FINAL REPORT
STRUCTURE FOUNDATION EXPLORATION
BRIDGE – FRA-71-0153 OVER BIG DARBY CREEK
FRA-71-0.00 IMPROVEMENTS
FRANKLIN COUNTY, OHIO
PID#: 93496

For:

Mead & Hunt, Inc.
5900 Wilcox Place
Dublin, OH 43016

Submitted by:

NEAS Inc.
National Engineering & Architectural Services Inc.

2800 Corporate Exchange Drive, Suite 240
Columbus, OH 43231

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EXECUTIVE SUMMARY

This report presents the results of a structure foundation exploration for replacing bridges FRA-71-0153 L&R carrying Interstate 71 (IR-71) over Big Darby Creek in Franklin County. This is a component of the larger IR-71 widening program, FRA-71-0.00, that includes ~6 miles of roadway and widening two additional bridges.

The original design concept for the project involved widening the twin structures to the inside, and geotechnical design recommendations were made on that basis in the draft report dated May 25, 2014. Subsequently, the decision to replace both bridges was made, creating the need for a revised report that reflects the use of Load and Resistance Factor Design (LRFD) methodology and the specific design details of the selected bridge type. The bridge will be designed in accordance with the LRFD method as set forth in the American Association of State Highway and Transportation Officials (AASHTO) Publication *Bridge Design Specifications, 7th Edition (with 2015 Interim Revisions)* (AASHTO, 2014) and ODOT *Bridge Design Manual*, [ODOT, 2007 (revised 2014)].

The site is located in the Darby Plain portion of the Southern Ohio Loamy Till Plain, part of the Central Lowlands, which is characterized by hummocky ground moraine of moderate relief and poorly drained swales that previously held wet prairies/meadows, and a few large streams. Bedrock is mapped as Silurian Salina undifferentiated dolomite.

Subsurface conditions were characterized on the basis of 12 historical boring logs, 2 of which were in the creek, and the results of two new borings drilled to depths of 62 and 72.4 feet (ft). The geotechnical conditions at the bridge site are good with bedrock encountered at relatively shallow depth.

Abutments will be supported on steel H-piles driven to bedrock, and the piers will be supported on drilled shafts socketed in bedrock.

1. INTRODUCTION

1.1. General

This report presents the results of a structure foundation exploration for replacement of bridges FRA-71-0153 L&R carrying IR-71 over Big Darby Creek in Franklin Count, a component of the larger IR-70 widening program, FRA-71-0.00, that includes ~6 miles of roadway and widening two additional bridges.

The exploration was conducted in general accordance with National Engineering & Architectural Services, Inc.'s (NEAS)[1] proposals to Mead & Hunt, Inc. dated October 9, 2013 and December 2014, and ODOT *Specifications for Geotechnical Explorations, 2014* (ODOT, 2014). The bridge will be designed in accordance with the LRFD method as set forth in AASHTO's Publication *Bridge Design Specifications, 7th Edition (with 2015 Interim Revisions)* (AASHTO, 2014) and ODOT *Bridge Design Manual*, [ODOT, 2007 (revised 2014)].

1.2. Proposed Construction

The existing FRA-71-0153 (L&R) structures are 3-span, continuous steel beam bridges with reinforced concrete decks and an overall length of about 330 ft. The abutments are supported on 10 BP 42 steel piles. The piers are on spread footings founded on rock and designed for a maximum bearing pressure of 10.0 tons per sq ft.

The original design concept for the project involved widening the existing twin structures to the inside, and geotechnical design recommendations were made on that basis in the draft report dated May 25, 2014. Subsequently, the decision to replace both bridges was made and two-span structures, pile-supported with semi-integral abutments and drilled shaft supported piers were selected.

Both bridges will be replaced with wider structures. The abutments will be supported on driven HP 12x53 piles, and piers will each consist of five, 48-inch-diameter columns, supported on individual drilled shafts. Drilled shafts will be 54-inches in diameter, reducing to 48-inches where they become socketed into bedrock.

[1] On October 19, 2014 Barr & Prevost Inc. (B&P) separated into two entities; Barr Engineering Inc. (BEI), the predecessor company to B&P, and Barr & Prevost, a JMT Division. BEI has retained the geotechnical exploration services for the FRA-71-0.00 project as a subcontractor to Barr & Prevost/JMT. On November 23, 2016, BEI was renamed to National Engineering & Architectures, Inc (NEAS).

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiology

The site is located in the Darby Plain portion of the Southern Ohio Loamy Till Plain, part of the Central Lowlands (Brockman, 1998). The area is characterized by hummocky ground moraine of moderate relief and poorly drained swales, which previously held wet prairies/meadows, and a few large streams. Over 150 ft of surficial unconsolidated material is mapped at the bridge site, reportedly consisting of alluvium, sand and gravel, till [up to 50 feet (ft)] and up to 90 ft of Teay's-age valley clay overlying Silurian-age carbonate rocks (dolomite) (Brockman et. al., 2005 and Shrake, 1994). While this profile is likely to be present in nearby areas, the thickness of unconsolidated material is clearly overstated at the bridge site, as discussed below.

Big Darby Creek (BDC) is at about elevation 780 mean sea level (msl) at the bridge site and its floodplain (~el 790 ft) is approximately 6,000 ft wide at this location. From the edges of the floodplain the terrain rises quickly to 850 ft, the general level of the adjacent Darby Plain (US Department of the Interior, 1966).

Bedrock is mapped as Silurian Salina undifferentiated dolomite present below elevation 700 ft in the vicinity of the bridge site (ODNR, Shrake, 1993). However, as indicated in Section 3, bedrock was actually encountered at an average elevation of 765 ft at the bridge site by The H. C. Nutting Company in 1962, and confirmed during this exploration. This suggests, based on the geological mapping, that bedrock is likely to be Devonian Age Columbus Limestone.

The Silurian/Devonian carbonate bedrock is shown as karst geology on ODNR's map *Known and Probable Karst in Ohio*, (ODNR, 2012). The map also indicates that the carbonate rock is overlain by more than 20 feet of glacial drift and/or alluvium, and no known or probable karst features are depicted.

The Scioto and Olentangy River Gorges (Hull, 1999) is a known karst region north of Columbus. These rivers both occupy steep-sided, shallow gorges incised 50-100 ft into limestone bedrock. The uplands adjacent to these gorges include areas of well-developed, active karst. To the north, in Delaware County, sinkhole concentrations of up to one per acre are not uncommon with sinkhole diameters of 10-100 ft. Evidence of karst in Franklin County is not widespread (ODNR, 1999[1] and 2007). However, given the geological settling, the presence of paleo-karst features (solution voids) in bedrock cannot be ruled out.

2.2. Geomorphology

The recent geomorphology of the creek channel is of interest when considering likely future changes that might affect the bridge during its design life. The portion of the BDC floodplain extending ~3,500 ft to the east, shows evidence of relic ox-bows and abandoned channels on satellite photography that indicate a continuous process of migration across it by the main channel. In 1962 when the original design drawings were produced, the low flow channel was on the order of 30 ft in width (based on the 776 ft contour) with a broader (~60 ft) channel suggested by the 778 ft contour, located within a meander. Construction of the bridge included a substantial excavation (up to about 6 ft in depth) of the floodplain creating a 250 ft wide channel at elevation 778 ft.

At the completion of construction, the flow in BDC emerged from a narrow channel upstream from the bridge into a low velocity pool ~ 4 times the width and 300 ft long. It is inevitable that this would cause bed load deposition, and that is apparently what has happened. A large island developed between piers 1 and 2 resulting in twin channels in BDC. The left channel was sustained by tributary drainage (Hellbranch Run) formed by a linear ditch created along the freeway that intercepted south flowing storm flows from the floodplain and diverted them west into BDC. This linear tributary was recently (circa 2008) modified to a high amplitude sinuous form - possibly in an effort to restore more natural stream dynamics. It appears that the island has now become joined to the left bank upstream from the confluence of Hellbranch Run. The current conditions are shown in Figure 1.

The relevance of this discussion is that the stream channel morphology at this location is complex and has resulted in a configuration whereby sediment accretion has been an extremely active process in the broad channel created when the bridges were first constructed. This is likely to continue into the foreseeable future and sedimentation, rather than scour erosion, will be the dominant geomorphologic force influencing the channel in the future. Some localized erosion is inevitable of course, given the presence of the meander - this special case is discussed in Section 5.

Figure 1: Channel Characteristics of Big Darby Creek at IR-71

2.3. Soils

Soils at the bridge site have been mapped by the Natural Resources Conservation Service (U.S. Department of Agriculture, 2013) as Ross silt loam, which are all rated as very limited for local road and street construction because of flooding, frost susceptibility, and low strength.

2.4. Seismicity

Earthquake hazard analysis in this part of the country is dominated by proximity to the New Madrid Fault Zone (NMFZ) approximately 400 miles to the southwest. Possible future movements along this fault could generate earthquakes of magnitude 7.0-8.0 with a recurrence period of 500-1,500 years (USGS, 2008). The resulting ground motion would be experienced over a wide area, with the Harrisburg area located within the possible zone of influence. In addition, earthquake epicenters of lesser magnitude (< ~ magnitude 5) occurred in southern Fairfield County (~40 miles southeast) in 1848/1870 and 1967, which indicate other potential earthquake sources that are contributory to seismic risk (ODNR, 2012[(1)]).

2.5. Hydrogeology

Surface water drainage in the area is dominated by the south flowing BDC, a tributary to the Scioto River. The creek is at an elevation of about 780 ft at this location. It should be noted that BDC is a designated National Wild and Scenic River and an Ohio Scenic River.

This portion of BDC is mapped by US Fish and Wildlife Service as a Riverine Wetland (USF&W, 2013). Two additional wetlands (8.77 acre freshwater pond & 2.88 acre freshwater forested/shrub wetland) are mapped 1,000 ft south of the bridge site and 2 about 500 ft north (0.32 and 0.17 Ac freshwater forested/shrub wetlands).

Based on the FEMA Flood Insurance Rate map of Franklin County, the bridge site lies within a special flood hazard area subject to inundation by the 100-year flood (FEMA, 2008). This area is designated AE and is also in the regulatory floodway, an area that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

2.6. Mining and Oil/Gas Production

No abandoned mines are noted on ODNR's Abandoned Underground Mine Locator in the vicinity of the bridge site (ODNR, 2013).

No oil or gas wells are noted within the immediate vicinity of the bridge site (ODNR, 2012[(3)]).

2.7. Site Reconnaissance

A preliminary site reconnaissance was conducted February 4, 2014 during field operation planning and borehole staking at which time the ground was partially snow covered. A second inspection of the area was performed May 24, 2014.

Rural and recreational land surrounds the bridge site. Immediately north and south are parcels included in the Battelle Darby Creek Metro Park (Central Ohio Metro Parks). The area to the north is forest and to the south is allocated for fields and meadows. More forest, followed by agricultural fields lay to the east and west.

The existing twin structures are each supported on piled abutments with spill-through slopes and two hammer-head piers founded on bedrock using spread footings. No evidence of distress or poor performance was observed at the supports that could be attributed to geotechnical factors. The bridge parapet lines appear to be straight and true which is interpreted to indicate absence of significant differential settlement (Photographs 1 and 2).

Photograph 1: FRA-71-0153 NB Looking North.

Note presence of island blocking much of the channel between Piers 1 and 2.

Photograph 2: Upstream Side of SB Structure.

Note debris accretion on island.

The spill through slopes have experienced some erosion and riprap that is shown on the original drawings is absent in some places, and additional armoring appears to have been placed at others. In Photograph 3 note the absence of riprap on the lower slope. A bench that was shown at elevation ~782 ft (about 10 ft below the girders) in the construction drawings is absent, suggesting some erosion of the slope.

Photograph 3: Rear Abutment of SB Structure.

The channel configuration discussed in Section 2.2 is quite striking when observed in the field. Photographs 1and 2 show the island between Piers 1 and 2 that extends the full length of the project area. This material has accumulated since the end of original construction and the island now supports mature woodlands.

Photograph 4 shows the Hellbranch Run channel (foreground) and the main BDC channel (middle distance). The Hellbranch Run channel is apparently not (or only poorly) connected to BDC upstream of the bridge and during low flow in the Run, probably functions as a fairly static backwater channel to BDC.

Photograph 4: Left Channel of BDC from SB Structure Looking South (west).

The area that will be occupied by the proposed widening is shown in Photograph 5.

Photograph 5: Area to be Occupied by Widening

3. EXPLORATION

3.1. Historical Boring Programs

A structure foundation investigation was conducted for the existing bridge in 1962 by The H. C. Nutting Company [*Report of Foundation Investigation, Interstate I-71, Bridge No. FRA-1-0153 (R & L)]*. Twelve borings (2 in the river) were drilled and sampled at the bridge site. Copies of the original boring logs are provided in Appendix A and drilling information is summarized in Table 1 below. The boring locations are shown on Exhibit 1.

Table 1: Historical Boring Summary

Boring Number	Surface Elevation NGVD 29[(1)] (ft)	Station/ Offset	Depth (ft)	Bottom of Hole Elevation (ft)	Depth Bedrock Encountered (ft)	Structure
B-001-B-62	783.3	921+94, 86' L	26.0	783.3	21.0	SB rear abutment
B-002-B-62[(2)]	774.1	922+88, 46' L	24.0	774.1	14.0	SB left pier 1
B-003-B-62	783.1	924+13, 103' L	26.0	757.1	16.0	SB left pier 2
B-004-B-62	784.5	925+10, 57' L	29.0	755.5	11.0	SB forward abutment
B-005-B-62	781.4	921+90, 31' L	27.0	754.4	17.0	NB rear abutment
B-006-B-62	777.6	922+85, 69' R	20.0	757.6	10.0	NB R Pier 1
B-007-B-62	785.0	924+13, 31' R	38.5	746.5	15.0	NB R Pier 2
B-008-B-62	784.6	925+11, 58' R	25.0	759.6	20.0	NB forward abutment
B-009-B-62[(2)]	774.5	922+88, 91' L	16.4	758.1	6.4	SB left pier 1
B-010-B-62	783.1	924+13, 39' L	26.0	757.1	16.0	SB L pier 2
B-011-B-62	776.8	922+88, 24' R	20.0	756.8	10.0	NB R pier 1
B-012-B-62	783.8	924+13, 66' R	27.0	756.8	17.0	NB R pier 2

(1) NGVD 29 - 0.610=NAVD 88
(2) Drilled in BDC.

H.C. Nutting reported the following:

> "The subsurface materials at the site are a combination of alluvial and some residual soils, and consist of a heterogeneous mixture of sands, silts, and clays of rather shallow depth immediately overlying the bedrock. The average elevation of bedrock is encountered in the borings at this site occurs at about elevation 765. Thus with an average ground elevation of 782, there is approximately 17-ft. of overburden at the site."

3.2. Field Exploration

Subsurface drilling was conducted by Central Star Drilling under subcontract to BEI between March 31st and April 1, 2014 and consisted of 2 borings drilled to depths of 62.0 and 72.4 ft below ground surface. All drilling was supervised and logged by a BEI representative. The locations of these borings are shown on Exhibit 1 and summarized below in Table 2. The Logs of Borings are provided in Appendix A.

Table 2: Boring Summary

Boring Number	Boring Location (Lat/Long)	NAVD 88 Surface Elevation (ft)	Depth (ft)	Bottom of Hole Elevation (ft)	Depth to Groundwater (ft)	Depth to Bedrock (ft)	Sub-structure
B-021-1-14	39.821008, -83.169800	797.0	72.4	724.6	18.0	31.5	abutment
B-021-2-14	39.821528, -83.168522	798.0	62.0	736.0	NE	32.7	abutment

The borings were drilled using a truck-mounted, CME 55 Automatic rig with 2.25-inch diameter hollow stem augers. Soil samples were recovered at 2.5-ft intervals until encountering bedrock using a split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils"). At bedrock the samples were collected in 5.0-ft increments using an NQ2, triple tube, core barrel, water method to core.

The standard penetration test (SPT) was conducted during sampling using an auto-hammer that was calibrated June 12, 2012 as 75% efficient. Field boring logs were prepared by the B&P field supervisor including lithological description and standard penetration test results, recorded as blows per 6-inch increment of penetration. Groundwater observations were recorded during the investigation. Hand penetrometer testing was conducted on a majority of cohesive SPT samples prior to removal from the sampler. Each boring was backfilled with bentonite grout and soil cuttings.

3.3. Laboratory Testing Program

The laboratory testing program consisted primarily of classification testing and moisture content determinations. Data from the laboratory-testing program were incorporated onto the logs of borings (Appendix A). Soil samples are retained at the laboratory for 60 days following report submittal, after which time they will be discarded.

3.3.1. Classification Testing

Natural moisture content tests were performed on all soil samples. Representative soil samples were selected for index property (Atterberg Limits) and gradation testing for classification purposes. The results are presented on the log of the boring. Mechanical soil classification (Plastic Limit, Liquid Limit and gradation testing) was conducted on 62% of the recovered samples enabling identification and testing of all significant soil units.

Final classification of soil strata in accordance with AASHTO M-145 "Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes," as modified by ODOT "Classification of Soils" was made once laboratory test results became available. Samples that were not tested were classified visually on the basis of comparison to those that were.

3.3.2. Standard Penetration Test Results

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils was performed at 2.5-foot intervals in all borings using a calibrated auto-hammer. The resulting N-values must then be adjusted to account for the high efficiency of the hammer, compared to those used historically when many of the correlations of N-value with engineering properties of soils were developed. Manual hammers used in the past are considered to have been approximately 60% efficient and so the field measured N-values are adjusted by a factor equal to the calibrated efficiency/60. The resulting N_{60} values are shown on the log of borings.

3.3.3. Unconfined Compression Testing

Eight samples of dolomite bedrock collected from borings drilled at the rear and forward abutments were tested in unconfined compression as a means of characterizing the strength of the overall rock formation. The results of these tests are presented in Appendix A and summarized in Table 3 below.

Table 3: Unconfined Compressive Strength Test Results

Boring	Depth (ft)	Elevation (ft)	Material	Unconfined Compressive Strength (psi)	Strain (%)	Density (pcf)	Moisture Content (%)
B-021-1-14	31.8-32.1	766.0	limestone	7,558	1.1	160.2	0.5
	32.5-32.8	765.4	limestone	3,697	0.5	154.7	1.0
	36.3-36.6	761.6	dolomite	8,960	1.0	151.7	4.1
	38.2-38.5	759.6	limestone	8,396	0.8	165.7	0.4
	47.5-47.8	750.3	siltstone	5,803	0.5	135.3	9.2
B-021-2-14	34.3-34.6	763.6	limestone	8,491	0.7	159.3	1.8
	38.5-38.8	759.3	limestone	12,175	0.7	162.7	0.9
	42.1-42.4	755.7	limestone	16,868	0.9	165.4	0.7

4. FINDINGS

The following interpretation of the subsurface conditions is based on results of the historical and current field explorations, laboratory testing, and consideration of the geological history of the site.

4.1. General

The stratigraphy at the bridge site is generally consistent with the geological model discussed above and consists of 6-20 ft of unconsolidated soils, comprised primarily of sand and gravel with occasional strata of clay, overlying limestone bedrock. Where the freeway approach embankments are present the overburden is approximately 15 ft thicker.

4.2. Overburden

Overburden soils are fairly consistent. The approach embankments are composed of sand, silt and clay mixtures (A-4a, A-6a, A-6b) that are typically stiff to very stiff, or dense. The embankments are founded on natural soils that are also fine grained and appear to be floodplain deposits with traces of organics (roots), and shells being reported in the samples. This stratum is on the order of 1.5 - 2.5 ft thick at the two project boring locations where it was observed. The remaining overburden is dominantly sand and gravel mixtures with varying amounts of silt and clay (A-1-a, A-2-6) and is medium dense to dense. Bedrock is occasionally mantled by a thin stratum of highly plastic reddish brown clay (A-7-5, A-7-6) up to about 4 ft in thickness. Alternatively, this stratum may be comprised of brown silt and clay mixtures (A-6a).

