     30200.  -0.003209   864.0810  4.769E+11      0.000      0.000      0.000
     3.360     0.6025   2100999.     30200.  -0.003185   890.1059  4.769E+11      0.000      0.000      0.000
     3.840     0.5843   2290171.     30200.  -0.003158   916.1140  4.769E+11      0.000      0.000      0.000
     4.320     0.5662   2479211.     30200.  -0.003129   942.1039  4.769E+11      0.000      0.000      0.000
     4.800     0.5482   2668107.     30200.  -0.003098   968.0740  4.769E+11      0.000      0.000      0.000
     5.280     0.5305   2856848.     30200.  -0.003065   994.0229  4.769E+11      0.000      0.000      0.000
     5.760     0.5129   3045423.     30200.  -0.003029  1019.9490  4.769E+11      0.000      0.000      0.000
     6.240     0.4956   3233822.     30200.  -0.002991  1045.8509  4.769E+11      0.000      0.000      0.000
     6.720     0.4785   3422034.     30200.  -0.002951  1071.7270  4.769E+11      0.000      0.000      0.000
     7.200     0.4616   3610047.     30200.  -0.002909  1097.5758  4.769E+11      0.000      0.000      0.000
     7.680     0.4449   3797851.     30200.  -0.002864  1123.3959  4.769E+11      0.000      0.000      0.000
     8.160     0.4286   3985435.     30200.  -0.002817  1149.1857  4.769E+11      0.000      0.000      0.000
     8.640     0.4125   4172788.     30200.  -0.002768  1174.9437  4.769E+11      0.000      0.000      0.000
     9.120     0.3967   4359899.     30200.  -0.002716  1200.6685  4.769E+11      0.000      0.000      0.000
     9.600     0.3812   4546758.     30200.  -0.002662  1226.3585  4.769E+11      0.000      0.000      0.000
    10.080     0.3660   4733353.     30200.  -0.002606  1252.0124  4.769E+11      0.000      0.000      0.000
    10.560     0.3512   4919673.     30200.  -0.002548  1277.6285  4.769E+11      0.000      0.000      0.000
    11.040     0.3367   5105709.     30200.  -0.002488  1303.2054  4.769E+11      0.000      0.000      0.000
    11.520     0.3225   5291448.     30200.  -0.002425  1328.7416  4.769E+11      0.000      0.000      0.000
    12.000     0.3087   5476881.     30200.  -0.002360  1354.2357  4.769E+11      0.000      0.000      0.000
    12.480     0.2953   5661997.     30200.  -0.002292  1379.6861  4.769E+11      0.000      0.000      0.000
    12.960     0.2823   5846784.     30200.  -0.002223  1405.0914  4.769E+11      0.000      0.000      0.000
    13.440     0.2697   6031233.     30200.  -0.002151  1430.4502  4.769E+11      0.000      0.000      0.000
    13.920     0.2575   6215332.     30200.  -0.002077  1455.7608  4.769E+11      0.000      0.000      0.000
    14.400     0.2458   6399071.     29860.  -0.002021  1049.7107  9.964E+11  -117.9797  2764.8000      0.000
    14.880     0.2343   6578718.     28808.  -0.001984  1066.2569  9.964E+11  -247.3736  6082.5600      0.000
    15.360     0.2229   6749975.     27249.  -0.001945  1082.0303  9.964E+11  -293.8569  7592.2978      0.000
    15.840     0.2118   6911296.     25452.  -0.001906  1096.8885  9.964E+11  -330.1549  8976.7423      0.000
    16.320     0.2010   7061471.     24455.  -0.001865  1110.7201  9.964E+11   -16.1822   463.7652      0.000
    16.800     0.1904   7210913.     24357.  -0.001824  1124.4843  9.964E+11   -17.6032   532.6509      0.000
    17.280     0.1800   7359572.     24252.  -0.001782  1138.1763  9.964E+11   -18.9020   604.9590      0.000
    17.760     0.1698   7507399.     24140.  -0.001739  1151.7917  9.964E+11   -20.1008   681.7439      0.000
    18.240     0.1599   7654351.     24021.  -0.001695  1165.3265  9.964E+11   -21.2111   763.8946      0.000
    18.720     0.1503   7800387.     23896.  -0.001650  1178.7769  9.964E+11   -22.2392   852.2702      0.000
    19.200     0.1409   7945468.     23765.  -0.001605  1192.1394  9.964E+11   -23.1883   947.7704      0.000
    19.680     0.1318   8089560.     23629.  -0.001559  1205.4108  9.964E+11   -24.0600  1051.3828      0.000
    20.160     0.1230   8232630.     23488.  -0.001511  1218.5880  9.964E+11   -24.8548  1164.2213      0.000
    20.640     0.1144   8374646.     23343.  -0.001463  1231.6682  9.964E+11   -25.5726  1287.5649      0.000
    21.120     0.1061   8515581.     23194.  -0.001415  1244.6489  9.964E+11   -26.2127  1422.9011      0.000
```

```
     21.600     0.0981   8655411.      23044.  -0.001365  1257.5277  9.964E+11   -25.8361  1516.9144       0.000
     22.080     0.0904   8794144.      22897.  -0.001315  1270.3055  9.964E+11   -25.1400  1602.0854       0.000
     22.560     0.0830   8931798.      20342.  -0.001263  1282.9839  9.964E+11  -862.0589     59853.       0.000
     23.040     0.0758   9040604.      15348.  -0.001211  1293.0053  9.964E+11  -871.9163     66228.       0.000
     23.520     0.0690   9120231.      10310.  -0.001159  1300.3392  9.964E+11  -877.2864     73228.       0.000
     24.000     0.0625   9170498.  5254.5820  -0.001106  1304.9691  9.964E+11  -878.1441     80952.       0.000
     24.480     0.0563   9191377.   207.0892  -0.001053  1306.8920  9.964E+11  -874.4575     89521.       0.000
     24.960     0.0504   9182988. -4805.9562  -0.001000  1306.1194  9.964E+11  -866.1832     99084.       0.000
     25.440     0.0447   9145606. -9757.9542  -0.000947  1302.6764  9.964E+11  -853.2605    109834.       0.000
     25.920     0.0394   9079662.     -14622.  -0.000894  1296.6027  9.964E+11  -835.6032    122018.       0.000
     26.400     0.0344   8985743.     -19370.  -0.000842  1287.9524  9.964E+11  -813.0887    135961.       0.000
     26.880     0.0297   8864598.     -23974.  -0.000790  1276.7945  9.964E+11  -785.5409    152108.       0.000
     27.360     0.0253   8717144.     -28404.  -0.000740  1263.2136  9.964E+11  -752.7056    171083.       0.000
     27.840     0.0212   8544476.     -32629.  -0.000690  1247.3102  9.964E+11  -714.2103    193797.       0.000
     28.320     0.0174   8347876.     -36614.  -0.000641  1229.2026  9.964E+11  -669.4980    221656.       0.000
     28.800     0.0138   8128831.     -40321.  -0.000593  1209.0278  9.964E+11  -617.7079    256974.       0.000
     29.280     0.0106   7889067.     -43692.  -0.000547  1186.9446  9.964E+11  -552.6788    301333.       0.000
     29.760   0.007546   7630747.     -46629.  -0.000502  1163.1525  9.964E+11  -467.1049    356553.       0.000
     30.240   0.004781   7356718.     -49045.  -0.000459  1137.9135  9.964E+11  -371.8412    447940.       0.000
     30.720   0.002262   7070149.     -50308.  -0.000417  1111.5194  9.964E+11   -66.5745    169530.       0.000
1st  31.200  -2.212E-05   6781174.     -50498.  -0.000377  1084.9038  9.964E+11     0.6639    172848.       0.000
neg  31.680  -0.002080   6492034.     -50312.  -0.000339  1058.2729  9.964E+11    63.6273    176166.       0.000
     32.160  -0.003922   6204824.     -49795.  -0.000302  1031.8199  9.964E+11   115.9954    170334.       0.000
     32.640  -0.005558   5921291.     -49063.  -0.000267  1005.7055  9.964E+11   138.0820    143102.       0.000
     33.120  -0.006996   5642175.     -48220.  -0.000233   979.9979  9.964E+11   154.9252    127549.       0.000
     33.600  -0.008247   5368042.     -47289.  -0.000202   954.7493  9.964E+11   168.2034    117484.       0.000
     34.080  -0.009318   5099341.     -46290.  -0.000171   930.0010  9.964E+11   178.8004    110523.       0.000
     34.560    -0.0102   4836430.     -45235.  -0.000143   905.7860  9.964E+11   187.2544    105534.       0.000
     35.040    -0.0110   4579599.     -44137.  -0.000115   882.1309  9.964E+11   193.9232    101906.       0.000
     35.520    -0.0115   4329074.     -43395. -8.963E-05   859.0566  9.964E+11    63.9743     31906.       0.000
     36.000    -0.0120   4080551.     -43007. -6.532E-05   836.1668  9.964E+11    70.5797     33896.       0.000
     36.480    -0.0123   3834257.     -42583. -4.244E-05   813.4823  9.964E+11    76.6456     35887.       0.000
max  36.960    -0.0125   3590400.     -42126. -2.098E-05   791.0221  9.964E+11    82.0849     37878.       0.000
neg  37.440    -0.0125   3349166.     -41640. -9.216E-07   768.8037  9.964E+11    86.8215     39868.       0.000
     37.920    -0.0125   3110720.     -41128.  1.775E-05   746.8419  9.964E+11    90.7900     41859.       0.000
     38.400    -0.0123   2875200.     -40596.  3.505E-05   725.1497  9.964E+11    93.9349     43850.       0.000
     38.880    -0.0121   2642716.     -40048.  5.100E-05   703.7371  9.964E+11    96.2115     45840.       0.000
     39.360    -0.0118   2413352.     -39490.  6.562E-05   682.6118  9.964E+11    97.5847     47831.       0.000
     39.840    -0.0113   2187158.     -38927.  7.892E-05   661.7786  9.964E+11    98.0293     49822.       0.000
     40.320    -0.0108   1964156.     -38364.  9.091E-05   641.2393  9.964E+11    97.5298     51812.       0.000
     40.800    -0.0103   1744335.     -37806.   0.000102   620.9930  9.964E+11    96.0799     53803.       0.000
     41.280  -0.009672   1527653.     -37260.   0.000111   601.0359  9.964E+11    93.6829     55794.       0.000
     41.760  -0.009006   1314038.     -36730.   0.000119   581.3611  9.964E+11    90.3509     57784.       0.000
     42.240  -0.008297   1103383.     -36221.   0.000126   561.9591  9.964E+11    86.1049     59775.       0.000
     42.720  -0.007551    895554.     -35740.   0.000132   542.8173  9.964E+11    80.9748     61766.       0.000
     43.200  -0.006776    690388.     -35291.   0.000137   523.9207  9.964E+11    74.9992     63756.       0.000
     43.680  -0.005977    487690.     -34879.   0.000140   505.2516  9.964E+11    68.2249     65747.       0.000
     44.160  -0.005162    287243.     -30353.   0.000142   486.7896  9.964E+11  1503.0507   1677100.       0.000
     44.640  -0.004338    136655.     -22056.   0.000144   472.9200  9.964E+11  1377.8110   1829547.       0.000
     45.120  -0.003509     31775.     -14519.   0.000144   463.2602  9.964E+11  1239.1824   2034224.       0.000
     45.600  -0.002679    -31992. -7832.3431   0.000144   463.2801  9.964E+11  1082.7372   2328157.       0.000
     46.080  -0.001850    -59835. -2122.8079   0.000144   465.8446  9.964E+11   899.7403   2801694.       0.000
     46.560  -0.001023    -57826.  2099.0643   0.000143   465.6595  9.964E+11   566.1875   3188582.       0.000
     47.040  -0.000198    -37030.  4051.4746   0.000143   463.7442  9.964E+11   111.7327   3254937.       0.000
     47.520   0.000626    -12526.  3333.5254   0.000143   461.4873  9.964E+11  -361.0206   3321292.       0.000
min  48.000   0.001450      0.000      0.000   0.000143   460.3336  9.964E+11  -796.4535   1582448.       0.000
tip
 * The above values of total stress are combined axial and bending stresses.