4.3. Bedrock

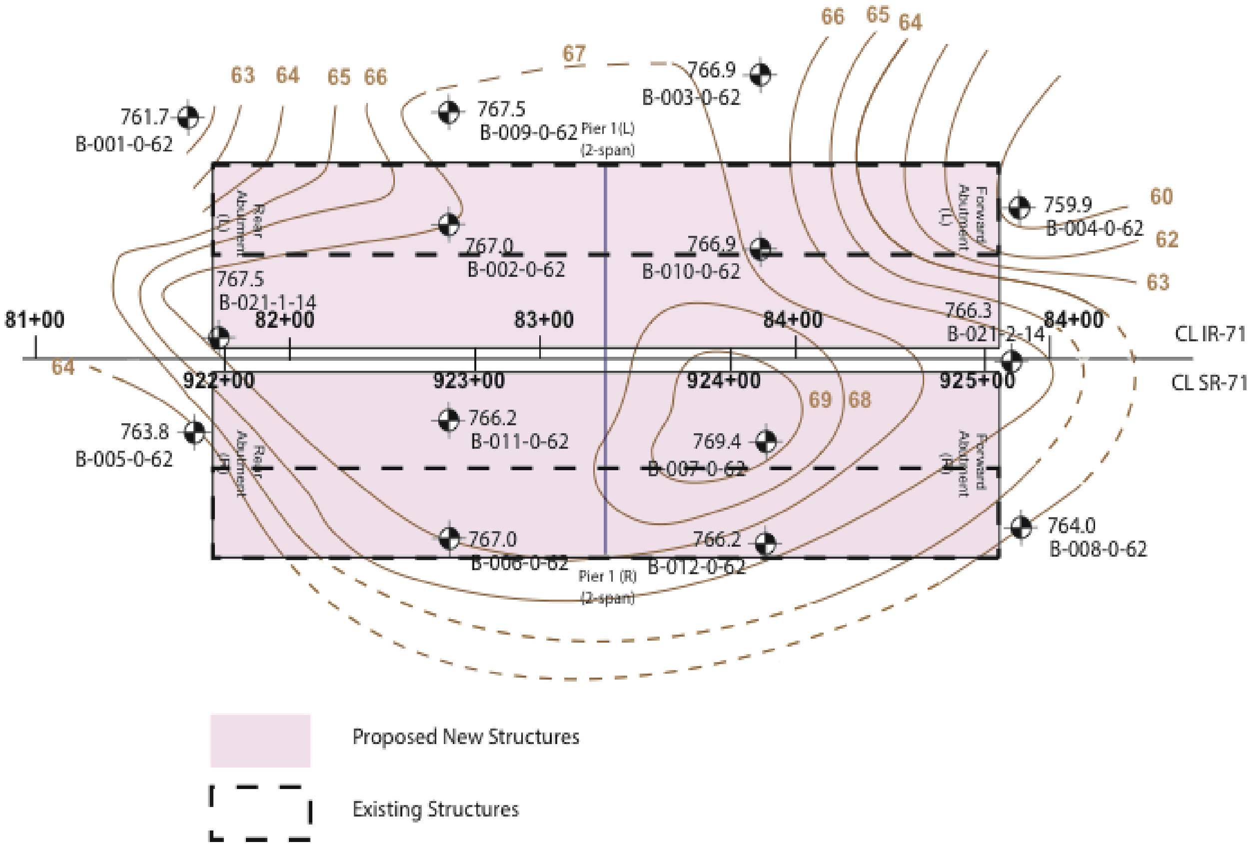
Bedrock was encountered at an elevation of 761-770 ft and consists of limestone with occasional dolomite and siltstone strata in the upper 30 ft.

A top of rock contour map has been developed using the boring data and is shown below in Figure 2. Note that this map is an interpretation of data collected at specific points and the actual surface may vary from that shown. However, the contours are considered useful for purposes of design.

As indicated above, the site lies in an area of karst geology. Surface features indicative of a karstic landscape are not present in the area, but this does not preclude the existence of paleo-karst solution features within the bedrock itself that were formed in the distant past. Regionally, these may have been rendered stable by the deposition of a thick glacial till mantle - at least from the standpoint of surface sinkhole development. Solution features can occur as cavities and channels that may be clay or air filled, as in the case of caves and voids that are known to exist in other parts of the Columbus Limestone deposit. The ability of these features to support concentrated point loads is difficult to predict.

One historical boring (B-007-B-62) shows evidence of solutioning. The 20 ft of core is characterized as containing at least three 4-6 inch thick zones of reddish brown soil (varying from silty clay to gravel and rock fragments to elastic clay), the deepest of which is 13 ft below the top of rock elevation. Core recovery is 25 - 30% throughout much of the zone indicating challenging drilling conditions. Project borings B-021-1-14 showed similar recoveries in the top 16 ft of bedrock.

Figure 2: Top of Bedrock



Source: H.C. Nutting Company, 1962 and B&P, 2014

4.4. Groundwater

Groundwater was encountered in the borings as summarized below in Table 4. Groundwater may be expected at, or a little above, the creek level.

Table 4: Groundwater Summary

Boring Number	Surface Elevation (ft)	Depth (ft)	Depth to Groundwater During Drilling (ft)	Elevation of Groundwater (ft)	Depth to Groundwater at 24 hrs (ft)	Elevation of Groundwater (ft)
B-002-B-62	774.1	21.5	in creek	776.6	in creek	776.6
B-003-B-62	783.1	26.0	6	777.1	6.3	776.8
B-004-B-62	784.5	29.0	7	777.5	6.7	777.8
B-005-B-62	781.4	27.0	4	777.4	-	-
B-006-B-62	777.6	20.0	2	775.6	-	-
B-007-B-62	785.0	38.5	-	-	8	777.0
B-008-B-62	784.6	25.0	8	776.6	7.3	777.3
B-009-B-62	774.5	16.4	in creek	776.6	in creek	776.6
B-010-B-62	783.1	26.0	7	776.1	-	-
B-011-B-62	776.8	20.0	0.2	776.6	0.2	776.6
B-021-1-14	797.0	18.0	18.0	779.0	-	-

Notes (1) Water used as drilling fluid for coring.
(2) 1962 borings are referenced to NGVD 29 (NAVD88 = 0.6 ft).

4.5. Soil Properties for Analysis

Generalized material profiles and physical properties for analysis have been developed. These are based primarily on published engineering correlations with index properties and consistency data as indicated by SPT results and hand penetrometer readings. The recommended soil properties are shown in Table 5.

Table 5: Geotechnical Soil Properties

Soil Type	Description	Property	Value	Source
	799	N_{60}	25	Appendix A
		WC	14%	Appendix A
1	Silt/Clay Mixture (A-6a, 6b)	S_u	2,000 psf	Appendix A
		c'	400 psf	GB 6
	781	Φ'	30º	GB 6
	781	N_{60}	18	Appendix A
		WC	20%	Appendix A
2	Silt/Sand/Clay (A-4a, A-6a)	S_u	1,000 psf	Appendix A
		c'	100 psf	Estimate
	779	Φ'	30º	GB 6
	779	N_{60}	36	Appendix A
		WC	14%	Appendix A
3	Sand/Gravel (A-2-4)	S_u	-	-
		c'	-	-
	770	Φ'	35º	Estimate
	770	N_{60}	22	Appendix A
		WC	40%	Appendix A
4	Elastic Clay (A-7-5)	S_u	2,000 psf	Estimate
		c'	400 psf	GB 6/Estimate
	766	Φ'	20%	GB 6/Estimate
5	766 Limestone Bedrock Potential for low recovery indicating clay filled voids 750	RQD	21%	Appendix A
6	750 Limestone Bedrock 737	RQD	60%	Appendix A

4.6. Unconfined Compression Strength of Rock

A series of 8 unconfined compression tests were performed on samples of bedrock into which the proposed drilled shafts will be socketed. Samples were selected so as to be as broadly representative of the rock as possible. Results of the tests are provided in Appendix A and are summarized in Table 3.

The samples were recovered from elevations between ~ 750 and 766 ft. The top of the drilled shaft rock sockets are expected to be at ~ 767 ft. Six of the samples are limestone, one is dolomite and the other siltstone. They are all treated as members of a single population for purpose of analysis. The properties of the rock were evaluated statistically to establish strength parameters for use in design (See Appendix B). The arithmetic mean of the unconfined compressive strength is 8,994 psi and the range is 3,697 psi - 16,868 psi. The lower confidence limit (lcl) of the mean at 95% confidence level is 5,634 psi and the upper confidence limit (ucl) is 12,353 psi, based on Student's t-test. The lcl should be used where a cautious mean is required for axial resistance calculations, for example, and the ucl where excavation difficulty is being considered.

4.7. Scour

This section provides an analysis of the available grain size data for soils in the bed of the Big Darby Creek and the adjacent over-bank areas. The current exploration was structured to maximize the utilization of available historical boring logs, and to minimized any environmental impacts to the Big Darby Creek by drilling. Accordingly, the two new project borings were drilled from the approach embankments and did not penetrate the creek channel directly. The historical exploration did include 2 borings in the channel, 2 very close to the channel and 8 in the over-bank area (as it then existed). There is, therefore, a hybrid body of grain size data that may be used for scour analysis: samples from the elevations representative of, but lateral to the over-bank and channel bottom collected and analyzed during this exploration and for which there are D_{50} and D_{95} results, and the upper two or three samples from the historical exploration that are actually from the channel and over-bank area, but for which there is only broad classification data (gravel, coarse sand etc. fractions). Channel bottom elevations are considered to be those between 777 ft and 771 ft, and over-bank 783 ft and 777 ft.

The currently measured grain size results (D_{50} and D_{95}) are presented in Table 6 together with their broad classification profiles. The classification profiles of the historical samples are presented in Table 7 for comparison. In both cases the results are separated for the channel and over-bank locations. An attempt has been made to create categories of material whereby soils that were currently tested can be correlated with the historical samples and their D_{50} and D_{95} values estimated.

Table 6: Project Grain Size Analysis

Project Boring	Sample	Elevation	D_{95}	D_{50}	Fractions					Soil Type
					Gravel	Coarse Sand	Fine Sand	Silt	Clay	
Overbank										
B-021-1-14	7	781.0-779.5	0.375	0.0.17	1	2	28	37	32	
	8	778.5-777.0	0.425	0.067	1	4	44	30	21	
B-021-2-14	7	782.0-778.5	1.345	0.081	2	11	39	29	19	D
	8a	779.5-778.5	3.794	0.14	10	22	28	20	26	A
	8b	778.5-778.0	20.992	1.378	45	21	1	15	8	C
	9	777.0-775.5	11.129	0.809	28	38	15	13	6	
Channel										
B-021-1-14	9	776.0-774.5	29.315	0.839	38	22	13	23	4	C
B-021-2-14	10	774.5-773.0	3.051	0.549	8	50	24	12	6	C
B-021-1-14	10	774.0-772.5	33.945	1.112	41	22	16	18	3	A
B-021-2-14	11	772.0-770.5	7.658	0.053	13	12	20	35	20	1

Table 7: Historical Grain Size Analysis

<table>
<tr><th rowspan="2">Historical Boring</th><th colspan="5">Fractions</th><th rowspan="2">Soil Type</th></tr>
<tr><th>Gravel</th><th>Coarse Sand</th><th>Fine Sand</th><th>Silt</th><th>Clay</th></tr>
<tr><td colspan="7">Overbank</td></tr>
<tr><td rowspan="2">B-012-B-62</td><td>42</td><td>21</td><td>15</td><td>15</td><td>7</td><td>C</td></tr>
<tr><td>43</td><td>29</td><td>14</td><td colspan="2">14</td><td>C</td></tr>
<tr><td rowspan="2">B-010-B-62</td><td>35</td><td>25</td><td>17</td><td>15</td><td>8</td><td>C</td></tr>
<tr><td>61</td><td>19</td><td>9</td><td>9</td><td>4</td><td>B</td></tr>
<tr><td rowspan="2">B-008-B-62</td><td>23</td><td>14</td><td>36</td><td>19</td><td>8</td><td>C</td></tr>
<tr><td>4</td><td>16</td><td>46</td><td>25</td><td>9</td><td>D</td></tr>
<tr><td rowspan="2">B-007-B-62</td><td>46</td><td>20</td><td>15</td><td>12</td><td>7</td><td>C</td></tr>
<tr><td>71</td><td>14</td><td>6</td><td colspan="2">9</td><td>B</td></tr>
<tr><td rowspan="2">B-005-B-62</td><td>66</td><td>15</td><td>6</td><td colspan="2">13</td><td>B</td></tr>
<tr><td>70</td><td>19</td><td>5</td><td colspan="2">6</td><td>B</td></tr>
<tr><td rowspan="2">B-004-B-62</td><td>55</td><td>23</td><td>12</td><td colspan="2">10</td><td>B</td></tr>
<tr><td>24</td><td>23</td><td>29</td><td>19</td><td>5</td><td>C</td></tr>
<tr><td rowspan="2">B-003-B-62</td><td>68</td><td>20</td><td>5</td><td colspan="2">7</td><td>B</td></tr>
<tr><td>71</td><td>17</td><td>4</td><td colspan="2">8</td><td>B</td></tr>
<tr><td rowspan="2">B-001-B-62</td><td>24</td><td>8</td><td>25</td><td>27</td><td>6</td><td>A</td></tr>
<tr><td>76</td><td>9</td><td>3</td><td colspan="2">10</td><td>B</td></tr>
<tr><td colspan="7">Channel</td></tr>
<tr><td rowspan="2">B-002-B-62</td><td>71</td><td>12</td><td>6</td><td colspan="2">11</td><td>B</td></tr>
<tr><td>70</td><td>13</td><td>6</td><td colspan="2">11</td><td>B</td></tr>
<tr><td rowspan="4">B-006-B-62</td><td>65</td><td>17</td><td>7</td><td colspan="2">11</td><td>B</td></tr>
<tr><td>64</td><td>23</td><td>6</td><td colspan="2">7</td><td>B</td></tr>
<tr><td>16</td><td>11</td><td>21</td><td>41</td><td>11</td><td>A</td></tr>
<tr><td>4</td><td>8</td><td>16</td><td>35</td><td>27</td><td>A</td></tr>
<tr><td rowspan="3">B-009-B-62</td><td>67</td><td>15</td><td>7</td><td colspan="2">11</td><td>B</td></tr>
<tr><td>67</td><td>17</td><td>6</td><td colspan="2">10</td><td>B</td></tr>
<tr><td>68</td><td>19</td><td>5</td><td colspan="2">8</td><td>B</td></tr>
<tr><td rowspan="2">B-011-B-62</td><td>71</td><td>14</td><td>5</td><td colspan="2">10</td><td>B</td></tr>
<tr><td>62</td><td>16</td><td>7</td><td>12</td><td>3</td><td>B</td></tr>
<tr><td>Average For Type B Soils</td><td>67</td><td>16</td><td>6</td><td colspan="2">11</td><td></td></tr>
</table>

Table 8: Soil Type C Analysis

Historical Boring	Fractions					Soil Type
	Gravel	Coarse Sand	Fine Sand	Silt	Clay	
Overbank						
B-012-B-62	42	21	15	15	7	C
	43	29	14	9	5	C
B-010-B-62	35	25	17	15	8	C
B-008-B-62	23	14	36	19	8	C
B-007-B-62	46	20	15	12	7	C
B-004-B-62	24	23	29	19	5	C
Averages						
Historicals	36	22	21	15	7	C
Project	41	22	10	19	5	C

It should be recognized that the morphology of the valley fill sediments as glacial outwash deposits, and the current fluvial environment has the potential to result in significant heterogeneity of material type over short distances, and this is clearly revealed in the data.

4.7.1. Channel Bed Conditions

Historical results show the upper 6 ft of the channel bed to be comprised (on average) primarily of gravel (67%) and sand (22%). The project borings show a more uniformly graded material in this elevation range: 26% gravel, 47% sand, 27% silt and clay

4.7.2. Over-bank Conditions

Historical borings show conditions in the over-bank area in two broad categories - the first is similar to the creek bed with a high (60-70%) gravel fraction and the second is more uniformly graded material, but still dominantly granular. Project borings in this elevation range include soils that have similar gradations and some that are finer silt and clay dominated mixtures.

4.7.3. D_{50} and D_{95} Values for Design

4.7.3.1. Channel Bed

The channel bed is dominated by soil type B - high gravel content. There is a reasonable correlation between the gravel fraction percentage and both D_{50} and D_{95} as shown in Figures 3 and 4. The projected values of 1.85 mm and 29 mm are considered conservative (low) values based on the trends that could be inferred from the data.

Figure 3: D_{95} Estimation

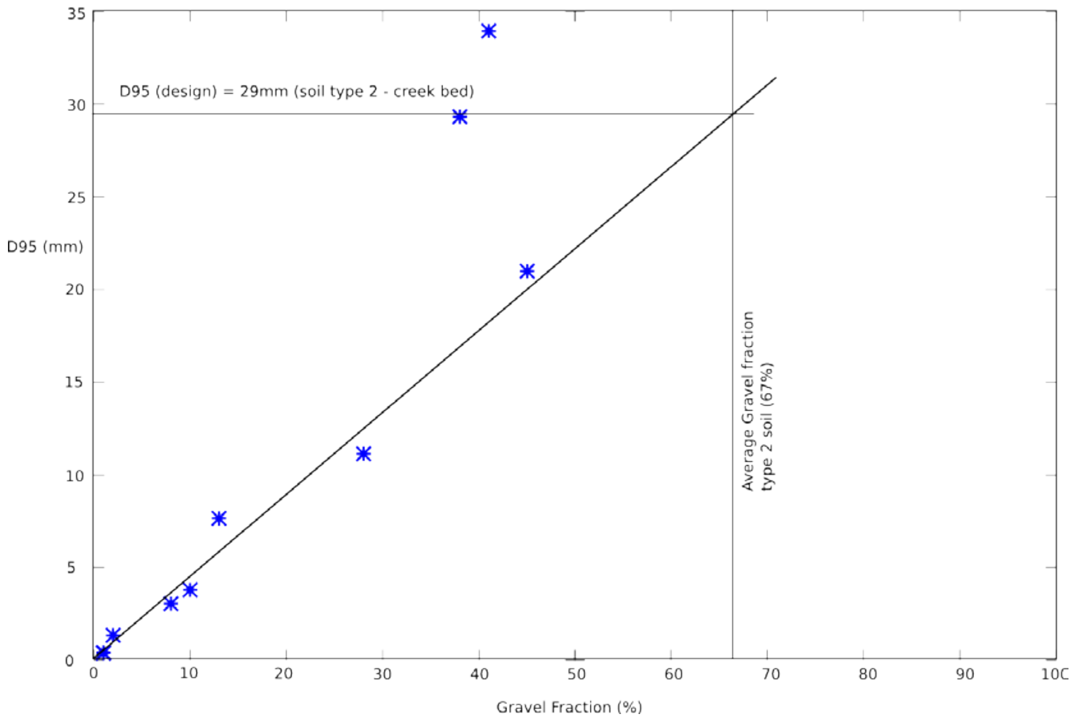
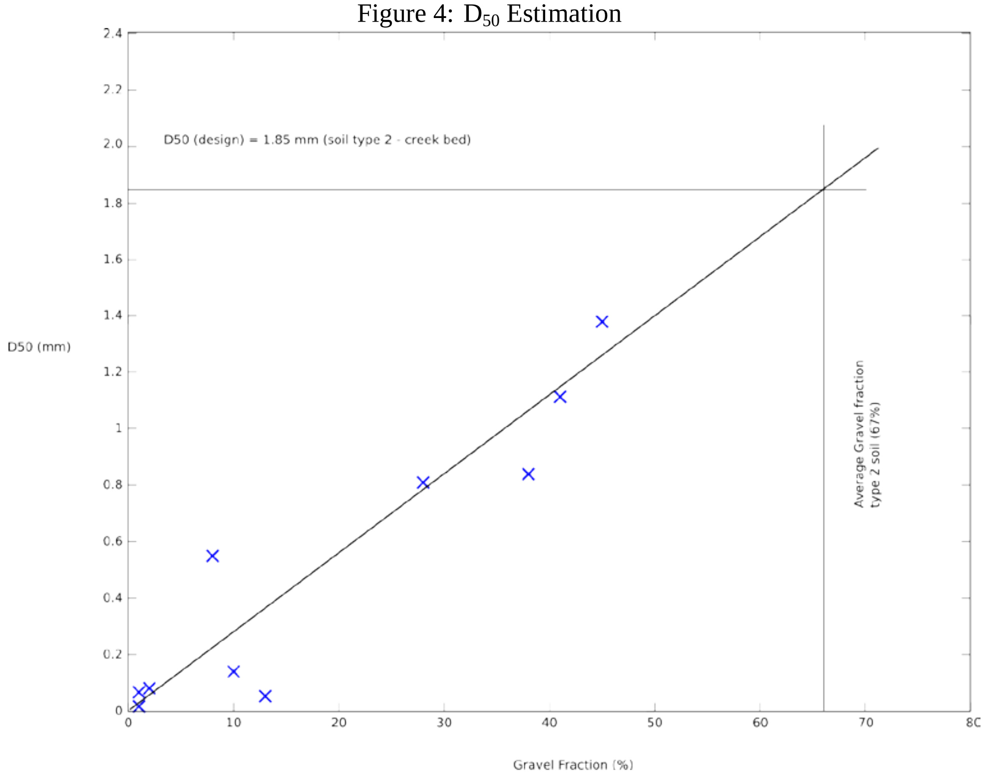


Figure 4: D_{50} Estimation



4.7.3.2. Over-bank conditions

Over-bank conditions are, as indicated above split between channel bed type soils (50%) and those that are somewhat finer grained, but still dominantly granular - characterized as soil type 3 (~40%). Based on similarities in grain size profile between the project and historical type C soils, it is reasonable to attribute the same D_{50} and D_{95} values to them, the averages of which are 1.1 mm and 28 mm respectively. These being slightly lower than the channel bed values (also the other 50% of the overbank), it is recommended that they be used for overbank scour calculations.