 Output Summary for Load Case No. 1:

 Pile-head deflection             =      0.7337992 inches
 Computed slope at pile head      =     -0.0033064 radians
 Maximum bending moment           =       9191377. inch-lbs
 Maximum shear force              =        -50498. lbs
 Depth of maximum bending moment  =     24.4800000 feet below pile head
 Depth of maximum shear force     =     31.2000000 feet below pile head
 Number of iterations             =             14
```

```
Number of zero deflection points =             2



--------------------------------------------------------------------------------
                 Computed Values of Pile Loading and Deflection
                   for Lateral Loading for Load Case Number 2
--------------------------------------------------------------------------------

Pile-head conditions are Shear and Pile-head Rotation (Loading Type 2)

Shear force at pile head                             =      28700.0 lbs
Rotation of pile head                                =    0.000E+00 radians
Axial load at pile head                              =     809300.0 lbs

(Zero slope for this load indicates fixed-head conditions)

   Depth    Deflect.    Bending     Shear       Slope       Total     Bending   Soil Res.  Soil Spr.   Distrib.
     X         y        Moment      Force         S        Stress    Stiffness      p         Es*h    Lat. Load
    feet     inches     in-lbs       lbs       radians      psi*      lb-in^2     lb/in     lb/inch    lb/inch
 ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
       0.00     0.1820   -4428647.     28700.      0.000  1193.0138  4.769E+11      0.000      0.000      0.000
      0.480     0.1819   -4263210.     28700. -5.249E-05  1170.2689  4.769E+11      0.000      0.000      0.000
      0.960     0.1814   -4097534.     28700.  -0.000103  1147.4910  4.769E+11      0.000      0.000      0.000
      1.440     0.1807   -3931626.     28700.  -0.000151  1124.6813  4.769E+11      0.000      0.000      0.000
      1.920     0.1797   -3765497.     28700.  -0.000198  1101.8413  4.769E+11      0.000      0.000      0.000
      2.400     0.1784   -3599157.     28700.  -0.000242  1078.9721  4.769E+11      0.000      0.000      0.000
      2.880     0.1769   -3432613.     28700.  -0.000285  1056.0750  4.769E+11      0.000      0.000      0.000
      3.360     0.1751   -3265876.     28700.  -0.000325  1033.1514  4.769E+11      0.000      0.000      0.000
      3.840     0.1731   -3098956.     28700.  -0.000364  1010.2025  4.769E+11      0.000      0.000      0.000
      4.320     0.1709   -2931861.     28700.  -0.000400   987.2295  4.769E+11      0.000      0.000      0.000
      4.800     0.1685   -2764600.     28700.  -0.000435   964.2339  4.769E+11      0.000      0.000      0.000
      5.280     0.1659   -2597184.     28700.  -0.000467   941.2169  4.769E+11      0.000      0.000      0.000
      5.760     0.1631   -2429622.     28700.  -0.000497   918.1798  4.769E+11      0.000      0.000      0.000
      6.240     0.1602   -2261923.     28700.  -0.000526   895.1239  4.769E+11      0.000      0.000      0.000
      6.720     0.1571   -2094097.     28700.  -0.000552   872.0505  4.769E+11      0.000      0.000      0.000
      7.200     0.1538   -1926153.     28700.  -0.000576   848.9608  4.769E+11      0.000      0.000      0.000
      7.680     0.1504   -1758100.     28700.  -0.000599   825.8563  4.769E+11      0.000      0.000      0.000
      8.160     0.1469   -1589949.     28700.  -0.000619   802.7381  4.769E+11      0.000      0.000      0.000
      8.640     0.1433   -1421708.     28700.  -0.000637   779.6076  4.769E+11      0.000      0.000      0.000
      9.120     0.1396   -1253386.     28700.  -0.000653   756.4662  4.769E+11      0.000      0.000      0.000
      9.600     0.1358   -1084995.     28700.  -0.000667   733.3150  4.769E+11      0.000      0.000      0.000
     10.080     0.1319    -916542.     28700.  -0.000679   710.1554  4.769E+11      0.000      0.000      0.000
     10.560     0.1280    -748037.     28700.  -0.000689   686.9887  4.769E+11      0.000      0.000      0.000
     11.040     0.1240    -579491.     28700.  -0.000697   663.8163  4.769E+11      0.000      0.000      0.000
     11.520     0.1199    -410911.     28700.  -0.000703   640.6393  4.769E+11      0.000      0.000      0.000
     12.000     0.1159    -242309.     28700.  -0.000707   617.4592  4.769E+11      0.000      0.000      0.000
     12.480     0.1118     -73693.     28700.  -0.000709   594.2772  4.769E+11      0.000      0.000      0.000
     12.960     0.1077      94927.     28700.  -0.000709   597.1965  4.769E+11      0.000      0.000      0.000
     13.440     0.1036     263542.     28700.  -0.000707   620.3783  4.769E+11      0.000      0.000      0.000
     13.920     0.0996     432142.     28700.  -0.000703   643.5582  4.769E+11      0.000      0.000      0.000
     14.400     0.0955     600717.     28568.  -0.000698   502.5647  9.964E+11   -45.8502  2764.8000      0.000
     14.880     0.0915     767756.     28158.  -0.000694   517.9495  9.964E+11   -96.6330  6082.5600      0.000
     15.360     0.0875     931567.     27468.  -0.000690   533.0371  9.964E+11  -142.8350  9400.3200      0.000
     15.840     0.0836    1090615.     26525.  -0.000684   547.6859  9.964E+11  -184.5124     12718.      0.000
     16.320     0.0796    1243511.     25960.  -0.000677   561.7682  9.964E+11   -11.8866   859.6434      0.000
     16.800     0.0758    1395980.     25888.  -0.000669   575.8111  9.964E+11   -12.9494   984.4397      0.000
     17.280     0.0719    1547981.     25811.  -0.000661   589.8110  9.964E+11   -13.9243  1114.9462      0.000
     17.760     0.0682    1699478.     25728.  -0.000651   603.7644  9.964E+11   -14.8273  1253.1025      0.000
     18.240     0.0644    1850438.     25640.  -0.000641   617.6684  9.964E+11   -15.6663  1400.5303      0.000
     18.720     0.0608    2000828.     25548.  -0.000630   631.5199  9.964E+11   -16.4456  1558.7876      0.000
     19.200     0.0572    2150619.     25451.  -0.000618   645.3161  9.964E+11   -17.1670  1729.5006      0.000
     19.680     0.0536    2299782.     25350.  -0.000605   659.0546  9.964E+11   -17.8316  1914.4500      0.000
     20.160     0.0502    2448292.     25245.  -0.000591   672.7328  9.964E+11   -18.4394  2115.6415      0.000
     20.640     0.0468    2596124.     25138.  -0.000577   686.3487  9.964E+11   -18.9899  2335.3756      0.000
     21.120     0.0436    2743255.     25027.  -0.000561   699.9000  9.964E+11   -19.4824  2576.3226      0.000
     21.600     0.0404    2889667.     24915.  -0.000545   713.3850  9.964E+11   -19.2182  2742.0794      0.000
     22.080     0.0373    3035363.     24806.  -0.000528   726.8042  9.964E+11   -18.7146  2891.6849      0.000
     22.560     0.0343    3180356.     23156.  -0.000510   740.1586  9.964E+11  -554.2316     93107.      0.000
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       23.040      0.0314   3306876.      19944.  -0.000491   751.8115  9.964E+11  -561.1176     102923.       0.000
       23.520      0.0286   3414690.      16700.  -0.000472   761.7415  9.964E+11  -565.0926     113698.       0.000
       24.000      0.0260   3503663.      13442.  -0.000452   769.9363  9.964E+11  -566.1411     125582.       0.000
       24.480      0.0234   3573759.      10187.  -0.000431   776.3923  9.964E+11  -564.2459     138758.       0.000
       24.960      0.0210   3625038.  6950.8448  -0.000411   781.1153  9.964E+11  -559.3860     153452.       0.000
       25.440      0.0187   3657661.  3751.3991  -0.000390   784.1200  9.964E+11  -551.5327     169950.       0.000
       25.920      0.0165   3671886.   605.9277  -0.000368   785.4302  9.964E+11  -540.6449     188621.       0.000
       26.400      0.0144   3668075. -2467.9193  -0.000347   785.0792  9.964E+11  -526.6631     209945.       0.000
       26.880      0.0125   3646692. -5452.0668  -0.000326   783.1097  9.964E+11  -509.4993     234570.       0.000
       27.360      0.0107   3608306. -8327.8126  -0.000305   779.5743  9.964E+11  -489.0236     263395.       0.000
       27.840    0.008997   3553599.     -11076.  -0.000284   774.5355  9.964E+11  -465.0422     297715.       0.000
       28.320    0.007419   3483367.     -13674.  -0.000264   768.0669  9.964E+11  -437.2617     339488.       0.000
       28.800    0.005956   3398534.     -16101.  -0.000244   760.2535  9.964E+11  -405.2270     391862.       0.000
       29.280    0.004607   3300164.     -18251.  -0.000225   751.1933  9.964E+11  -341.3893     426814.       0.000
       29.760    0.003368   3190380.     -19981.  -0.000206   741.0818  9.964E+11  -259.2496     443403.       0.000
       30.240    0.002235   3071908.     -21241.  -0.000188   730.1701  9.964E+11  -178.4549     459992.       0.000
       30.720    0.001204   2947432.     -21857.  -0.000170   718.7054  9.964E+11   -35.4288     169530.       0.000
       31.200    0.000271   2821702.     -21983.  -0.000154   707.1252  9.964E+11    -8.1327     172848.       0.000
1st    31.680   -0.000568   2695626.     -21956.  -0.000138   695.5132  9.964E+11    17.3643     176166.       0.000
neg    32.160   -0.001317   2570053.     -21788.  -0.000123   683.9475  9.964E+11    41.0305     179483.       0.000
       32.640   -0.001980   2445773.     -21489.  -0.000108   672.5008  9.964E+11    62.8435     182801.       0.000
       33.120   -0.002562   2323511.     -21069. -9.432E-05   661.2401  9.964E+11    82.7893     186119.       0.000
       33.600   -0.003067   2203933.     -20540. -8.124E-05   650.2266  9.964E+11   100.8609     189437.       0.000
       34.080   -0.003498   2087643.     -19934. -6.883E-05   639.5158  9.964E+11   109.5326     180363.       0.000
       34.560   -0.003860   1974930.     -19288. -5.709E-05   629.1345  9.964E+11   115.0599     171709.       0.000
       35.040   -0.004156   1865982.     -18612. -4.599E-05   619.1000  9.964E+11   119.3924     165486.       0.000
       35.520   -0.004389   1760944.     -18199. -3.550E-05   609.4256  9.964E+11    24.3140      31906.       0.000
       36.000   -0.004565   1656665.     -18051. -2.562E-05   599.8212  9.964E+11    26.8619      33896.       0.000
       36.480   -0.004685   1553234.     -17890. -1.634E-05   590.2948  9.964E+11    29.1871      35887.       0.000
       36.960   -0.004753   1450728.     -17716. -7.662E-06   580.8537  9.964E+11    31.2551      37878.       0.000
max    37.440   -0.004773   1349221.     -17530.  4.315E-07   571.5045  9.964E+11    33.0360      39868.       0.000
neg    37.920   -0.004748   1248773.     -17336.  7.941E-06   562.2529  9.964E+11    34.5042      41859.       0.000
       38.400   -0.004681   1149436.     -17134.  1.487E-05   553.1036  9.964E+11    35.6386      43850.       0.000
       38.880   -0.004577   1051251.     -16926.  2.123E-05   544.0604  9.964E+11    36.4224      45840.       0.000
       39.360   -0.004437    954246.     -16715.  2.703E-05   535.1259  9.964E+11    36.8431      47831.       0.000
       39.840   -0.004265    858437.     -16503.  3.227E-05   526.3015  9.964E+11    36.8923      49822.       0.000
       40.320   -0.004065    763829.     -16292.  3.696E-05   517.5878  9.964E+11    36.5658      51812.       0.000
       40.800   -0.003839    670414.     -16083.  4.111E-05   508.9840  9.964E+11    35.8633      53803.       0.000
       41.280   -0.003591    578171.     -15879.  4.471E-05   500.4880  9.964E+11    34.7887      55794.       0.000
       41.760   -0.003324    487066.     -15683.  4.779E-05   492.0970  9.964E+11    33.3495      57784.       0.000
       42.240   -0.003041    397055.     -15496.  5.035E-05   483.8066  9.964E+11    31.5573      59775.       0.000
       42.720   -0.002744    308080.     -15321.  5.239E-05   475.6117  9.964E+11    29.4275      61766.       0.000
       43.200   -0.002437    220073.     -15158.  5.391E-05   467.5059  9.964E+11    26.9791      63756.       0.000
       43.680   -0.002123    132955.     -15011.  5.493E-05   459.4821  9.964E+11    24.2350      65747.       0.000
       44.160   -0.001805     46637.     -12382.  5.545E-05   451.5319  9.964E+11   888.6595    2836526.       0.000
       44.640   -0.001484    -10198. -7652.6861  5.556E-05   448.1757  9.964E+11   753.3069    2923161.       0.000
       45.120   -0.001165    -42039. -3742.4908  5.541E-05   451.1084  9.964E+11   604.3998    2989516.       0.000
       45.600   -0.000846    -53828.  -709.0849  5.513E-05   452.1942  9.964E+11   448.8661    3055871.       0.000
       46.080   -0.000529    -50722.  1410.1140  5.483E-05   451.9081  9.964E+11   286.9668    3122226.       0.000
       46.560   -0.000214    -38094.  2578.4538  5.457E-05   450.7451  9.964E+11   118.7067    3188582.       0.000
       47.040   9.926E-05    -21527.  2758.7800  5.440E-05   449.2192  9.964E+11   -56.0935    3254937.       0.000
       47.520    0.000412 -6820.3526  1912.6316  5.432E-05   447.8646  9.964E+11  -237.7080    3321292.       0.000
min    48.000    0.000725      0.000      0.000  5.430E-05   447.2364  9.964E+11  -426.4002    1693824.       0.000
Tip
    * The above values of total stress are combined axial and bending stresses.