5. ANALYSIS AND RECOMMENDATIONS

5.1. Abutments

5.1.1. Global Stability

No significant modification to the embankments is planned and, therefore, the global stability of the abutment spill through slopes is expected to remain unchanged. The existing embankments are stable and have performed satisfactorily. Stability was evaluated using PC based software entitled Slide 7.0 by Rock Science, Inc. implementing the Modified Bishop and simplified Janbu analysis methods of computation. The soil profile and material properties were those presented in Table 9. The computer input and output are provided in Appendix B. The analysis includes the addition of a live load surcharge equivalent to 2 ft of soil along the approach embankments. Strength properties for the retained soils (rear embankment extensions) are assumed as indicated above. The analysis was conducted considering short-term (end of construction - total stress) conditions and long-term (effective stress) conditions, with seismic effects.

AASHTO requires that the analysis be conducted at the Service I limit state in which most of the load factors are 1.0. The acceptable resistance factor is 0.67 when the slope supports another structural element such as the bridge abutment. AASHTO accepts the reciprocal of a software-generated factor of safety (FOS) because it depends somewhat on the strength properties of the soils which are proportional to resistance. In other words, if the reciprocal of the calculated FOS is <= 0.67, the result is satisfactory. The equivalent 'resistance' factor is satisfied if the calculated FOS is >= 1.5. The results of the analyses are summarized in Table 9 and the computer-generated reports, each including a cross-section showing the most critical failure circle, are presented in Appendix B-1.

Table 9: Results of Global Stability Analysis

Case	Description	Factor of Safety	Equivalent Resistance Factor
1	End of Construction/Total Stress	2.635	0.38
2	Long Term/Effective Stress	1.875	0.53

5.1.2. Deep Foundations

The abutments will be supported on deep foundations that derive their load bearing capacity from the bedrock. Loads and foundation elevations provided by the designers presented in Table 10.

Table 10: Abutment Loads (Factored) – Front and Rear

Load Conditions	Maximum (kips)	Minimum (kips)
Service Limit State	261.53	28.20
Strength Limit State	354.56	-16.68 (uplift)

It is proposed to use HP 12x53 piles, with a maximum structural ultimate bearing resistance of 380 kips. This resistance exceeds the maximum load at the strength limit state. Pile lengths are expected to be as shown in Table 11 based on the rock contours shown on Figure 2 and pile cap elevations provided by the designers. The calculated 'estimated' length assumes penetration through one foot of broken rock, one-foot embedment in the pile cap, and rounding up to the nearest 5 ft. Note that estimated order lengths from the 1962 design were 25 ft (based on the original schedule of quantities).

Table 11: Estimated Pile Lengths

Location	Pile Cap Elevation (ft)	Top of Rock Elevation (ft)	Geotechnical Design Length (ft)	Estimated Length (ft)
Rear Abutment L	781.17	763	18	25
Rear Abutment R	781.17	766	15	20
Forward Abutment L	781.17	764	19	25
Forward Abutment R	781.17	765	18	25

Pile points should be provided to maximize penetration through any zones of broken rock encountered at the bedrock surface.

5.1.3. Uplift Resistance

At the strength limit state, the designers estimated that the abutment foundations may experience uplift forces that amount to 16.7 kips per pile. According to AASHTO Sections 10.7.3.8.6., 10.7.3.10 and 10.7.3.11 and BDM Section 202.2.3.2.i “Uplift Resistance of Piles”, the uplift resistance of the proposed single driven H-pile has been evaluated with the static analysis method by using the computer program Driven. Driven results can be found in Appendix B. With respect to the soil profile and the soil's engineering properties, the findings of the exploration and soil properties presented in Table 5 of this report were used in our analyses. It should be noted that the drained friction angle of Soil Type 3 has been decreased to 34 degree for cases of new construction within unconsolidated, alluvium deposits. In order to obtain the factored uplift resistance, the reduction factor for uplift friction of 0.25 for clays and 0.35 for sands was applied (AASHTO LRFD Table 10.5.5.2.3-1).

The uplift resistance of pile group has been evaluated based on AASHTO Figure 10.7.3.11-1 because cohesionless soil conditions are predominant in the soil conditions located at the bottom of the proposed pile cap elevation to an arbitrary depth of 15ft below. The uplift design of HP pile group was evaluated using a MathCAD-based software solution, as described in Appendix B. The results are summarized in Table 12.

Table 12: Uplift Resistance Results

Pile Cap Elevation (ft)	Minimum Tip Elevation (ft)	Minimum Geotechnical Design Length for Uplift (ft)	Uplift Check for Single Pile	Uplift Check for Pile Group
781.17	768.17	12	OK	OK

5.2. Piers

5.2.1. Deep Foundations

The widened structures will be supported on deep foundations that derive support from the rock encountered beneath the site, and will consist of drilled shafts socketed into the bedrock. It is proposed to utilize 54" shafts with 48"-diameter sockets. Nominal and factored resistance values have been computed using the procedures in AASHTO 10.8 as shown in Appendix B. Results are summarized in Table 13. The maximum proposed factored axial load is 1,418 kips/shaft.

The side resistance was computed using the lower confidence limit mean value for limestone based on the results of laboratory unconfined compression testing (see Appendix A and B). Base resistance was computed using the estimated strength computed in consideration of the rock mass index. Note that the side resistance associated with the overburden is ignored in this calculation.

The upper 2 feet of rock is ignored when considering side resistance, and the base resistance is not considered unless the socket is at least 1.5*diameter long (8 ft in this case). A resistance factor of 0.5 was applied for both tip and site resistance on the assumption that load testing will not be conducted.

Table 13: Drilled Shaft Resistance Estimates

Sub-structure	Total Socket Length (ft)	Nominal Side Resistance (kips)	Nominal Base Resistance (kips)	Factored Side Resistance (kips)	Factored Base Resistance (kips)	Total Nominal Resistance (kips)	Total Factored Resistance (kips)
Pier 48"	7	455		250	12,666	455	250

Rock Socket	8	607	25,333	334	12,666	25,941	13,001
	10	911	25,333	501	12,666	26,244	13,168
	12	1214	25,333	668	12,666	26,548	13,335
	14	1518	25,333	835	12,666	26,852	13,502

Table 14: Estimated Tip Elevations-48-Inch-Diameter Drilled Shafts

Pier	Column (numbered left to right)	Drilled Shaft Tip Elevation (10 ft socket) (ft)
1 Left	1	757
	2	758
	3	758
	4	758
	5	758
1 Right	1	758
	2	758
	3	758
	4	758
	5	757

An 8-ft-long socket is the minimum allowable based on the design standards. However, uncertainties regarding the quality of the upper zone of rock are such that, for design, it is recommended that an additional 2 ft of rock be assume unsuitable and that the design length be increased to 10 ft.

The 10-ft long socket is recommended provided the rock is sound. To guard against possible problems associated with the presence of karstic voids, a core hole should be drilled to a depth of 25 ft below top of rock at each shaft location prior to its construction under the supervision of a geotechnical engineer or geologist.

5.2.2. Lateral Load Resistance

Deep foundation elements subjected to horizontal loads and/or moments have been analyzed for maximum bending moments and lateral deflections. The Generalized Soil Parameters, including recommended lateral soil modulus, and soil strain to be used to analyze the laterally loaded pile by the p-y curve method are shown in Table 15. The p-y curve method for laterally loaded pile analyses has been performed using the L-Pile Plus computer software for load cases provided by the designer. The results are presented in Appendix B.

Table 15: Generalized Soil Parameters for Lateral Load Pile Analysis

Type	Idealized Soil Profile	Applicable Length	Saturated Unit Weight (pci)	Friction Angle	Undrained Shear Strength (psi)	Lateral Soil Modulus Parameter, k (pci)	Soil Strain Parameter, E_{50} (%)
3	sand/gravel	see App B	0.036	36	-	60	-
4	elastic clay	see App B	0.036	-	6.9	500	0.007

5	weak rock	24 in	0.051	-	500	1.4E+6	0.0005
6	sound rock	96 in	0.056	-	5600	-	-

Groundwater Level = 1,005 ft

5.3. Scour

Erosion at river crossings can take a number of forms including traditionally analyzed bed erosion that occurs during high flow events as either general/contraction scour or local pier scour. Another form is lateral migration of the channel, a continuous process in streams flowing in oversized valleys with broad floodplains. Lateral migration is associated with meander development and results in a tendency for bank erosion on the outside of the meander, and deposition on the inside.

The sinuosity of a creek channel creates a tendency for migration of the meanders across the flood plain (Lagasse et al, 2012). This is manifest by differential erosion of the riverbank from the inside to the outside of each meander bend. The velocity distribution of the high flow events is such that there is an increase in velocity on the outside of the bend - and flow may become turbulent. This is the phenomenon that produces bank erosion and tries to undermine the spill through slopes of abutments located on the outside of bends.

The BDC crossing is located within a meander in which the main channel changes direction about 90º to the left. The rear abutments are therefore vulnerable to erosion. The presence of the island attests to the depositional portion of the process on the inside of the bend around the piers. ODOT prescribes rock channel protection for the whole spill through slope (BDM, 203.3) based on the flow velocity in the channel. The applicable outside-of-bend flow velocity for use in design of rock protection may be calculated using the procedure provided in Engineering Manual 1110-2-1601, (Dept. of the Army, 1994). This produces a multiplier for the mean flow velocity based on the sinuosity of the channel that, when applied, gives an estimate of the outside-of-the-bend velocity.

For BDC, the bend radius (R) is ~ 500 ft and the bank-full channel width (W) is ~120 ft.

$$R/W = 500/120 = 1.4$$

Referring to Plate B-33 (RipRap Design Velocities - provided in Appendix B), the multiplier is 1.4. So, for example, if the mean flow velocity in the channel is 6 fps, the riprap design velocity will be 6 * 1.4 = 8.4 fps.

It would seem logical that the forward abutments are located near the inside of the meander and subject more to accretion than erosion. However, the inflow of water from Hellbranch Run appears to be creating a more complex pattern as it reflects off the island and is diverted 90º left. This places the forward abutments on the outside of a new meander that will attempt to form, with associated potential for erosion. Velocities are expected to be lower, but the approach to velocity adjustment discussed above is recommended, again using a factor of 1.4.

This is a simplistic model, but appears appropriate, at a minimum, for the current needs at this stage of design.

5.4. Seismic Design

ODOT has determined that the whole state lies within Seismic Zone 1. Based on the results of the subsurface exploration, the laboratory test data, and our review of the AASHTO Site Class Definition, we recommend a project site classification of B (rock with 2,500 ft/s,<$V_s$avg<5,000 ft/s.

5.5. Recommendations

5.5.1. General

1. Replacement of the existing structures is planned for bridges FRA-71-0153 L&R.

2. Based on the geotechnical information considered during this investigation, drilled shafts and rock sockets appear to represent the best candidate for supporting the proposed bridge piers. Abutments will be supported on H-piles driven to rock.

3. Foundation analyses were performed in accordance with the provisions of LRFD (AASHTO, 2014, 7th Edition) and the ODOT BDM (2007).

4. The proposed substructures and site development concepts were provided by Mead & Hunt, Inc.

5.5.2. Abutment Stability and Foundations

1. The existing spill through abutments will be largely unchanged and the current 2:1 slopes meet requirements for global stability.

2. HP 12x53 piles driven to rock will provide maximum factored structural resistance of 380 kips. Uplift resistance of 16.7 kips will be provided for piles that are at least 12.9 ft in length.

5.5.3. Drilled Shafts

1. Axial capacity estimates for rock supported drilled shafts/rock sockets are provided in Appendix B and Table 12. The capacities presented are for single shafts and do not include group reduction factors. Tip elevations for 48-inch-diameter drilled shafts are shown in Table 13.

2. If load testing of drilled shafts in soil is conducted, the resistance factor used to determine the allowable axial capacity for design purposes may be revised to 0.7 as outlined in AASHTO Table 10.5.5.2.4-1.

3. All shaft capacities presented in Appendix B are for single shafts. In addition to applying appropriate resistance factors, individual capacities for shafts in-group configurations should be further evaluated. For shafts bearing on rock, the Group Efficiency Factor shall be 1.0.

4. The Contractor should embed the drilled shafts to the plan tip elevation or to an elevation as directed by the Engineer.

5. If a temporary casing for drilled shafts is used during construction, the Contractor should either wait until concrete has been placed for the entire length of the shaft before pulling the casing, or the level of the concrete being placed should be maintained several feet above the hydrostatic head as the casing is retrieved. These measures should be implemented by the Contractor to reduce the likelihood of soils collapsing into the shaft excavation and detrimentally affecting the structural integrity of the drilled shafts.

6. If drilling slurry is to be used during shaft installations, the slurry should be capable of suspending the soil particles encountered and not leave a thick coating of slurry, or “mud”, on the excavation sides or bottom. The Contractor shall submit a detailed plan for its use and disposal along with a drilled shaft installation plan to the Engineer for approval prior to implementation. The Contractor shall supply all equipment and construction techniques involving slurry that are necessary to maintain environmental standards.

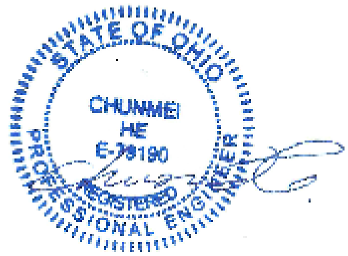
6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the site of bridges FRA-71-0153 L&R, performing geotechnical engineering analyses, and providing recommendations for the design and construction of the foundations only. The analyses and recommendations submitted in this report are based upon data obtained from borings drilled at the locations shown on Exhibit 1 and as presented on the Logs of Boring (Appendix A). This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes in the nature, design or location of the proposed wall is made, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed, and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to Mead & Hunt, Inc. in performing the ongoing geotechnical explorations for the FRA-71-0.00 project.

Respectfully Submitted,

Barr Engineering, Inc.



3/24/17

Enoch Chipukaizer
Principal

Chunmei (Melinda) He, Ph.D., P.E.
Geotechnical Engineer

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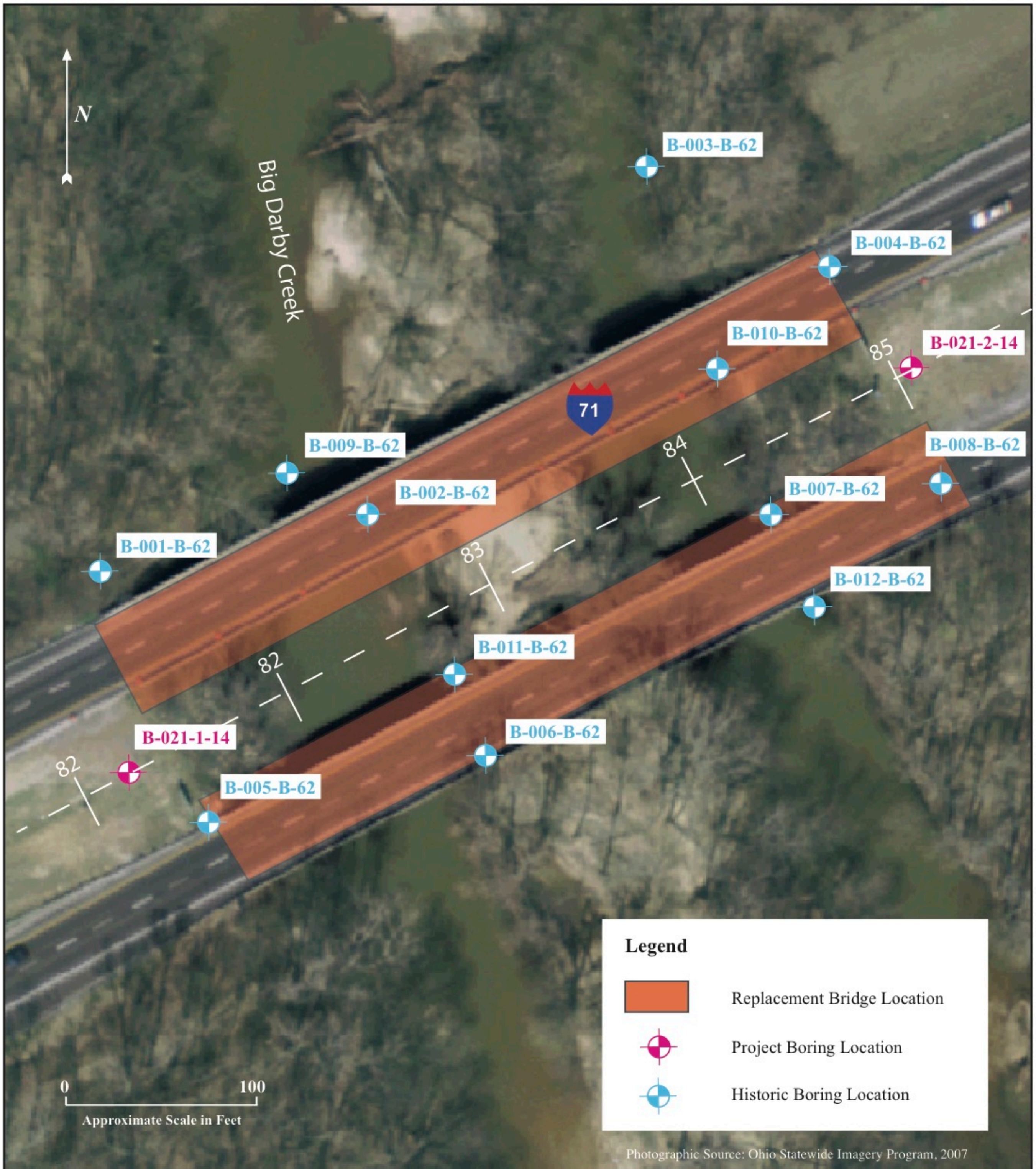


Area of Detail

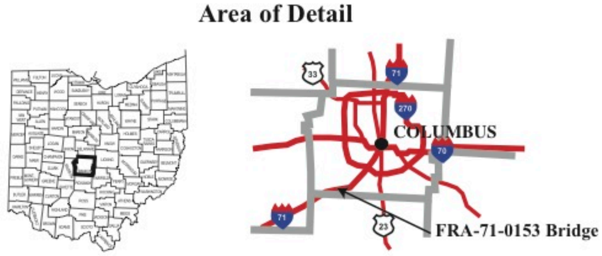


Exhibit 1

BARR ENGINEERING, INC.