  Output Summary for Load Case No. 2:

  Pile-head deflection             =      0.1820212 inches
  Computed slope at pile head      =       0.000000 radians
  Maximum bending moment           =      -4428647. inch-lbs
  Maximum shear force              =         28700. lbs
  Depth of maximum bending moment  =       0.000000 feet below pile head
  Depth of maximum shear force     =     11.5200000 feet below pile head
  Number of iterations             =             10
  Number of zero deflection points =              2
```

```
--------------------------------------------------------------------------------
                           Summary of Pile Response(s)
--------------------------------------------------------------------------------


Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs
Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian
Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians


               Pile-head      Pile-head                                   Maximum        Maximum

Load  Load    Condition 1    Condition 2        Axial        Pile-head    Moment          Shear         Pile-head

Case  Type    V(lbs) or     in-lb, rad.,       Loading      Deflection    in Pile        in Pile        Rotation

 No.   No.    y(inches)     or in-lb/rad.        lbs          inches      in-lbs           lbs          radians

----  ----  --------------  --------------  -------------  -------------  -------------  -------------
-------------
  1     1   V =     30200.  M =    774000.        833000.     0.73379922       9191377.        -50498.
-0.00330638
  2     2   V =     28700.  S =      0.000        809300.     0.18202120      -4428647.         28700.
-0.00000000