Boring Locations
FRA-71-0153 L&R
Bridge over Big Darby Creek

FRA-71-0.00 Improvements
PID#: 93496

APPENDIX A

LOGS OF BORINGS, ROCK CORE PHOTOGRAPHS AND LABORATORY TESTING RESULTS

BARR ENGINEERING, INC.

LEGEND

SYMBOL	DESCRIPTION	ODOT CLASSIFICATION
	Gravel and/or Stone Fragments	A-1-a
	Gravel and/or Stone Fragments with Sand	A-1-b
FS	Fine Sand	A-3
	Coarse and Fine Sand	A-3a
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4 A-2-5
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6 A-2-7
	Sandy Silt	A-4a
	Silt	A-4b
	Elastic Silt and Clay	A-5
	Silt and Clay	A-6a
	Silty Clay	A-6b
	Elastic Clay	A-7-5
	Clay	A-7-6
	Organic Silt	A-8a
	Organic Clay	A-8b

SYMBOL	DESCRIPTION	ODOT CLASSIFICATION
	Shale	Visual
	Weathered Shale	Visual
	Sandstone	Visual
	Siltstone	Visual
	Limestone	Visual
	Interbedded Sandstone and Shale	Visual
	Dolomite	Visual

SAMPLER SYMBOLS

- Shelby Tube
- Rock Core
- Split Spoon Sample (SS)
- * Indicates a Sample Taken Within 3 ft of Proposed Grade

GRADATION (%)

GR Gravel
CS Coarse Sand
MS Medium Sand
FS Fine Sand
SI Silt
CL Clay (<5 micron)

ABBREVIATIONS

LL	LIQUID LIMIT (%)	HP	HAND PENETROMETER
PI	PLASTIC INDEX (%0	TR	TOP OF ROCK
WC	MOISTURE CONTENT (%)	UC	UNCONFINED COMPRESSION
SPT	STANDARD PENETRATION TEST	ppm	PARTS PER MILLION
NP	NON PLASTIC	*w*	WATER FIRST ENCOUNTERED
-200	PERCENT PASSING NO. 200 SIEVE	▼	WATER LEVEL UPON COMPLETION
N_{60}	ADJUSTED SPT RESULT	Rec	RECOVERY %
EOB	END OF BORING	RQD	ROCK QUALITY DESIGINATION %

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Concrete
- Uncontrolled Fill (Describe)
- Bouldery Zone
- Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc

PROJECT: FRA-71-00.00	DRILLING FIRM / OPERATOR: CENTRAL STAR / MJ	DRILL RIG: CME 55 (CS)	STATION / OFFSET: 81+23, 5 RT	EXPLORATION ID B-021-1-14
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: B&P / Z. JEWELL	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. IR 71	
PID: 93496 BR ID: FRA-71-0153	DRILLING METHOD: 2.25" HSA / NQ2	CALIBRATION DATE: 6/12/12	ELEVATION: 797.0 (MSL) EOB: 72.4 ft.	PAGE 1 OF 3
START: 4/1/14 END: 4/1/14	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 74.9	LAT / LONG: 39.821008370, -83.169799670	

MATERIAL DESCRIPTION AND NOTES	ELEV. 797.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
12.0", **TOPSOIL**	795.8																	
HARD, BROWN, **SANDY SILT**, SOME GRAVEL, LITTLE CLAY, CONTAINS LARGE ROOT, DAMP **(FILL)**	794.0	1 – 2	3 8 13	26	22	SS-1	4.50	30	11	12	28	19	25	15	10	10	A-4a (2)	
MEDIUM STIFF TO VERY STIFF, BROWN, **SILT AND CLAY**, SOME SAND, TRACE TO LITTLE GRAVEL, DAMP **(FILL)**		3 – 5	4 2 2	5	100	SS-2	.75 - 2.25	10	12	16	34	28	30	16	14	16	A-6a (7)	
@6.0'; SS-3 HARD		6 – 7	21 18 25	54	33	SS-3	4.5+	-	-	-	-	-	-	-	-	10	A-6a (V)	
@8.5'; SS-4 NO RECOVERY		8 – 10	9 11 9	25	0	SS-4	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
	784.0	11 – 12	8 8 8	20	67	SS-5	2.25 - 2.5	10	12	15	35	28	28	15	13	13	A-6a (7)	
VERY STIFF, DARK GRAY, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, CONTAINS SHELLS AND DECAYING WOOD, DAMP TO MOIST		13 – 15	10 12 10	27	50	SS-6	2.0 - 3.25	-	-	-	-	-	-	-	-	18	A-6b (V)	
@16.0'; SS-7 NO LONGER FILL	779.0	16 – 17, W 18	6 8 7	19	67	SS-7	2.25 - 2.5	1	2	28	37	32	37	21	16	22	A-6b (9)	
SOFT TO MEDIUM STIFF, DARK GRAY, **SILT AND CLAY**, "AND" SAND, TRACE GRAVEL, CONTAINS SHELLS AND ROOTS, MOIST	776.5	18 – 20	3 2 5	9	100	SS-8	0.50	1	4	44	30	21	32	19	13	28	A-6a (4)	
DENSE, LIGHT BROWN WITH BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP	774.0	21 – 22	18 19 13	40	39	SS-9	-	38	22	13	23	4	NP	NP	NP	10	A-2-4 (0)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP		23 – 25	5 8 13	26	44	SS-10	-	41	22	16	18	3	NP	NP	NP	9	A-1-b (0)	
	769.0	26 – 28	11 16 22	47	44	SS-11	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
VERY DENSE, GRAYISH BROWN MOTTLED WITH GRAY, **SANDY SILT**, TRACE CLAY, TRACE GRAVEL, DAMP		29	40 50/5"	-	100	SS-12	-	-	-	-	-	-	-	-	-	9	A-4a (V)	

MATERIAL DESCRIPTION AND NOTES	ELEV. 767.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	765.5	31 TR																
LIMESTONE, LIGHT GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, THICK BEDDED, FOSSILIFEROUS, PARTLY VUGGY, CLAY LAYERS- WASHED OUT DURING DRILLING, JOINT DISCONTINUITIES: LOW AND DIAGONAL FRACTURES WITH HIGH ANGLE FRACTURE AT 43.0' TO 43.5', MODERATELY FRACTURED, NARROW TO OPEN, SLIGHTLY TO VERY ROUGH; RQD 16.6%, REC 38.7%.		32	59		98	NQ2-1											CORE	
		33–37	28		56	NQ2-2											CORE	
		38–42	14		25	NQ2-3											CORE	
	749.6	43–47	0		24	NQ2-4											CORE	
SILTSTONE, GRAYISH BROWN, UNWEATHERED, SLIGHTLY STRONG, VERY THICK BEDDED, FEW IRON STAINED CRACKS, JOINT DISCONTINUITIES: LOW ANGLE FRACTURES WITH HIGH ANGLE FRACTURES AT 47.8' TO 48.4' AND 48.7' TO 49.2', MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 68.2%, REC 96.1%.	744.2	48–52	66		96	NQ2-5											CORE	
DOLOMITE, GRAYISH BROWN, SLIGHTLY WEATHERED, MODERATELY STRONG, MEDIUM BEDDED, FOSSILIFEROUS- INCREASES WITH DEPTH, PARTLY VUGGY, IRON STAINS, BEDDING DISCONTINUITIES: LOW ANGLE FRACTURES WITH HIGH ANGLES FRACTURES AT 55.4' TO 55.7' AND 57.3' TO 57.5', MODERATELY FRACTURED, NARROW; RQD 57.4%, REC 100%.		53–57	83		100	NQ2-6											CORE	
		58–61	45		100	NQ2-7											CORE	

PID: 93496	BR ID: FRA-71-0153	PROJECT: FRA-71-00.00	STATION / OFFSET: 81+23, 5 RT	START: 4/1/14	END: 4/1/14	PG 3 OF 3	B-021-1-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 734.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	732.0	63–65	48		100	NQ2-8											CORE	
SILTSTONE, GRAY, UNWEATHERED, MODERATELY STRONG, VERY THICK BEDDED, JOINT DISCONTINUITIES: LOW ANGLE FRACTURE, FRACTURED TO MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 54%, REC 97.2%.	729.3	65–68																
DOLOMITE, GRAY, MODERATELY WEATHERED, SLIGHTLY STRONG, THICK BEDDED, VUGGY, CONTAINS IRON STAINING, JOINT DISCONTINUITIES: LOW ANGLE FRACTURES WITH DIAGONAL TO HIGH ANGLE FRACTURE AT 70.6' TO 72.4', MODERATELY FRACTURED TO FRACTURED, NARROW TO OPEN, SLIGHTLY ROUGH; RQD 42.6%, REC 94.9%.	724.6	68–72, EOB	40		95	NQ2-9											CORE	

NOTES: GROUNDWATER ENCOUNTERED AT 18.0' DURING DRILLING. CAVE DEPTH 23.5'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 20 GAL. BENTONITE GROUT

BARR ENGINEERING, INC.

Rock Core B-021-1-14
NQ2 - 1 and 2

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-1-14

NQ2 - 3, 4 and 5

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496

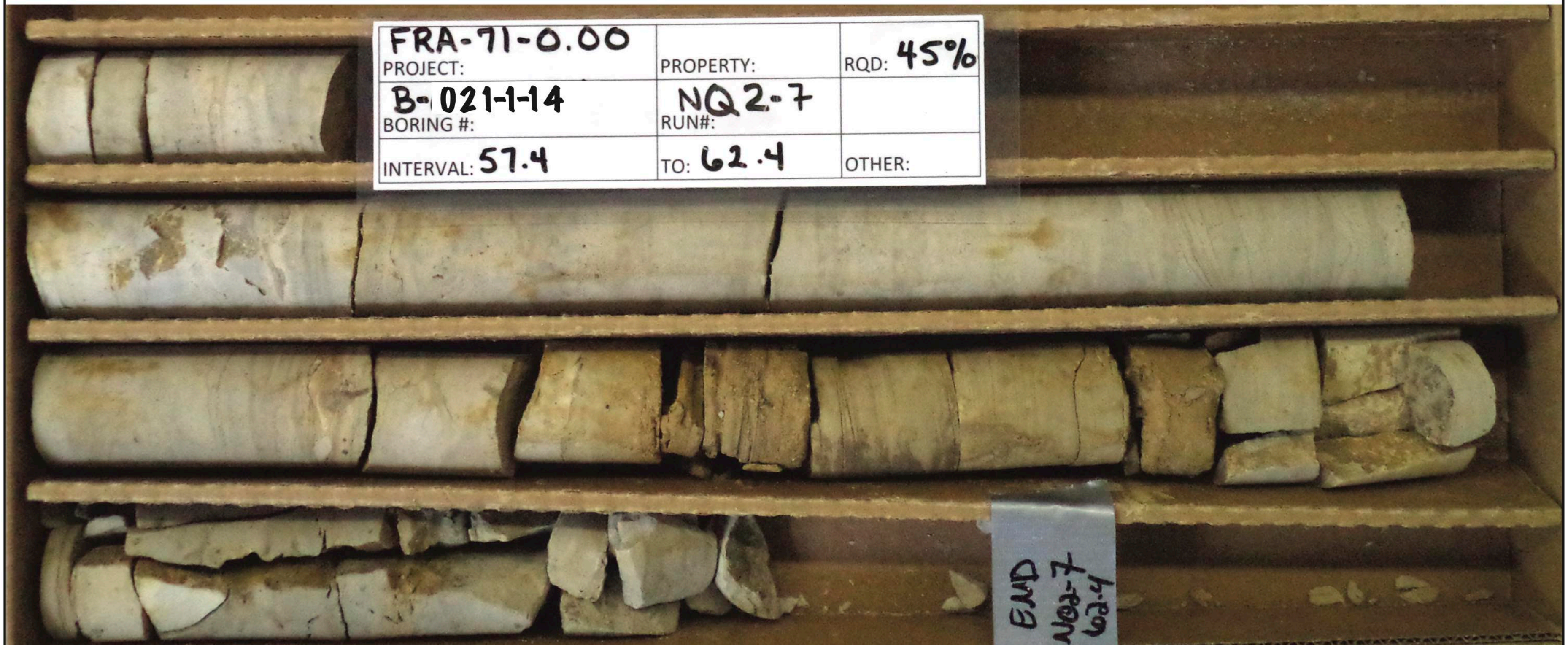
BARR ENGINEERING, INC.

Rock Core B-021-1-14

NQ2 - 6

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496



BARR ENGINEERING, INC.

Rock Core B-021-1-14
NQ2 - 7

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

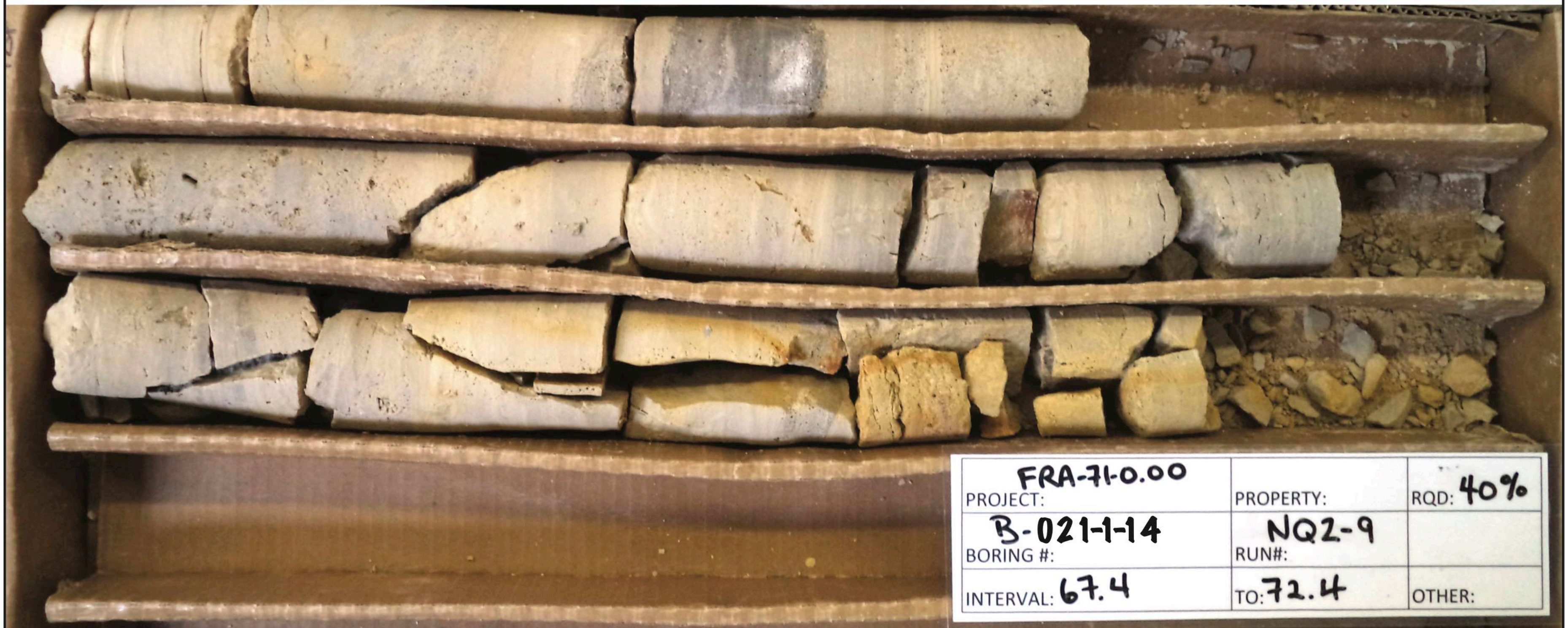
BARR ENGINEERING, INC.

Rock Core B-021-1-14

NQ2 - 8

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496



BARR ENGINEERING, INC.

Rock Core B-021-1-14
NQ2 - 9

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

PROJECT: FRA-71-00.00	DRILLING FIRM / OPERATOR: CENTRAL STAR / MJ	DRILL RIG: CME 55 (CS)	STATION / OFFSET: 85+29, 6 RT	EXPLORATION ID B-021-2-14
TYPE: BRIDGE	SAMPLING FIRM / LOGGER: B&P / Z. JEWELL	HAMMER: CME AUTOMATIC	ALIGNMENT: CL CONST. IR 71	
PID: 93496 BR ID: FRA-71-0153	DRILLING METHOD: 2.25" HSA / NQ2	CALIBRATION DATE: 6/12/12	ELEVATION: 798.0 (MSL) EOB: 62.0 ft.	PAGE 1 OF 2
START: 3/31/14 END: 3/31/14	SAMPLING METHOD: SPT / NQ2	ENERGY RATIO (%): 74.9	LAT / LONG: 39.821528460, -83.168521930	

MATERIAL DESCRIPTION AND NOTES	ELEV. 798.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
14.0", **TOPSOIL**	796.6	1																
VERY STIFF TO HARD, GRAY WITH GRAYISH BROWN, **SILT AND CLAY**, SOME SAND, LITTLE GRAVEL, SS-1 FEW ROOT HAIRS, DAMP		2	3 3 5	10	100	SS-1	2.5 - 3.5	11	14	16	35	24	27	14	13	11	A-6a (6)	
@3.5'; BECOMES GRAY		3, 4, 5	5 4 3	9	94	SS-2	4.00	-	-	-	-	-	-	-	-	13	A-6a (V)	
	790.0	6, 7, 8	5 7 12	24	100	SS-3	4.5+	-	-	-	-	-	-	-	-	11	A-6a (V)	
VERY DENSE, REDDISH BROWN, **SANDY SILT**, "AND" GRAVEL, LITTLE CLAY, DAMP **(Possible cuttings)**	787.5	9, 10	18 20 24	55	33	SS-4	-	-	-	-	-	-	-	-	-	9	A-4a (V)	
VERY STIFF, GRAYISH BROWN MOTTLED WITH DARK GRAY, **SILTY CLAY**, SOME SAND, LITTLE GRAVEL, DAMP	785.0	11, 12, 13	5 7 10	21	100	SS-5	3.0 - 3.25	14	12	20	27	27	38	18	20	17	A-6b (8)	
MEDIUM DENSE TO DENSE, DARK GRAYISH BROWN, **SANDY SILT**, TRACE TO LITTLE CLAY, TRACE TO LITTLE GRAVEL, DAMP TO WET		14, 15	10 8 19	34	44	SS-6	-	-	-	-	-	-	-	-	-	13	A-4a (V)	
@16.0'; SS-7 BECOMES DARK GRAY, CONTAINS WOOD FRAGMENTS	780.0	16, 17, 18	11 8 6	17	100	SS-7	-	2	11	39	29	19	NP	NP	NP	26	A-4a (3)	
MEDIUM STIFF TO STIFF, DARK GRAY, **SILT AND CLAY**, "AND" SAND, TRACE GRAVEL, CONTAINS SHELLS AND WOOD FRAGMENTS, MOIST	778.5	19	8 15 10	31	100	SS-8	1.00	10	22	28	20	20	33	19	14	20	A-6a (2)	
DENSE, GRAYISH BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST TO WET		20					-	45	21	11	15	8	NP	NP	NP	14	A-1-b (0)	
		21, 22, 23	10 12 16	35	100	SS-9	-	28	38	15	13	6	NP	NP	NP	13	A-1-b (0)	
	772.5	24, 25	9 12 13	31	33	SS-10	-	8	50	24	12	6	NP	NP	NP	18	A-1-b (0)	
HARD, GRAY, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, DAMP	770.0	26, 27, 28	13 17 17	42	100	SS-11	4.5+	13	12	20	35	20	20	13	7	10	A-4a (4)	
VERY DENSE, GRAY, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP		29	32 50/5"	-	45	SS-12	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 2/15/17 12:15 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\1ARCHIVE BY YEAR\2015 ARCHIVE\FRA-71-0.00 2014-15\COMBINE

PID: 93496	BR ID: FRA-71-0153	PROJECT: FRA-71-00.00	STATION / OFFSET: 85+29, 6 RT	START: 3/31/14	END: 3/31/14	PG 2 OF 2	B-021-2-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 768.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	767.5																	
DARK GRAY, **LIMESTONE BOULDER**	765.3	TR 31–32	50/1"	-	0	SS-13	-	-	-	-	-	-	-	-	-	-		
LIMESTONE, LIGHT GRAYISH BROWN, BECOMES GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG, MEDIUM BEDDED, CONGLOMERITIC, JOINT DISCONTINUITIES: LOW ANGLE FRACTURES WITH HIGH ANGLE FRACTURES AT 32.3' TO 33.0', 36.5' TO 36.8', AND 39.3' TO 40.0', FRACTURED TO MODERATELY FRACTURED, TIGHT TO NARROW, VERY ROUGH; RQD 33.7%, REC 95.2%.		32–37	14		90	NQ2-1											CORE	
	755.0	37–42	42		100	NQ2-2											CORE	
DOLOMITE, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG, THICK BEDDED, FEW VUGGY ZONES, FEW FOSSILS, CONGOLOMERATIC AT 43.5' TO 43.7', JOINT DISCONTINUTIES: LOW ANGLE FRACTURES WITH HIGH ANGLE FRACTURES AT 52.8' TO 53.1', MODERATELY FRACTURED, NARROW, SLIGHTLY ROUGH; RQD 63.6%, REC 97.4%.		42–47	62		100	NQ2-3											CORE	
48.1' to 48.3' Clay Seam		47–52	71		97	NQ2-4											CORE	
		52–57	67		100	NQ2-5											CORE	
56.4' to 56.5' Shale Layer																		
59.8' to 60.0' Shale Layer	736.0	57–62 EOB	56		93	NQ2-6											CORE	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 20 GAL. BENTONITE GROUT

BARR ENGINEERING, INC.

Rock Core B-021-2-14

NQ2 - 1

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-2-14
NQ2 - 2

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-2-14

NQ2 - 3

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-2-14
NQ2 - 4

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-2-14
NQ2 - 5

FRA-71-0.00 Bridge FRA-71-0153
PID#: 93496

BARR ENGINEERING, INC.

Rock Core B-021-2-14

NQ2 - 6

FRA-71-0.00 Bridge FRA-71-0153

PID#: 93496

B-001-B-62

Form No. 530—16-

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 7/20/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE None CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/20/62 CASING: LENGTH & NXM Core Barrel DIA. 3.5"I.D. AFTER 24 HOURS Hole Caved at 779.3. PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 1 STATION AND OFFSET 921+94, 86' L. of C.L. of SR-1 SURFACE ELEV. 783.3

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
783.3	0													
781.8	2	1	7-6-9	15"	Topsoil.	No Tests Performed								
778.3	4	2	10-11-12	10"	Dark brown silt and clay, some sand, some gravel, moist - stiff.	24	8	25	27	16	32	13	15	A-6a
	6	3	17-21-21	14"	Brown sandy gravel, little clay, wet - dense.	76	9	3	- 10 -				8	A-1-a
	8 10	4	13-13-17	10"	Brown sandy gravel, trace of silt, wet - medium dense.	79	10	3	- 8 -				8	A-1-a
	12	5	12-16-17	8"	Brown sandy gravel, trace of silt, wet - dense.	77	10	5	- 8 -				9	A-1-a
770.8 770.3		6	14	4"	Brn. gravelly sand, little silt, moist-medium dense.	57	17	8	17	1	17	2	13	A-1-b
	14	7	17-25	8"	Brown sandy silt, some gravel, moist - stiff.	35	13	15	24	13	24	10	12	A-4a
768.3	16	8	11-21-26	14"	Gray and brown clay, trace of sand, trace rock fragments, moist - stiff. (fat)	4	4	4	15	73	52	29	21	A-7-6
765.8	18	9	70	*	Brown sandy gravel. (Field Classification)									A-1-a
	20	10	50	4"	Brown sandy gravel, little clay, wet - very dense. (Visual)	Insufficient Sample For Testing.								A-1-a
762.3	22 24	11	NXM	88%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1/2" to 9", averaging 5"); evidence of high angle jointing.									
757.3	26				Boring Completed.									
	28													
	30													
	32													
	34													

B-002-B-62

Form No. 530—16-5

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 7/24/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE 776.6 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/24/62 CASING: LENGTH ____ DIA. 3.5"I.D. AFTER 24 HOURS 776.6 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 2 STATION AND OFFSET 922+88, 46' L. of C.L. of SR-1 SURFACE ELEV. 774.1 (Creek Bed Elevation)

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
776.6	0													
774.1	2				WATER									
	4	1	7-8-11	10"	Brown sandy gravel, little silt, wet - medium dense.	71	12	6	- 11 -				10	A-1-a
770.1	6	2	15-15-13	8"	Brown sandy gravel, little silt, wet - dense.	70	13	6	- 11 -				10	A-1-a
	8	3	16-18-22	6"	Reddish brown and gray clayey gravel, with sand, moist - stiff.	45	11	12	27	5	29	16	15	A-2-6
766.6	10													
	12	4	10-8-12	12"	Reddish brown clay, some sand, trace of gravel, moist - stiff.	7	11	11	12	59	74	48	38	A-7-6
762.6	14	5	14-13-14	10"	Reddish brown clay, some sand, little gravel, moist - stiff.	11	20	12	9	48	61	43	32	A-7-6
	16	6	NXM	74%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1" to 8", averaging about 5" with the thinner beds near top of core); 2" broken zone 3" below top of core, and more broken pieces near the bottom of the last run (Sample No. 8).									
	18	7	NXM	100%										
	20													
	22	8	NXM	34%										
752.6	24													
	26				Boring Completed.									
	28													
	30													
	32													
	34													

Form No. 530—16-59

THE H. C. NUTTING COMPANY

CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-003-B-62

LOG OF BORING

DATE STARTED 7/26/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE 777.1 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/26/62 CASING: LENGTH ______ DIA. 3.5"I.D. AFTER 24 HOURS 776.8 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 3 STATION AND OFFSET 924+13, 103' L. of C.L. of SR-1 SURFACE ELEV. 783.1

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
						Physical Characteristics								
783.1	0													
781.1	2	1	1-2-2	10"	Topsoil.	No Tests Performed.								
	4	2	9-8-7	12"	Brown sandy gravel, trace of silt, dry - medium dense.	68	20	5	- 7	-			8	A-1-a
	6	3	6-5-5	10"	Brown sandy gravel, trace of silt, moist - loose.	71	17	4	- 8	-			6	A-1-a
	8													
773.6	10	4	12-12-13	12"	Brown sandy gravel, little silt, wet - medium dense.	69	13	6	- 12	-			9	A-1-a
771.1	12	5	8-9-10	16"	Brown sandy silt, some gravel, moist - stiff.	22	14	16	32	16	23	10	13	A-4a
	14	6	8-11-16	12"	Brownish gray silt and clay, some sand, little gravel, moist - med.stiff.	17	11	18	38	16	24	11	14	A-6a
767.1	16	7	80	*	Brownish gray silt and clay. (Field Classification)									A-6a
	18													
	20	8	NXM	100%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1/2" to 7", averaging 2½" near top to 5" near bottom of core). 4" broken zone at depth of 5-ft. (at top of 2nd run or Sample No. 9), probably high angle joint.									
	22													
	24	9	NXM	90%										
757.1	26													
	28				Boring Completed.									
	30													
	32													
	34													

*No Recovery.

Form No. 530—16-5

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-004-B-62

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 7/26/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE 777.5 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/26/62 & NXM Core Barrel CASING: LENGTH DIA. 3.5"I.D. AFTER 24 HOURS 777.8 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 4 STATION AND OFFSET 925+10, 57' L. of C.L. of SR-1 SURFACE ELEV. 784.5

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
784.5	0													
782.5	2	1	1-1-2	8"	Topsoil.	No Tests Performed.								
779.5	4	2	7-5-3	10"	Brown sandy gravel, little silt,	55	23	12	- 10 -				5	A-1-a
777.5	6	3	5-5-5	14"	Brown clayey sand, some gravel, moist - loose.	24	23	29	19	5	29	11	19	A-2-6
	8													
774.5	10	4	12-12-22	10"	Brown sandy gravel, little silt, wet - dense.	63	15	8	- 14 -				9	A-1-a
772.5	12	5	18-21-24	6"	Brown gravelly sand, little silt, wet - dense.	58	15	7	18	2	17	2	11	A-1-b
770.0	14	6	14-20-22	8"	Brown sandy silt, little gravel, moist - stiff.	12	12	20	41	15	23	10	10	A-4a
767.5	16	7	21-24-26	14"	Brownish gray silt and clay, some sand, little gravel, moist - stiff.	10	9	16	36	29	27	13	11	A-6a
765.5	18	8	21-26-26	10"	Brown sandy silt, trace of gravel, moist - medium stiff.	8	5	21	38	28	23	10	14	A-4a
764.0	20	9	14	4"	Reddish brown gravelly sand, little silt, moist - medium dense.	71	7	3	- 19 -					A-1-b
	22	10	7-11	10"	Reddish brown elastic clay, moist- stiff.	0	0	0	8	92	129	86	41	A-7-5
760.5	24	11	8-11-16	15"	Reddish brown elastic clay, trace of sand, moist - stiff.	0	0	1	9	90	124	83	42	A-7-5
755.5	26 28	12	NXM	76%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1" to 4", averaging 3"), with 1/4" disconnected solution openings in upper 1½-ft. of core.									
	30 32 34				Boring Completed.									

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-005-B-62

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 7/24/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE 777.4 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/24/62 CASING: LENGTH ___ DIA. 3.5"I.D. AFTER ___ HOURS ___ PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 5 STATION AND OFFSET 921+90, 31' R. of C.L. of SR-1 SURFACE ELEV. 781.4

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
781.4	0													
		1	2-3-4	7"	Topsoil.	No Tests Performed.								
778.9	2													
	4	2	14-15-14	6"	Brown sandy gravel, little silt, moist - medium dense.	66	15	6	- 13 -				6	A-1-a
774.4	6	3	2-3-6	10"	Brown sandy gravel, trace of silt, wet - loose.	70	19	5	- 6 -				11	A-1-a
	8													
771.9		4	11-16-21	14"	Gray sandy silt, little gravel, moist - dense.	18	7	24	44	7	16	3	12	A-4a
770.4	10	5	35-62	7"	Brown silt and clay, some sand, little gravel, moist - stiff.	15	10	18	43	14	25	12	10	A-6a
	12													
		6	90	4"	Brownish gray sandy silt, some gravel, moist - stiff.	21	13	20	32	14	19	6	10	A-4a
	14													
	16	7	80	3"	do do do	(Visual) - Insufficient Sample.								A-4a
764.4														
	18													
	20	8	NXM	76%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1" to 7", averaging 2½" near top of core and 4" in bottom half). Top 1-ft. of core broken.									
	22													
	24	9	NXM	96%										
754.4	26	10	NXM	78%										
	28				Boring Completed.									
	30													
	32													
	34													

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-006-B-62

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 7/25/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE 775.6 CLIENT Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/25/62 CASING: LENGTH ______ DIA. 3.5"I.D. AFTER ______ HOURS ______ PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 6 STATION AND OFFSET 922+85, 69' R. of C.L. of SR-1 SURFACE ELEV. 777.6

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
777.6	0													
	2	1	5-6-8	14"	Brown sandy gravel, little silt, wet - medium dense.	65	17	7	- 11 -				8	A-1-a
774.6		2	9	6"	Brown sandy gravel, trace of silt, wet - medium dense.	64	23	6	- 7 -				12	A-1-a
772.6	4	3	5-6	8"	Gray sandy silt, little gravel, moist - medium stiff.	16	11	21	41	11	17	4	10	A-4a
	6	4	11-20-22	8"	Brown silt and clay, some sand, little gravel, moist - stiff.	14	8	16	35	27	27	13	12	A-6a
	8	5	10-16-21	10"	do do do	11	9	17	36	27	27	14	10	A-6a
767.6	10													
	12	6	NXM	62%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1/2" to 6", averaging 1½" in top 5-ft. and 3" in bottom 5-ft. of core). Broken zone near top of core and again near bottom of second run (Sample No. 7); bottom two runs give evidence of high angle jointing.									
	14	7	NXM	68%										
	16													
	18	8	NXM	80%										
757.6	20	9	NXM	100%										
	22				Boring Completed.									
	24													
	26													
	28													
	30													
	32													
	34													

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

B-007-B-62

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 8/11/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE None CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 8/11/62 CASING: LENGTH & NXM Core Barrel DIA. 3.5"I.D. AFTER 24 HOURS 777.0 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 7 STATION AND OFFSET 924+13, 31' R. of C.L. of SR-1 SURFACE ELEV. 785.0

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
785.0	0													
783.0	2	1	2-5-8	12"	Brown silty sand and gravel, moist - medium dense.	46	20	15	12	7	28	10	12	A-2-4
	4	2	5-4-5	5"	Brown sandy gravel, trace of silt, moist - loose.	71	14	6	- 9 -				5	A-1-a
	6	3	11-11-15	16"	Brown sandy gravel, little silt, moist - medium dense.	56	21	10	- 13 -				4	A-1-a
	8	4	11-14-18	11"	Brown sandy gravel, trace of silt, moist - dense.	72	12	6	- 10 -				4	A-1-a
	10, 12	5	17-24-27	14"	Brown sandy gravel, trace of silt, wet - very dense.	74	9	8	- 9 -				8	A-1-a
770.0	14	6	16-18-19	10"	Brown sandy gravel, little silt, wet - dense.	68	12	7	- 13 -				8	A-1-a
	16	6A	NXM	25%	Dolomite, white, crystalline, dense, hard, broken, evidence of vertical joint.									
	18	7	-35-	4"	Reddish brn. silty clay, some sand, some gravel.*	31	10	13	23	23	38	25	14	A-6b
		8	-72-	6"	Brownish gray silty gravel, some sand.*	46	15	12	18	9	20	7	9	A-2-4
	20, 22	9	NXM	25%	Dolomite, white, crystalline, dense, hard, broken, evidence of vertical joint.									
	24	10	3-5-8	4"	Reddish brown gravel and rock fragments, trace of sand, some silt. *	68	7	2	- 23 -				51	A-1-b
	26, 28	11	NXM	30%	Dolomite, white, crystalline, medium dense, hard, broken, evidence of vertical joint.									
		12	- 0 -	6"	Reddish brown elastic clay. *	1	2	1	11	85	126	85	70	A-7-5
	30, 32	13	NXM	96%	Dolomite, cream white, crystalline, fine grained, dense, hard, thinly bedded, (1" to 6", averaging 3"). Evidence of high angle jointing near middle of sample No. 14.									
	34	14												

* = Soils contained in vertical joint or crack in bedrock.

Form No. 530—16-5

THE H. C. NUTTING COMPANY

[illegible] ROAD
CINCINNATI 26, OHIO

B-007-B-62

TESTING ENGINEERS AND SOILS CONSULTANTS

LOG OF BORING

DATE STARTED 8/11/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE None CLIENT: Barrett-Cargo-Wither & Assoc.

DATE COMPLETED 8/11/62 CASING: LENGTH ______ DIA. 3.5"I.D. AFTER 24 HOURS 777.0 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 7 STATION AND OFFSET 924+13, 31' R. of C.L. of SR-1 SURFACE ELEV. 785.0

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
	34													
	36	14 (Continued)			Dolomite. (See description on previous page.)									
746.5	38	15												
	40				Boring Completed.									
	42													
	44													

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-008-B-62

LOG OF BORING

DATE STARTED 7/25/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE 776.6 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 7/25/62 & NXM Core Barrel CASING: LENGTH ____ DIA. 3.5"I.D. AFTER 24 HOURS 777.3 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 8 STATION AND OFFSET 925+11, 58' R. of C.L. of SR-1 SURFACE ELEV. 784.6

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
784.6	0													
783.1	2	1	2-3-5	10"	Topsoil.	No Tests Performed.								
	4	2	3-3-3	12"	Brown silty sand, some gravel, moist - loose.	23	14	36	19	8	23	7	19	A-2-4
777.6	6	3	3-3-2	10"	Brown and gray silt sand, trace of gravel, moist - loose.	4	16	46	25	9	31	9	47	A-2-4
	8	4	3-5-9	10"	Brown and gray gravelly sand, some silt, moist - medium dense.	31	21	27	16	5	28	6	15	A-1-b
772.6	10, 12	5	26-18-18	12"	Brown gravelly sand, little silt, wet - dense.	63	13	7	- 17 -				11	A-1-b
769.6	14	6	15-15-19	14"	Gray sandy silt, some gravel, moist - stiff.	22	16	18	31	13	19	7	8	A-4a
767.6	16	7	13-15-21	15"	Brown silty sand and gravel, moist - dense.	37	13	15	24	11	20	8	8	A-2-4
764.6	18, 20	8	34-37-45	10"	Brown gravelly sand, little silt, wet - very dense.	46	18	21	- 15 -				11	A-1-b
	22	9	NXM	100%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (½" to 4", averaging 2"). Two 1" broken zones near center of run 2 (Sample No. 10).									
	24	10	NXM	100%										
759.6		11	NXM	92%										
	26				Boring Completed.									
	28													
	30													
	32													
	34													

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-009-B-62

LOG OF BORING

DATE STARTED 8/9/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA 2" O.D. WATER ELEV. IMMEDIATE 776.6 CLIENT Barrett-Cargo-Winhers & Assoc.