The analysis ended normally.
```

APPENDIX D

LABORATORY RESULTS

Unconfined Compression Test

Test Data Sheet

Project: Bridge 108 on Lizzie Mill Rd over I-95
TIP No. : I-5786

Boring No.: RS-4
Sample ID: B1-B
Depth, ft.: 49.7 - 50.0

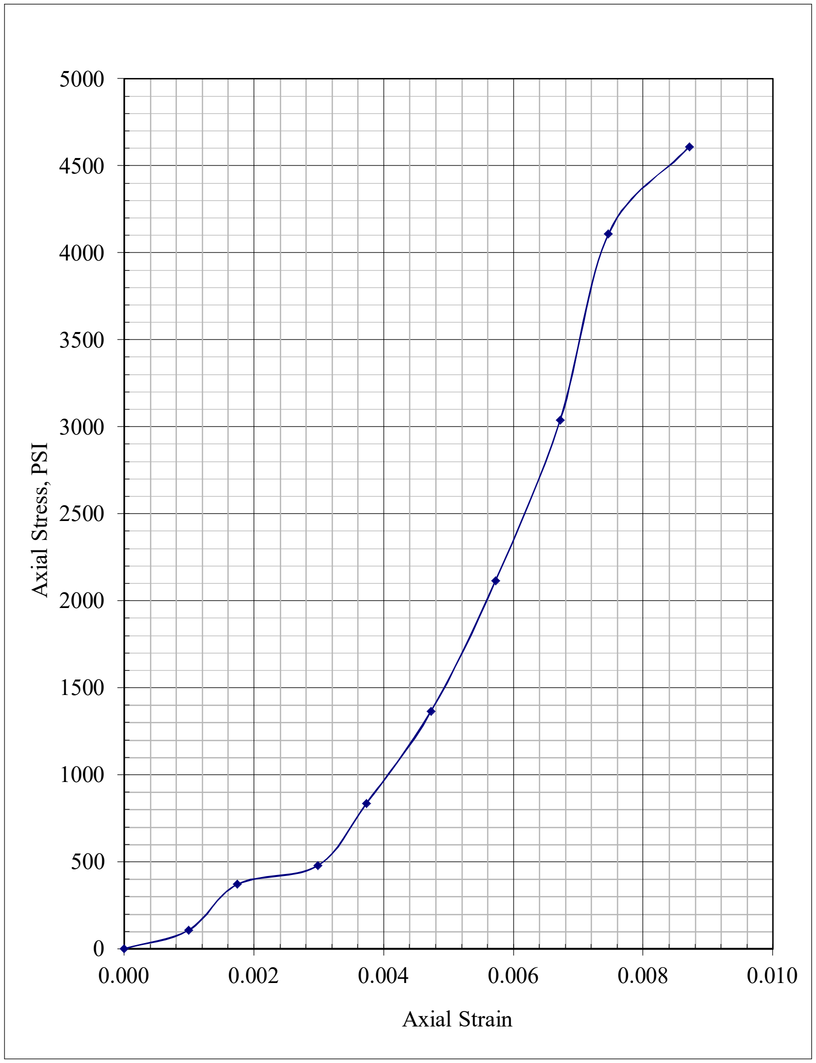
Specimen Description: Meta-Argillite

Specimen Conditions	
Diameter (in.)	1.77
Height (in.)	4.02
Area (in^2)	2.45
Unit Wt. (pcf)	163.9

Shear Testing Conditions	
Loading Rate (%/min):	0.02 in/min.

Youngs Modulus (ave., ksi):	**724**

Reading No.	Dial Guage Reading (in.)	Axial Load (lbs)*	Total Axial Deformation (in.)	Axial Strain	Corrected Area^1 (in^2)	Axial Stress (psi)	Axial Stress (Kpa)
1	0.000	0	0.000	0.0000	2.45	0.00	0.00
2	0.004	264	0.004	0.0010	2.45	107.55	741.54
3	0.007	911	0.007	0.0017	2.45	371.13	2558.86
4	0.012	1176	0.012	0.0030	2.45	479.08	3303.16
5	0.015	2049	0.015	0.0037	2.45	834.72	5755.21
6	0.019	3347	0.019	0.0047	2.45	1363.49	9400.93
7	0.023	5189	0.023	0.0057	2.45	2113.86	14574.53
8	0.027	7455	0.027	0.0067	2.45	3036.93	20938.91
9	0.030	10087	0.030	0.0075	2.45	4109.09	28331.22
10	0.035	11313	0.035	0.0087	2.45	4608.47	31774.27



LAB Technecian: Saja Alkhafaji

S Alkhafaji

Notes: 1. Right Cylinder Correction Method 2. *Specimen failed violently resulting in complete destruction of sample

Unconfined Compression Test

Test Data Sheet

Project: Bridge 108 on Lizzie Mill Rd over I-95
TIP No. : I-5786

Boring No.: RS-3
Sample ID: B1-B
Depth, ft.: 44.3 - 44.6

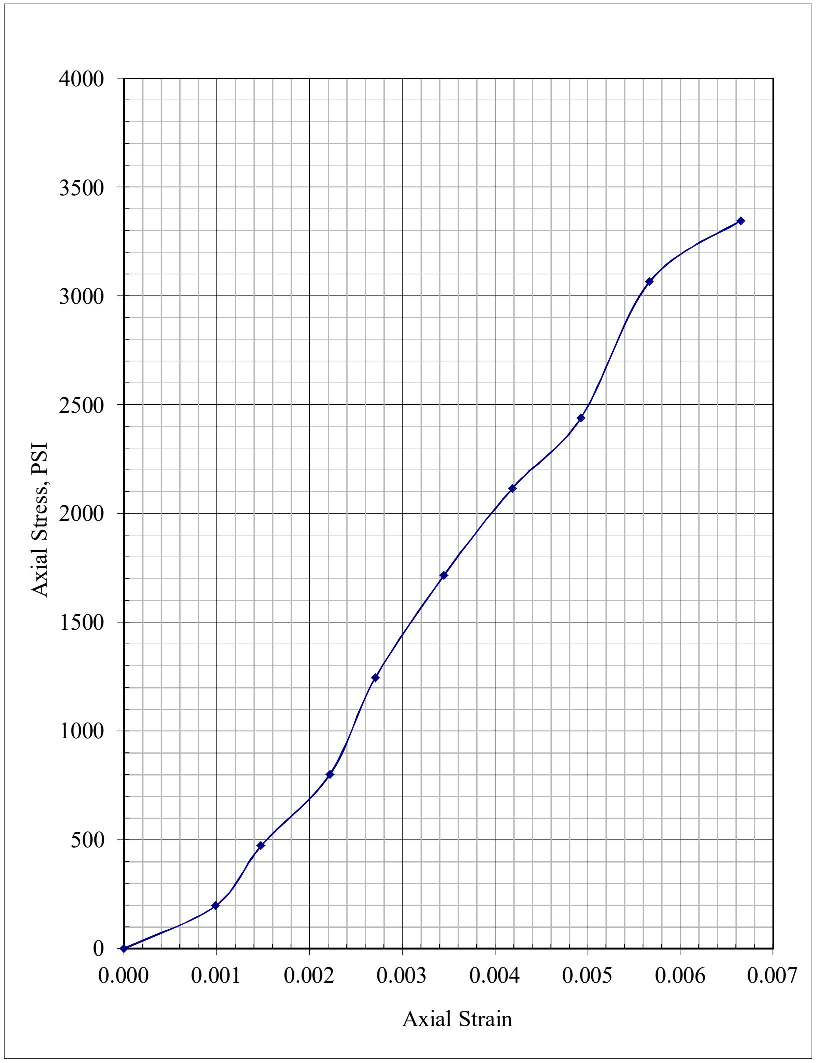
Specimen Description: Meta-Argillite

Specimen Conditions	
Diameter (in.)	1.77
Height (in.)	4.06
Area (in^2)	2.48
Unit Wt. (pcf)	162.2

Shear Testing Conditions	
Loading Rate (%/min):	0.02 in/min.