DATE COMPLETED 8/10/62 CASING: LENGTH ______ DIA. 3.5" I.D. AFTER 48 HOURS 776.6 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 9 STATION AND OFFSET 922+88, 91' L. of C.L. of SR-1 SURFACE ELEV. 774.5 (Creek Bed Elevation)

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
776.6	0													
774.5	2				WATER									
	4	1	8-10-15	12"	Brown sandy gravel, little silt, wet - medium dense.	67	15	7	- 11	-			9	A-1-a
	6	2	10-14-12	14"	do do do	67	17	6	- 10	-			11	A-1-a
768.1	8	3	35-68	10"	Brown sandy gravel, trace of silt, wet - very dense.	68	19	5	- 8	-			9	A-1-a
	10	4	NXM	78%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1" to 8", averaging 2" in top half and 5" in bottom half of 10' core). Several broken zones at top and center of run 2 (Sample No. 5), probably high angle joints.									
	12	5	NXM	84%										
	14													
	16	6	NXM	100%										
758.1	18													
	20				Boring Completed.									
	22													
	24													
	26													
	28													
	30													
	32													
	34													

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THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-010-B-62

LOG OF BORING

DATE STARTED 8/12/62 SAMPLER: TYPE Split Spoon & NXM Core Barrel DIA. 2"O.D. WATER ELEV. IMMEDIATE 776.1 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 8/12/62 CASING: LENGTH ______ DIA. 3.5"I.D. AFTER ______ HOURS ______ PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 10 STATION AND OFFSET 924+13, 39' L. of C.L. of SR-1 SURFACE ELEV. 783.1

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	% AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
783.1	0													
781.1	2	1	2-3-4	10"	Brown silty sand and gravel, moist - loose.	35	25	17	15	8	26	10	8	A-2-4
778.6	4	2	10-11-11	14"	Brown sandy gravel, little silt, moist - medium dense.	61	17	9	9	4	19	4	7	A-1-a
776.1	6	3	19-15-15	10"	Brown gravelly sand, some silt, moist - medium dense.	44	20	15	16	5	21	6	8	A-1-b
	8	4	16-18-29	8"	Brown sandy gravel, little silt, moist - dense.	62	17	8	- 13 -				5	A-1-a
772.1	10	5	25-22	8"	do do do	69	13	6	10	2	17	3	13	A-1-a
	12	6	15	6"	Brown silt and clay, some sand, little gravel, moist - stiff.	17	9	17	32	25	26	13	12	A-6a
768.6	14	7	30-26-21	8"	Brown silt and clay and gravel, little sand, moist - medium stiff.	43	6	9	23	19	27	13	14	A-6a
767.1	16	8	6		Reddish brown clay, little sand, some gravel, moist - medium stiff.	21	7	7	14	51	77	51	23	A-7-6
	18													
	20	9	NXM	100%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (1" to 10", averaging 2" in top half and 5" in bottom half of 10' core). Traces of red clay on some bedding planes in the top run (Sample No. 9).									
	22	10	NXM	100%										
	24	11	NXM	100%										
757.1	26													
	28				Boring Completed.									
	30													
	32													
	34													

Form No. 530—16-5

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-011-B-62

LOG OF BORING

DATE STARTED 8/10/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE 776.6 CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 8/10/62 & NXM Core Barrel CASING: LENGTH DIA. 3.5"I.D. AFTER 48 HOURS 776.6 PROJECT: Interstate 71 Bridge No. FRA-1-0153 (R&L)

BORING No. 11 STATION AND OFFSET 922+88, 24' R. of C.L. of SR-1 SURFACE ELEV. 776.8

Elev.	Depth	Sample No.	Std. Pen. (N)	% Rec.	Description	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class
776.8	0													
	2	1	3-8-22	14"	Brown sandy gravel, little silt, wet - dense.	71	14	5	- 1	0 -			10	A-1-a
	4	2	16-20-23	15"	do do do	62	16	7	12	3	20	4	12	A-1-a
770.8	6	3	NXM	33%	do do do (Visual)	(Boulder)								A-1-a
768.3	8	4	29-27-19	16"	Brown silty sand and gravel, moist - dense.	36	17	16	23	8	20	5	10	A-2-4
766.8	10	5	26-38	8"	Brown silty clay and gravel, some sand, moist - stiff.	37	17	10	20	16	31	17	15	A-6b
	12													
	14	6	NXM	100%	Dolomite, cream white, crystalline, fine-grained, dense, hard, thinly bedded (½" to 7", averaging 2½" in top half and 4" in bottom half of 10' core). Upper run (Sample No. 6) rather porous below top of core.									
	16													
	18	7	NXM	100%										
756.8	20													
	22				Boring Completed.									
	24													
	26													
	28													
	30													
	32													
	34													

THE H. C. NUTTING COMPANY

4120 AIRPORT ROAD
CINCINNATI 26, OHIO

TESTING ENGINEERS AND SOILS CONSULTANTS

B-012-B-62

LOG OF BORING

DATE STARTED 8/10/62 SAMPLER: TYPE Split Spoon DIA. 2"O.D. WATER ELEV. IMMEDIATE None CLIENT: Barrett-Cargo-Withers & Assoc.

DATE COMPLETED 8/11/62 CASING: LENGTH & NXM Core Barrel DIA. 3.5"I.D. AFTER 24 HOURS None PROJECT: Interstate 71
Bridge No. FRA-1-0153 (R&L)

BORING No. 12 STATION AND OFFSET 924+13, 66' R. of C.L. of SR-1 SURFACE ELEV. 783.8

ELEV.	DEPTH	SAMPLE No.	STD. PEN. (N)	% REC.	DESCRIPTION	Physical Characteristics % AGG.	% C.S.	% F.S.	% SILT	% CLAY	L.L.	P.I.	W.C.	SHTL CLASS
783.8	0													
781.8	2	1	3-3-4	10"	Brown silty sand and gravel, moist - loose.	42	21	15	15	7	26	8	7	A-2-4
779.3	4	2	6-6-6	16"	Brown gravelly sand, little silt, moist - medium dense.	43	29	14	-1	4 -			5	A-1-b
	6	3	12-22-35	10"	Brown sandy gravel, little silt, moist - very dense.	56	20	11	-1	3 -			4	A-1-a
774.3	8 / 10	4	26-12-12	16"	Brown sandy gravel, little silt, moist - medium dense.	56	21	8	-1	5 -			4	A-1-a
771.8	12	5	21-21-20	12"	Brown gravelly sand, little silt, wet - dense.	60	14	8	-1	8 -			10	A-1-b
769.3	14	6	16-17-18	15"	Brown sandy gravel, little silt, wet - dense.	68	12	6	-1	4 -			10	A-1-a
766.8	16	7	13-30-33	12"	Brown silt and clay, some sand, trace of gravel, moist - stiff.	8	7	18	36	31	28	14	13	A-6a
	18 / 20 / 22	8	NXM	78%	Dolomite, cream white, crystalline, medium-fine grained, dense, hard, thinly bedded (½" to 6", averaging 2" in upper half and 4" in lower half of 10' core). Upper 1-ft. of first run (Sample No. 8) and upper 4-inches of second run (Sample No. 9) are broken.									
	24	9	NXM	100%										
756.8	26	10	NXM	73%										
	28				Boring Completed.									
	30													
	32													
	34													

APPENDIX B

CALCULATIONS

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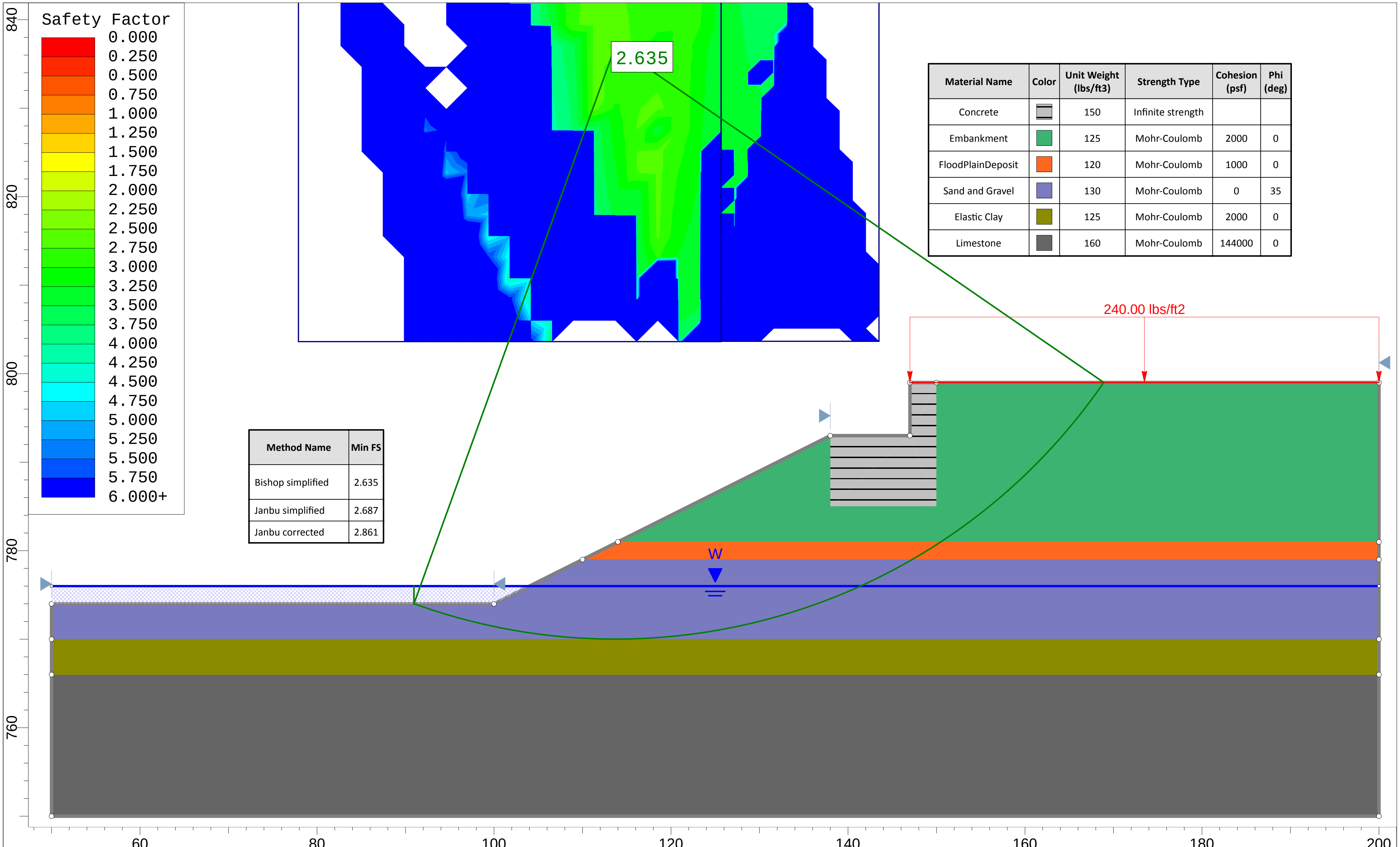
APPENDIX B-1

GLOBAL STABILITY

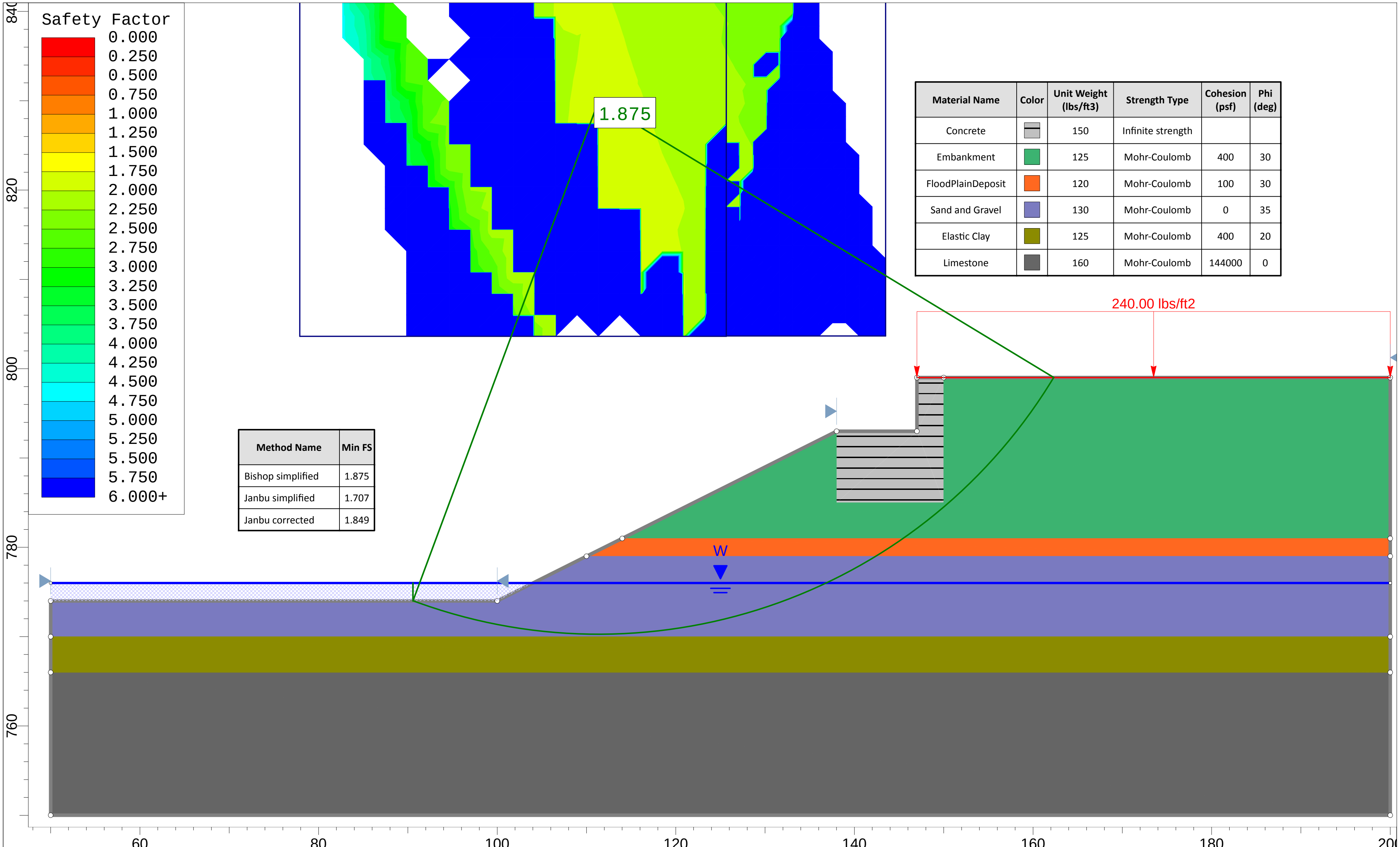


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Concrete		150	Infinite strength		
Embankment		125	Mohr-Coulomb	2000	0
FloodPlainDeposit		120	Mohr-Coulomb	1000	0
Sand and Gravel		130	Mohr-Coulomb	0	35
Elastic Clay		125	Mohr-Coulomb	2000	0
Limestone		160	Mohr-Coulomb	144000	0

Method Name	Min FS
Bishop simplified	2.635
Janbu simplified	2.687
Janbu corrected	2.861

Project	FRA-71-0153 Bridge
Analysis Description	Global Stability-Total Stress Analysis
Drawn By	ZM
Scale	1:180
Company	NEAS,Inc.
Date	3/21/2017, 10:21:16 AM
File Name	FRA-71-0153-Total-Circular.slim

SLIDEINTERPRET 7.022



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Concrete		150	Infinite strength		
Embankment		125	Mohr-Coulomb	400	30
FloodPlainDeposit		120	Mohr-Coulomb	100	30
Sand and Gravel		130	Mohr-Coulomb	0	35
Elastic Clay		125	Mohr-Coulomb	400	20
Limestone		160	Mohr-Coulomb	144000	0

Method Name	Min FS
Bishop simplified	1.875
Janbu simplified	1.707
Janbu corrected	1.849

rocscience

SLIDEINTERPRET 7.022

Project	FRA-71-0153 Bridge
Analysis Description	Global Stability-Effective Stress Analysis
Drawn By	ZM
Scale	1:178
Company	NEAS,Inc.
Date	3/21/2017, 10:21:16 AM
File Name	FRA-71-0153-Effective-Circular.slim

APPENDIX B-2

DRILLED SHAFT AXIAL RESISTANCE

FRA-70-0.00 Big Darby
Calculation of Rock Socket Resistance
Ref: AASHTO LRFD 7th Edition

Rock is dominantly limestone (Logs of Borings - Appendix A)
Unconfined Compressive Strength Results - 8 UCT tests (Appendix A)

Use R to determine cautious mean for analysis at 95% CI

LCL=5634 psi @ 95% CL, use 5600 psi as mean qus for side resistance
Use qub = 5600 psi for base resistance - overall mean may considered representative of rock below tip.

Use RQD LCL mean of 19%

```
SEsMacPro:~ stu$ cd /Volumes/Giganotosaurus/BP/BP_FRA-71-0.00/
SEsMacPro:BP_FRA-71-0.00 stu$ cd BP_FRA-71-0153BigDarby/
SEsMacPro:BP_FRA-71-0153BigDarby stu$ cd analysis/
SEsMacPro:analysis stu$ R

R version 3.0.2 (2013-09-25) -- "Frisbee Sailing"
Copyright (C) 2013 The R Foundation for Statistical Computing
Platform: x86_64-apple-darwin10.8.0 (64-bit)

> uct<-read.table("uct.csv",header=TRUE,sep=",")
> uct
  boring elevation depth elevation.1 UCS.psi γd.pcf mc.. mstrain.. Ex.106.psi RQD.. material
1      1       798  32.0       766.0    7558  160.2  0.5       1.1        1.7    28       ls
2      1       798  32.6       765.4    3697  154.7  1.3       0.5        1.1    28       ls
3      1       798  38.4       759.6    8396  165.7  0.4       0.8        2.6    14       ls
4      1       798  47.7       750.3    5803  147.7  9.2       0.5        1.9    66      sis
5      1       798  36.4       761.6    8960  151.7  4.1       1.0        1.8    28      dol
6      2       798  34.4       763.6    8491  159.3  1.8       0.7        2.4    14       ls
7      2       798  38.7       759.3   12175  162.7  0.9       0.7        2.8    42       ls
8      2       798  42.3       755.7   16868  165.4  0.7       0.9        3.4    62       ls
> library(pastecs)
> stat.desc(uct)
                 boring elevation       depth  elevation.1      UCS.psi
nbr.val       8.0000000         8   8.0000000 8.000000e+00 8.000000e+00
nbr.null      0.0000000         0   0.0000000 0.000000e+00 0.000000e+00
nbr.na        0.0000000         0   0.0000000 0.000000e+00 0.000000e+00
min           1.0000000       798  32.0000000 7.503000e+02 3.697000e+03
max           2.0000000       798  47.7000000 7.660000e+02 1.686800e+04
range         1.0000000         0  15.7000000 1.570000e+01 1.317100e+04
sum          11.0000000      6384 302.5000000 6.081500e+03 7.194800e+04
median        1.0000000       798  37.4000000 7.606000e+02 8.443500e+03
mean          1.3750000       798  37.8125000 7.601875e+02 8.993500e+03
SE.mean       0.1829813         0   1.8594774 1.859477e+00 1.420722e+03
CI.mean.0.95  0.4326819         0   4.3969654 4.396965e+00 3.359473e+03
var           0.2678571         0  27.6612500 2.766125e+01 1.614760e+07
std.dev       0.5175492         0   5.2593964 5.259396e+00 4.018407e+03
coef.var      0.3763994         0   0.1390915 6.918551e-03 4.468124e-01
                    γd.pcf       mc..   strain.. E...x.106.psi       RQD..
nbr.val          8.0000000  8.000000 8.00000000     8.0000000   8.0000000
nbr.null         0.0000000  0.000000 0.00000000     0.0000000   0.0000000
nbr.na           0.0000000  0.000000 0.00000000     0.0000000   0.0000000
min            147.7000000  0.400000 0.50000000     1.1000000  14.0000000
max            165.7000000  9.200000 1.10000000     3.4000000  66.0000000
range           18.0000000  8.800000 0.60000000     2.3000000  52.0000000
sum           1267.4000000 18.900000 6.20000000    17.7000000 282.0000000
median         159.7500000  1.100000 0.75000000     2.1500000  28.0000000
mean           158.4250000  2.362500 0.77500000     2.2125000  35.2500000
SE.mean          2.3064157  1.064347 0.07734431     0.2573474   7.0299106
CI.mean.0.95     5.4538066  2.516781 0.18289024     0.6085299  16.6230971
var             42.5564286  9.062679 0.04785714     0.5298214 395.3571429
std.dev          6.5235288  3.010428 0.21876275     0.7278883  19.8835898
coef.var         0.0411774  1.274255 0.28227452     0.3289891   0.5640735


> t.test (uct$UCS.psi, y=NULL, conf.level=0.95)

        One Sample t-test

data:  uct$UCS.psi
t = 6.3302, df = 7, p-value = 0.0003926
alternative hypothesis: true mean is not equal to 0
95 percent confidence interval:
  5634.027 12352.973
sample estimates:
mean of x
   8993.5
```

```
> t.test(uct$RQD.., y=NULL, conf.level=0.95)

        One Sample t-test

data:  uct$RQD..
t = 5.0143, df = 7, p-value = 0.00154
alternative hypothesis: true mean is not equal to 0
95 percent confidence interval:
 18.6269 51.8731
sample estimates:
mean of x
    35.25
```