Youngs Modulus (ave., ksi):	**500**

Reading No.	Dial Guage Reading (in.)	Axial Load (lbs)*	Total Axial Deformation (in.)	Axial Strain	Corrected Area^1 (in^2)	Axial Stress (psi)	Axial Stress (Kpa)
1	0.000	0	0.000	0.0000	2.48	0.00	0.00
2	0.004	487	0.004	0.0010	2.48	196.73	1356.38
3	0.006	1171	0.006	0.0015	2.48	473.03	3261.43
4	0.009	1979	0.009	0.0022	2.48	799.42	5511.81
5	0.011	3084	0.011	0.0027	2.48	1245.78	8589.35
6	0.014	4246	0.014	0.0034	2.48	1715.16	11825.59
7	0.017	5238	0.017	0.0042	2.48	2115.86	14588.31
8	0.020	6039	0.020	0.0049	2.48	2439.40	16819.05
9	0.023	7588	0.023	0.0057	2.48	3065.08	21132.97
10	0.027	8285	0.027	0.0067	2.48	3346.59	23073.92



LAB Technecian: Saja Alkhafaji

S Alkhafaji

Notes: 1. Right Cylinder Correction Method 2. *Specimen failed violently resulting in complete destruction of sample

Unconfined Compression Test

Test Data Sheet

Project: Bridge 108 on Lizzie Mill Rd over I-95
TIP No. : I-5786

Boring No.: RS-2
Sample ID: B1-A
Depth, ft.: 70.5 -70.9

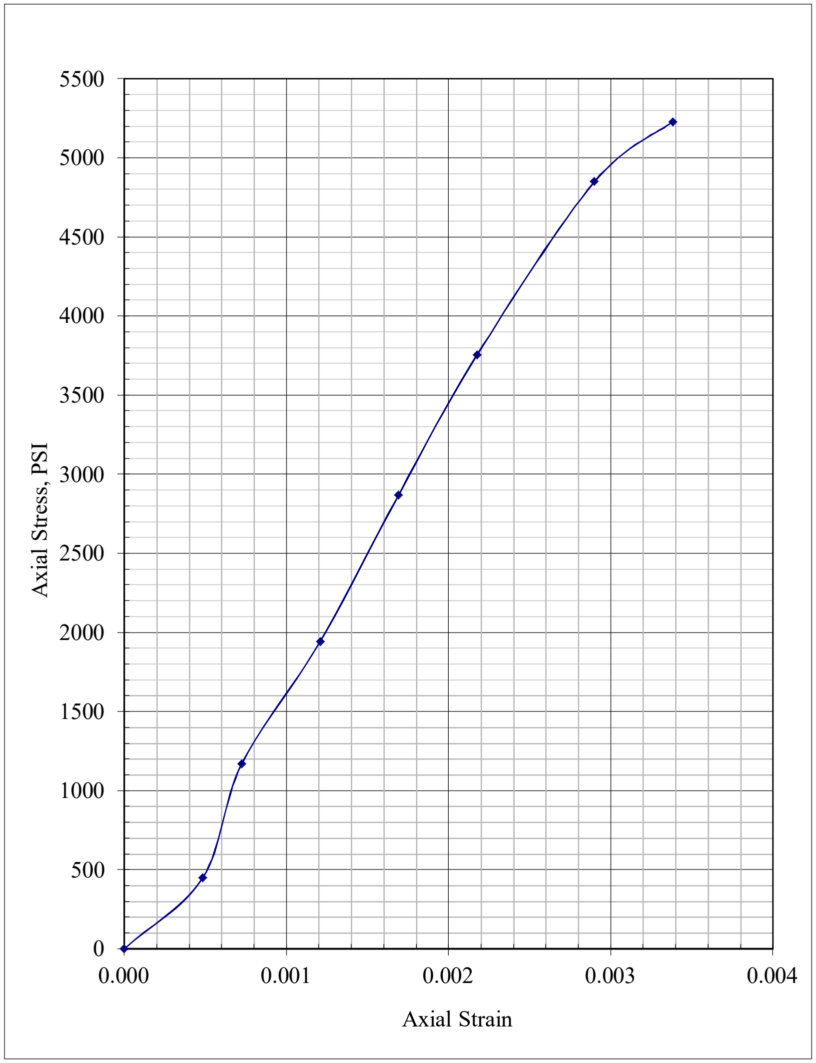
Specimen Description: Meta-Argillite

Specimen Conditions	
Diameter (in.)	1.77
Height (in.)	4.14
Area (in^2)	2.46
Unit Wt. (pcf)	158.7

Shear Testing Conditions	
Loading Rate (%/min):	0.02 in/min.

Youngs Modulus (ave., ksi):	**1,537**

Reading No.	Dial Guage Reading (in.)	Axial Load (lbs)*	Total Axial Deformation (in.)	Axial Strain	Corrected Area[1] (in^2)	Axial Stress (psi)	Axial Stress (Kpa)
1	0.000	0	0.000	0.0000	2.46	0.00	0.00
2	0.002	1103	0.002	0.0005	2.46	448.34	3091.20
3	0.003	2879	0.003	0.0007	2.46	1170.23	8068.48
4	0.005	4775	0.005	0.0012	2.46	1940.90	13382.01
5	0.007	7060	0.007	0.0017	2.46	2869.67	19785.67
6	0.009	9232	0.009	0.0022	2.46	3752.50	25872.58
7	0.012	11930	0.012	0.0029	2.46	4849.11	33433.45
8	0.014	12859	0.014	0.0034	2.46	5226.69	36036.77



Notes: 1. Right Cylinder Correction Method 2. *Specimen failed violently resulting in complete destruction of sample

Unconfined Compression Test

Test Data Sheet

Project: Bridge 108 on Lizzie Mill Rd over I-95
TIP No. : I-5786

Boring No.: RS-1
Sample ID: B1-A
Depth, ft.: 62.6 - 62.9

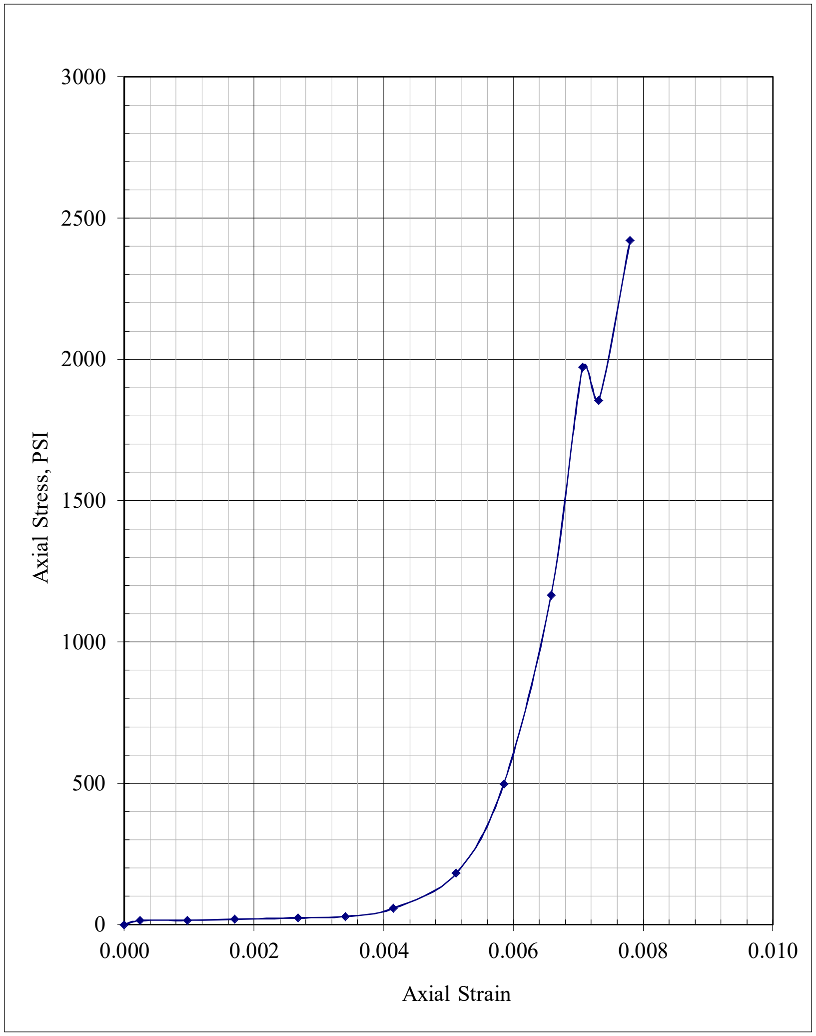
Specimen Description: Meta-Argillite

cimen Conditions	
Diameter (in.)	1.78
Height (in.)	4.10
Area (in^2)	2.47
Unit Wt. (pcf)	163.0

Shear Testing Conditions	
Loading Rate (%/min):	0.02 in/min.

Youngs Modulus (ave., ksi):	**830**

Reading No.	Dial Guage Reading (in.)	Axial Load (lbs)*	Total Axial Deformation (in.)	Axial Strain	Corrected Area^1 (in^2)	Axial Stress (psi)	Axial Stress (Kpa)
1	0.000	0	0.000	0.0000	2.47	0.00	0.00
2	0.001	36	0.001	0.0002	2.47	14.55	100.35
3	0.004	38	0.004	0.0010	2.47	15.36	105.93
4	0.007	47	0.007	0.0017	2.47	19.00	131.01
5	0.011	59	0.011	0.0027	2.47	23.85	164.46
6	0.014	72	0.014	0.0034	2.47	29.11	200.70
7	0.017	142	0.017	0.0041	2.47	57.41	395.82
8	0.021	449	0.021	0.0051	2.47	181.52	1251.55
9	0.024	1233	0.024	0.0059	2.47	498.48	3436.87
10	0.027	2884	0.027	0.0066	2.47	1165.93	8038.81
11	0.029	4876	0.029	0.0071	2.47	1971.24	13591.20
12	0.030	4590	0.030	0.0073	2.47	1855.61	12793.98
13	0.032	5990	0.032	0.0078	2.47	2421.58	16696.21



Notes: 1. Right Cylinder Correction Method 2. *Specimen failed violently resulting in complete destruction of sample

HQ: 3015 DUMBARTON ROAD RICHMOND, VIRGINIA 23228 T 804.264.2701 F 804.264.1202 www.fandr.com

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