Compute side resistance - 10.8.3.5.4b					
side shear resistance	qs	0.65*αE*pa*(qu/pa)^0.5		83.92	psi
where					
rock jointing reduction factor for av. RQD = 19%	αE	0.45			
(T-10.8.3.5.4b-1 and T-10.4.6.5-1)					
atmospheric pressure	pa	14.7	psi		
LCL mean compressive strength	qus	5600	psi		
Check concrete strength limit-assume	f'c	4000	psi		
		C*pa*(f'c/pa)^0.5		242.49	psi
if concrete shear strength > qs			OK		
Compute nominal side resistance per unit length of shafts		π*d*qus*12/1000	kips/ft		
diameter (inches)	d				
	36			113.90	klf
	42			132.88	klf
	48			151.86	klf
	54			170.85	klf
	60			189.83	klf
	66			208.81	klf
	72			227.79	klf
Compute base resistance (10.8.3.5.4c-1)	qb	2.5*qub		14000.00	psi
where	qub	5600	psi		
Compute nominal base resistance for shafts		(π*(d^2)/4)*qb/1000)			
diameter (inches)	d				
	36			14250.26	kips
	48			25333.80	kips

Compute resistance for given shaft lengths (ft) L
Assume upper 2 ft of rock is ignored
Assume base resistance requires socket >= 1.5*d long
use resistance factor = 0.55 (side) (10.5.5.2.4-1) φs 0.55
use resistance factor = 0.50 (base) (10.5.5.2.4-1) φb 0.5

	L		side resistance (kips)		base resistance (kips)		total resistance (kips)		
			nominal	factored	nominal	factored	nominal	factored	sock length (ft)
for 36" dia socket	4	e78*(c99-2)	227.79	125.29			227.79	125.29	6
	5		341.69	187.93	14250.26	7125.13	14591.96	7313.06	7
	6		455.59	250.57	14250.26	7125.13	14705.85	7375.71	8
	8		683.38	375.86	14250.26	7125.13	14933.65	7500.99	10
	10		911.18	501.15	14250.26	7125.13	15161.44	7626.28	12
	12		1138.97	626.43	14250.26	7125.13	15389.23	7751.57	14
	14		1366.76	751.72	14250.26	7125.13	15617.03	7876.85	16
	16		1594.56	877.01	14250.26	7125.13	15844.82	8002.14	18
	18		1822.35	1002.29	14250.26	7125.13	16072.62	8127.43	20
	20		2050.15	1127.58	14250.26	7125.13	16300.41	8252.71	22

Compute resistance for given shaft lengths (ft) L
Assume upper 2 ft of rock is ignored
Assume base resistance requires socket >= 1.5*d long
use resistance factor = 0.55 (side) (10.5.5.2.4-1) φs 0.55
use resistance factor = 0.50 (base) (10.5.5.2.4-1) φb 0.5

	L		side resistance (kips)		base resistance (kips)		total resistance (kips)		
			nominal	factored	nominal	factored	nominal	factored	sock length (ft)
for 48" dia socket	4	e80*(C114-2)	303.73	167.05			303.73	167.05	6
	5		455.59	250.57			455.59	250.57	7
	6		607.45	334.10	25333.80	12666.90	25941.25	13001.00	8
	8		911.18	501.15	25333.80	12666.90	26244.98	13168.05	10
	10		1214.90	668.20	25333.80	12666.90	26548.70	13335.10	12
	12		1518.63	835.24	25333.80	12666.90	26852.43	13502.15	14
	14		1822.35	1002.29	25333.80	12666.90	27156.15	13669.19	16
	16		2126.08	1169.34	25333.80	12666.90	27459.88	13836.24	18
	18		2429.80	1336.39	25333.80	12666.90	27763.61	14003.29	20
	20		2733.53	1503.44	25333.80	12666.90	28067.33	14170.34	22

APPENDIX B-3

UPLIFT RESISTANCE

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\B153.DVN
Project Name: FRA-71-0153 Project Date: 03/22/2017
Project Client: Mead _Hunt, Inc
Computed By: ZM
Project Manager:

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 17.83 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	22.00 ft
	- Driving/Restrike	22.00 ft
	- Ultimate:	22.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	18.00 ft	33.00%	125.00 pcf	2000.00 psf	T-80 Same
2	Cohesive	2.00 ft	17.00%	120.00 pcf	1000.00 psf	T-80 Same
3	Cohesionless	9.00 ft	0.00%	130.00 pcf	34.0/34.0	Nordlund
4	Cohesive	4.00 ft	50.00%	125.00 pcf	2000.00 psf	T-80 Same

RESTRIKE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	1375.16 psf	0.00 Kips
17.99 ft	Cohesive	N/A	N/A	1375.16 psf	0.87 Kips
18.01 ft	Cohesive	N/A	N/A	923.00 psf	0.96 Kips
19.99 ft	Cohesive	N/A	N/A	923.00 psf	8.22 Kips
20.01 ft	Cohesionless	2490.65 psf	25.61	N/A	8.30 Kips
21.99 ft	Cohesionless	2619.35 psf	25.61	N/A	17.02 Kips
22.01 ft	Cohesionless	2750.34 psf	25.61	N/A	17.11 Kips
28.99 ft	Cohesionless	2986.26 psf	25.61	N/A	52.16 Kips
29.01 ft	Cohesive	N/A	N/A	1375.16 psf	52.27 Kips
32.99 ft	Cohesive	N/A	N/A	1375.16 psf	74.01 Kips

RESTRIKE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
17.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
19.99 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
20.01 ft	Cohesionless	2491.30 psf	55.60	7.91 Kips	7.91 Kips
21.99 ft	Cohesionless	2748.70 psf	55.60	7.91 Kips	7.91 Kips
22.01 ft	Cohesionless	2750.68 psf	55.60	7.91 Kips	7.91 Kips
28.99 ft	Cohesionless	3222.52 psf	55.60	7.91 Kips	7.91 Kips
29.01 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
32.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips

RESTRIKE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.82 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.83 ft	0.00 Kips	1.94 Kips	1.94 Kips
17.99 ft	0.87 Kips	1.94 Kips	2.81 Kips
18.01 ft	0.96 Kips	0.97 Kips	1.93 Kips
19.99 ft	8.22 Kips	0.97 Kips	9.19 Kips
20.01 ft	8.30 Kips	7.91 Kips	16.21 Kips
21.99 ft	17.02 Kips	7.91 Kips	24.94 Kips
22.01 ft	17.11 Kips	7.91 Kips	25.03 Kips
28.99 ft	52.16 Kips	7.91 Kips	60.08 Kips
29.01 ft	52.27 Kips	1.94 Kips	54.21 Kips
32.99 ft	74.01 Kips	1.94 Kips	75.94 Kips

DRIVING - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	1375.16 psf	0.00 Kips
17.99 ft	Cohesive	N/A	N/A	1375.16 psf	0.59 Kips
18.01 ft	Cohesive	N/A	N/A	923.00 psf	0.66 Kips
19.99 ft	Cohesive	N/A	N/A	923.00 psf	6.68 Kips
20.01 ft	Cohesionless	2490.65 psf	25.61	N/A	6.76 Kips
21.99 ft	Cohesionless	2619.35 psf	25.61	N/A	15.48 Kips
22.01 ft	Cohesionless	2750.34 psf	25.61	N/A	15.58 Kips
28.99 ft	Cohesionless	2986.26 psf	25.61	N/A	50.63 Kips
29.01 ft	Cohesive	N/A	N/A	1375.16 psf	50.68 Kips
32.99 ft	Cohesive	N/A	N/A	1375.16 psf	61.55 Kips

DRIVING - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
17.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
19.99 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
20.01 ft	Cohesionless	2491.30 psf	55.60	7.91 Kips	7.91 Kips
21.99 ft	Cohesionless	2748.70 psf	55.60	7.91 Kips	7.91 Kips
22.01 ft	Cohesionless	2750.68 psf	55.60	7.91 Kips	7.91 Kips
28.99 ft	Cohesionless	3222.52 psf	55.60	7.91 Kips	7.91 Kips
29.01 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
32.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips

DRIVING - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.82 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.83 ft	0.00 Kips	1.94 Kips	1.94 Kips
17.99 ft	0.59 Kips	1.94 Kips	2.52 Kips
18.01 ft	0.66 Kips	0.97 Kips	1.63 Kips
19.99 ft	6.68 Kips	0.97 Kips	7.65 Kips
20.01 ft	6.76 Kips	7.91 Kips	14.68 Kips
21.99 ft	15.48 Kips	7.91 Kips	23.40 Kips
22.01 ft	15.58 Kips	7.91 Kips	23.49 Kips
28.99 ft	50.63 Kips	7.91 Kips	58.54 Kips
29.01 ft	50.68 Kips	1.94 Kips	52.62 Kips
32.99 ft	61.55 Kips	1.94 Kips	63.49 Kips

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	1375.16 psf	0.00 Kips
17.99 ft	Cohesive	N/A	N/A	1375.16 psf	0.87 Kips
18.01 ft	Cohesive	N/A	N/A	923.00 psf	0.96 Kips
19.99 ft	Cohesive	N/A	N/A	923.00 psf	8.22 Kips
20.01 ft	Cohesionless	2490.65 psf	25.61	N/A	8.30 Kips
21.99 ft	Cohesionless	2619.35 psf	25.61	N/A	17.02 Kips
22.01 ft	Cohesionless	2750.34 psf	25.61	N/A	17.11 Kips
28.99 ft	Cohesionless	2986.26 psf	25.61	N/A	52.16 Kips
29.01 ft	Cohesive	N/A	N/A	1375.16 psf	52.27 Kips
32.99 ft	Cohesive	N/A	N/A	1375.16 psf	74.01 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.82 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
17.83 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
17.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
19.99 ft	Cohesive	N/A	N/A	N/A	0.97 Kips
20.01 ft	Cohesionless	2491.30 psf	55.60	7.91 Kips	7.91 Kips
21.99 ft	Cohesionless	2748.70 psf	55.60	7.91 Kips	7.91 Kips
22.01 ft	Cohesionless	2750.68 psf	55.60	7.91 Kips	7.91 Kips
28.99 ft	Cohesionless	3222.52 psf	55.60	7.91 Kips	7.91 Kips
29.01 ft	Cohesive	N/A	N/A	N/A	1.94 Kips
32.99 ft	Cohesive	N/A	N/A	N/A	1.94 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.82 ft	0.00 Kips	0.00 Kips	0.00 Kips
17.83 ft	0.00 Kips	1.94 Kips	1.94 Kips
17.99 ft	0.87 Kips	1.94 Kips	2.81 Kips
18.01 ft	0.96 Kips	0.97 Kips	1.93 Kips
19.99 ft	8.22 Kips	0.97 Kips	9.19 Kips
20.01 ft	8.30 Kips	7.91 Kips	16.21 Kips
21.99 ft	17.02 Kips	7.91 Kips	24.94 Kips
22.01 ft	17.11 Kips	7.91 Kips	25.03 Kips
28.99 ft	52.16 Kips	7.91 Kips	60.08 Kips
29.01 ft	52.27 Kips	1.94 Kips	54.21 Kips
32.99 ft	74.01 Kips	1.94 Kips	75.94 Kips

Objective: To determine the uplift resistance and the minimum tip elevation of deep foundation .
Method: In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.7.3.10 and Sect. 10.7.3.8.6].
Assumptions:

- **Static Analysis Methods (LRFD Sect. 10.7.3.8.6) are used to determine the nominal uplift resistance due to side resistance.**
- **Deep foundation is in predominantly granular soil conditions.**

Givens:

Soil Parameters:

$L_{uplift} := 16.7\ kip$ — Factored uplift load for single pile

Soil Parameters:

$\gamma := 120\ \frac{lbf}{ft^3}$ — Average Unit weight of overburden soils

$Su_{Ave} := 1\ ksf$ — Average undrained shear strength along the sides of pile group

Ground Surface Elevation:

$G := 799\ ft$ — Ground Surface

Water Condition:

$E_w := 770\ ft$ — Elevation of Groundwater

$\gamma_w := 62.4\ \frac{lbf}{ft^3}$ — Unit weight of Water

Pile Condition:

$E := 781.17\ ft$ — Pile Cap Elevation

$L_{min} := 15\ ft$ — Estimated Minimum Geotechnical Design Pile Length

$b := 12\ in$ — Width/Breadth of HP 12X53 pile

$W_{HPpile} := 53\ \frac{lb}{ft}$ — Weight of HP 12X53 pile

$A_{pile} := 15.5\ in^2$ — Area of HP 12X53 pile

Pile Group Condition:

$n := 60$ — Number of piles in pile group

$Z_{cap} := 3.5\ ft$ — Height of pile group cap

$X := 12\ ft$ — Overall width of pile group

$Y := 152\ ft$ — Overall length of pile group

$Z := L_{min} = 15\ ft$ — Depth of the soil block below pile cap

$\gamma_{con} := 150\ \frac{lbf}{ft^3}$ — Unit weight of concrete

Uplift Resistance of Single Piles

$R_s := 74.01\ kip$ | $R_s = 74\ kip$ | Nominal uplift resistance for single pile from Driven results

$R_R := 22.90\ kip$ | $R_R = 22.9\ kip$ | Factored uplift resistance for single pile from Driven results

$SP := \text{if}\left(R_R > L_{uplift}, 1, 0\right)$ | $SP = 1$ | Uplift resistance check for single pile, "1" indicates OK, "0" indicates No

Uplift Resistance of Pile Group

Case 1: Sum of individual pile uplift resistance

$R_{ug1} := n \cdot R_s$ | $R_{ug1} = 4440.6\ kip$ | Sum of factored uplift resistance for pile group

Case 2: Uplift resistance of the block of pile group:

$A := b \cdot b$ | $A = 1\ ft^2$ | Area of HP 12X53 Pile

$W_{pile} := \gamma_{con} \cdot \left(A - A_{pile}\right) \cdot Z + W_{HPpile} \cdot Z \cdot g$ | $W_{pile} = 2802.8\ lbf$ | Weight of single pile

$W_{cap} := \gamma_{con} \cdot (X + 1\ ft) \cdot (Y + 1\ ft) \cdot Z_{cap}$ | $W_{cap} = 1044225\ lbf$ | Weight of pile group cap

$d_{PinW} := E_w - (E - Z)$ | $d_{PinW} = 3.8\ ft$ | Height of Pile group below groudwater level

$$W_{soil} := \gamma \cdot \left(X \cdot Y \cdot Z - A \cdot Z + X \cdot \frac{Z}{4} \cdot Z\right) - \gamma_w \cdot \left(X \cdot Y \cdot d_{PinW} - A \cdot d_{PinW} + X \cdot \frac{d_{PinW}}{4} \cdot d_{PinW}\right)$$

$W_{soil} = 2923971.6\ lbf$ | Weight of soil block

$Wg := W_{pile} \cdot n + W_{cap} + W_{soil}$ | $Wg = 4136.4\ kip$ | Weight of pile group, pile cap and soil block

$R_{ug2} := (2\ X \cdot Z + 2\ Y \cdot Z)\ Su_{Ave} + Wg$ | $R_{ug2} = 9056.4\ kip$ | Nomimal group uplift resistance (LRFD [Eq.10.7.3.11-2])

$\varphi_{ug} := 0.5$ | Resistance factor for group uplift resistance (LRFD [Table 10.5.5.2.3-1)

$R_{RG} := \varphi_{ug} \cdot min\left(R_{ug1}, R_{ug2}\right)$ | $R_{RG} = 2220.3\ kip$ | Factor group uplift resistance

$SG := \text{if}\left(R_{RG} > n \cdot L_{uplift}, 1, 0\right)$ | $SG = 1$ | Uplift resistance check for pile group, "1" indicates OK, "0" indicates No

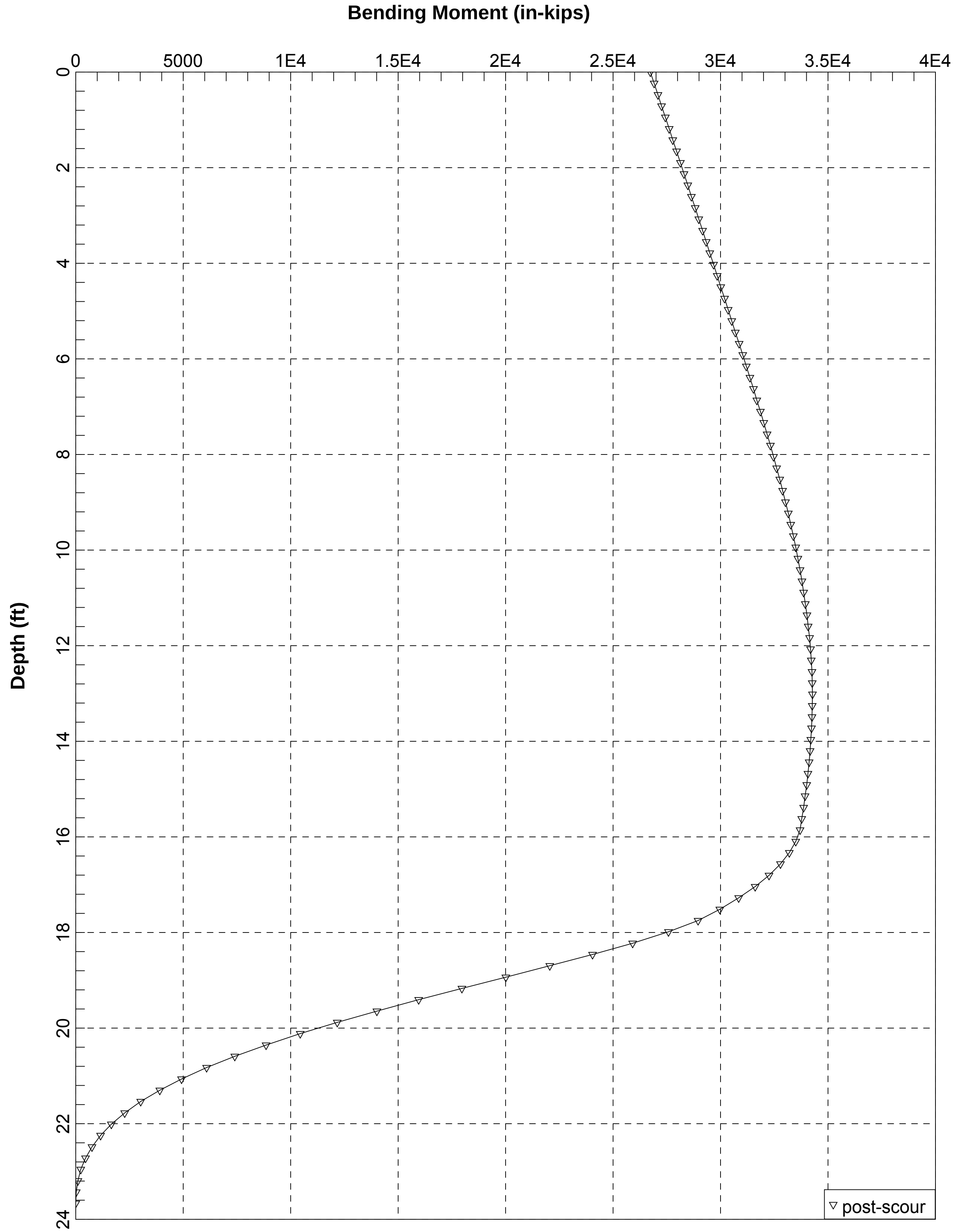
Uplift Check Results

$SP := \text{if}\left(R_R > L_{uplift}, 1, 0\right)$ $SP = 1$ Uplift resistance check for single pile, "1" indicates OK, "0" indicates No

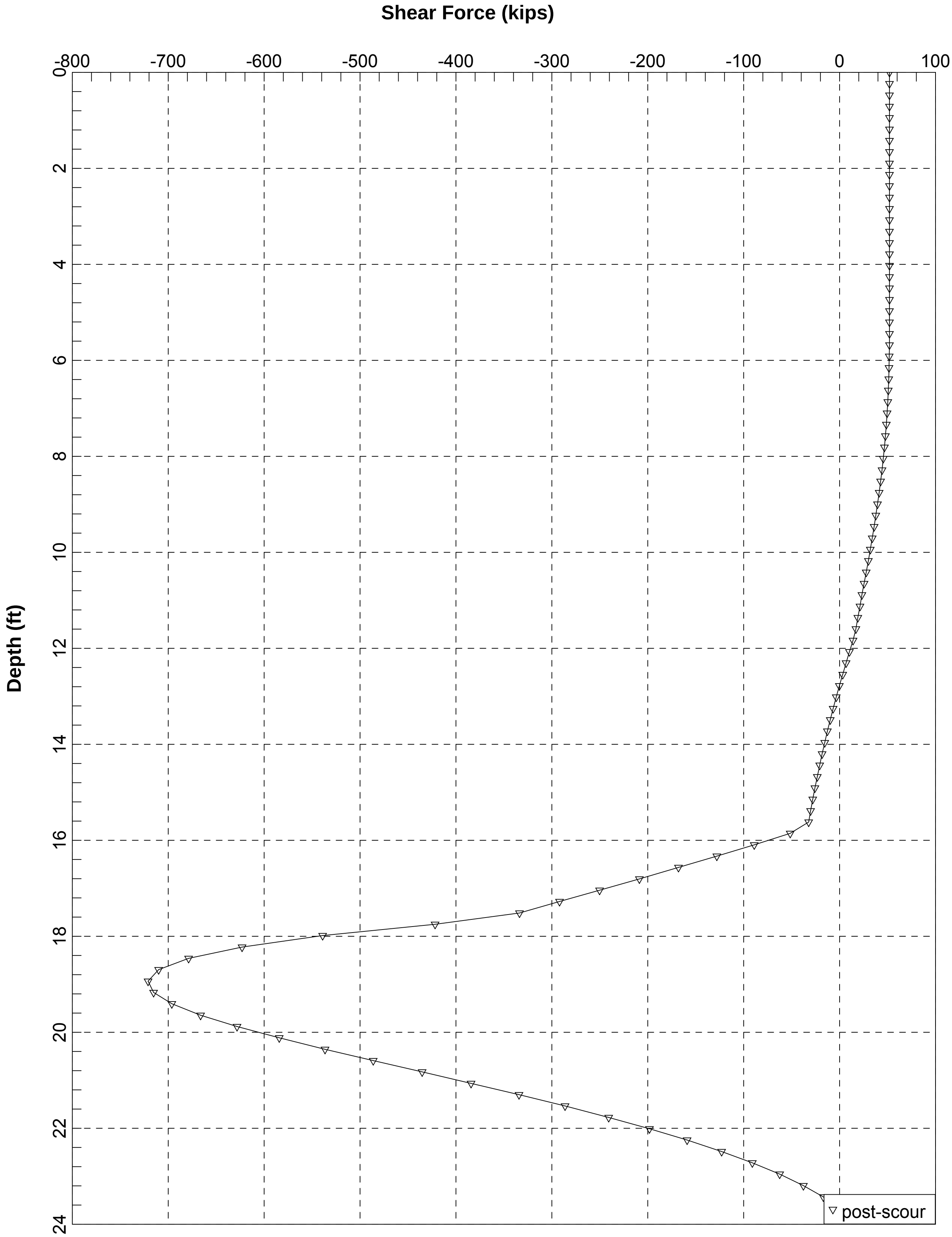
$SG := \text{if}\left(R_{RG} > n \cdot L_{uplift}, 1, 0\right)$ $SG = 1$ Uplift resistance check for pile group, "1" indicates OK, "0" indicates No

APPENDIX B-4

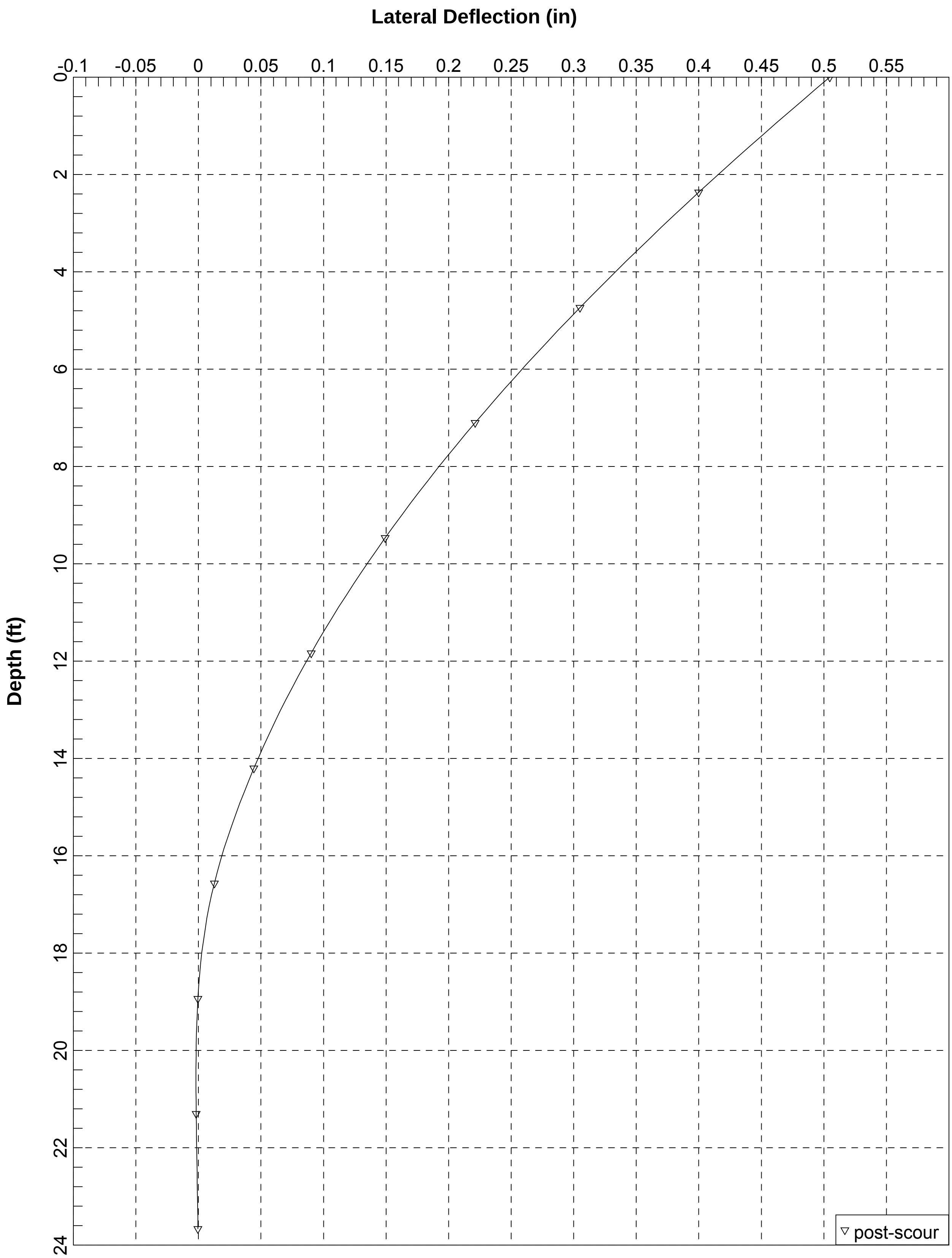
LATERAL LOAD ANALYSIS



Big Darby Pier 1L Strength - Max Vertical Load and Max Moment X axis



Big Darby Pier 1L Strength - Maximum Vertical Load and Moment X-axis



Big Darby Pier 1L Service Maximum Moment - X axis

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

               LPILE Plus for Windows, Version 4.0 (4.0.10)

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

                    (c) Copyright ENSOFT, Inc., 1985-2003
                             All Rights Reserved

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


This program is licensed to:

Portable
Flash Memory

Path to file locations:      F:\FRA\93496\geotechnical\engapps\lpile\BDP2L\BDP148\
Name of input data file:     RBigDarbyShaft_P1L.lpd
Name of output file:         RBigDarbyShaft_P1L.lpo
Name of plot output file:    RBigDarbyShaft_P1L.lpp
Name of runtime file:        RBigDarbyShaft_P1L.lpr


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

               Date:  February 18, 2015     Time:  21:54:22


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

FRA-71-0153 BigDarby P1L drilled shaft. Strength Limit State - post scour


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

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments            =         100
- Maximum number of iterations allowed =         100
```

- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 284.00 in
Depth of ground surface below top of pile = 66.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 6 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	54.000	417446.0000	2290.0000	2900000.000
2	66.0000	54.000	417446.0000	2290.0000	2900000.000
3	66.0000	54.000	417446.0000	2290.0000	2900000.000
4	188.0000	54.000	417446.0000	2290.0000	2900000.000
5	188.0000	48.000	260610.0000	1809.0000	2900000.000
6	284.0000	48.000	260610.0000	1809.0000	2900000.000

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 66.000 in
Distance from top of pile to bottom of layer = 140.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 140.000 in
Distance from top of pile to bottom of layer = 188.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 3 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer = 188.000 in
Distance from top of pile to bottom of layer = 212.000 in
Initial modulus of rock at top of layer = 1.4000E+06 lbs/in**2
Initial modulus of rock at bottom of layer = 1.4000E+06 lbs/in**2

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 212.000 in
Distance from top of pile to bottom of layer = 325.000 in

(Depth of lowest layer extends 41.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

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

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	66.00	.03600
2	140.00	.03600
3	140.00	.03600
4	188.00	.03600
5	188.00	.05100
6	212.00	.05100
7	212.00	.05600
8	325.00	.05600

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	66.000	.00000	36.00	------	------
2	140.000	.00000	36.00	------	------
3	140.000	6.90000	.00	.00700	.0
4	188.000	6.90000	.00	.00700	.0
5	188.000	500.00000	.00	.00050	15.0
6	212.000	500.00000	.00	.00050	15.0
7	212.000	5600.00000	.00	------	------
8	325.000	5600.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 52000.000 lbs
Bending moment at pile head = 26736000.000 in-lbs
Axial load at pile head = 1422000.000 lbs

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

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 52000.000 lbs
Specified bending moment at pile head = 26736000.000 in-lbs
Specified axial load at pile head = 1422000.000 lbs

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

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.878476	2.674E+07	52000.0000	-.006802	2350.2191	0.0000
2.840	.859247	2.691E+07	51999.9999	-.006739	2361.5395	0.0000
5.680	.840197	2.709E+07	52000.0000	-.006676	2372.8433	0.0000
8.520	.821328	2.726E+07	52000.0000	-.006612	2384.1306	0.0000
11.360	.802640	2.743E+07	52000.0000	-.006548	2395.4012	0.0000
14.200	.784135	2.761E+07	52000.0000	-.006483	2406.6550	0.0000
17.040	.765814	2.778E+07	52000.0000	-.006418	2417.8918	0.0000
19.880	.747678	2.796E+07	52000.0000	-.006353	2429.1116	0.0000
22.720	.729729	2.813E+07	52000.0000	-.006287	2440.3143	0.0000
25.560	.711966	2.830E+07	51999.9999	-.006221	2451.4998	0.0000
28.400	.694393	2.847E+07	52000.0000	-.006155	2462.6679	0.0000
31.240	.677009	2.865E+07	52000.0000	-.006088	2473.8186	0.0000
34.080	.659815	2.882E+07	52000.0000	-.006020	2484.9517	0.0000
36.920	.642814	2.899E+07	52000.0000	-.005952	2496.0671	0.0000
39.760	.626006	2.916E+07	52000.0000	-.005884	2507.1648	0.0000
42.600	.609393	2.933E+07	52000.0000	-.005815	2518.2446	0.0000
45.440	.592974	2.950E+07	52000.0000	-.005746	2529.3065	0.0000
48.280	.576753	2.968E+07	52000.0000	-.005677	2540.3503	0.0000
51.120	.560729	2.985E+07	52000.0000	-.005607	2551.3758	0.0000
53.960	.544904	3.002E+07	52000.0000	-.005537	2562.3831	0.0000
56.800	.529279	3.019E+07	52000.0000	-.005466	2573.3720	0.0000
59.640	.513855	3.036E+07	52000.0000	-.005395	2584.3424	0.0000
62.480	.498633	3.053E+07	52000.0000	-.005324	2595.2942	0.0000
65.320	.483615	3.069E+07	52000.0000	-.005252	2606.2273	0.0000
68.160	.468801	3.086E+07	51960.3732	-.005180	2617.1416	-27.9062
71.000	.454192	3.103E+07	51827.2225	-.005107	2628.0224	-65.8619
73.840	.439791	3.120E+07	51584.5609	-.005034	2638.8498	-105.0266
76.680	.425597	3.136E+07	51229.3030	-.004961	2649.6034	-145.1550
79.520	.411613	3.153E+07	50759.0575	-.004887	2660.2620	-186.0037
82.360	.397838	3.169E+07	50172.1223	-.004813	2670.8042	-227.3309
85.200	.384275	3.185E+07	49467.4785	-.004739	2681.2084	-268.8971
88.040	.370923	3.201E+07	48644.7845	-.004664	2691.4528	-310.4649

90.880	.357785	3.217E+07	47704.3692	-.004588	2701.5157	-351.7994
93.720	.344862	3.232E+07	46646.7664	-.004513	2711.3753	-392.9913
96.560	.332153	3.247E+07	45471.4684	-.004437	2721.0101	-434.6833
99.400	.319661	3.261E+07	44178.4794	-.004360	2730.3982	-475.8724
102.240	.307387	3.275E+07	42769.4895	-.004284	2739.5181	-516.3741
105.080	.295330	3.289E+07	41246.7071	-.004207	2748.3486	-556.0079
107.920	.283493	3.302E+07	39612.8477	-.004129	2756.8688	-594.5973
110.760	.271876	3.315E+07	37871.1220	-.004052	2765.0586	-631.9701
113.600	.260479	3.327E+07	36025.2239	-.003974	2772.8984	-667.9581
116.440	.249304	3.339E+07	34079.3185	-.003896	2780.3694	-702.3978
119.280	.238352	3.350E+07	32038.0287	-.003817	2787.4535	-735.1302
122.120	.227623	3.360E+07	29905.7824	-.003738	2794.1336	-766.4517
124.960	.217118	3.370E+07	27726.7545	-.003660	2800.3932	-768.0750
127.800	.206837	3.379E+07	25547.0181	-.003580	2806.2315	-766.9506
130.640	.196781	3.387E+07	23374.2106	-.003501	2811.6490	-763.1955
133.480	.186951	3.395E+07	21215.6365	-.003421	2816.6476	-756.9271
136.320	.177347	3.402E+07	19078.2668	-.003342	2821.2306	-748.2629
139.160	.167970	3.408E+07	16968.7383	-.003262	2825.4023	-737.3205
142.000	.158820	3.414E+07	14035.4731	-.003182	2829.1685	-1328.3592
144.840	.149897	3.419E+07	10316.6849	-.003102	2832.2208	-1290.5057
147.680	.141202	3.422E+07	6705.5906	-.003021	2834.5790	-1252.5184
150.520	.132735	3.425E+07	3202.5865	-.002941	2836.2627	-1214.3859
153.360	.124497	3.427E+07	-191.8963	-.002861	2837.2920	-1176.0949
156.200	.116486	3.427E+07	-3477.3856	-.002780	2837.6867	-1137.6299
159.040	.108704	3.427E+07	-6653.3614	-.002700	2837.4670	-1098.9728
161.880	.101150	3.426E+07	-9719.2484	-.002620	2836.6529	-1060.1025
164.720	.093825	3.424E+07	-12674.4059	-.002539	2835.2648	-1020.9942
167.560	.086728	3.421E+07	-15518.1163	-.002459	2833.3232	-981.6187
170.400	.079858	3.417E+07	-18249.5713	-.002379	2830.8484	-941.9412
173.240	.073216	3.412E+07	-20867.8546	-.002299	2827.8614	-901.9203
176.080	.066802	3.407E+07	-23371.9203	-.002219	2824.3829	-861.5062
178.920	.060614	3.401E+07	-25760.5656	-.002139	2820.4341	-820.6384
181.760	.054653	3.394E+07	-28032.3966	-.002059	2816.0364	-779.2425
184.600	.048918	3.386E+07	-30185.7824	-.001980	2811.2114	-737.2263
187.440	.043409	3.378E+07	-32218.7957	-.001900	2805.9810	-694.4732
190.280	.038125	3.370E+07	-51567.4476	-.001797	3889.1644	-12931.3380
193.120	.033200	3.350E+07	-89046.1169	-.001671	3871.5198	-13462.0911
195.960	.028633	3.320E+07	-127911.8357	-.001546	3843.8291	-13908.1334
198.800	.024420	3.279E+07	-167916.5368	-.001422	3805.7614	-14264.1913
201.640	.020558	3.226E+07	-208795.6137	-.001300	3757.0528	-14523.8910
204.480	.017039	3.161E+07	-250264.1277	-.001179	3697.5111	-14679.2879
207.320	.013858	3.085E+07	-292011.2680	-.001062	3627.0218	-14720.1067
210.160	.011006	2.996E+07	-333691.8879	-9.479E-04	3545.5557	-14632.4425
213.000	.008474	2.896E+07	-421857.7718	-8.372E-04	3453.1791	-47456.2081
215.840	.006251	2.758E+07	-538956.3224	-7.309E-04	3325.5128	-35007.5599
218.680	.004323	2.591E+07	-623040.8547	-6.304E-04	3171.8053	-24206.8995
221.520	.002670	2.404E+07	-678650.0096	-5.366E-04	3000.0813	-14954.4772
224.360	.001275	2.206E+07	-710022.5369	-4.500E-04	2817.2159	-7138.8518
227.200	1.15E-04	2.001E+07	-721070.4257	-3.709E-04	2629.0171	-641.3516
230.040	-8.32E-04	1.796E+07	-715363.7232	-2.996E-04	2440.3139	4660.1562
232.880	-.001587	1.595E+07	-696125.1542	-2.359E-04	2255.0471	8888.1318
235.720	-.002172	1.401E+07	-666232.8237	-1.796E-04	2076.3599	12162.8051
238.560	-.002607	1.217E+07	-628229.4466	-1.304E-04	1906.6873	14600.1365
241.400	-.002913	1.044E+07	-584336.7178	-8.790E-05	1747.8423	16310.2359
244.240	-.003106	8.850E+06	-536473.5976	-5.165E-05	1601.0976	17396.1868
247.080	-.003206	7.397E+06	-486277.4396	-2.113E-05	1467.2619	17953.2202
249.920	-.003226	6.088E+06	-435127.0375	4.210E-06	1346.7511	18068.1896
252.760	-.003182	4.925E+06	-384166.8004	2.490E-05	1239.6523	17819.3013
255.600	-.003085	3.906E+06	-334331.3908	4.150E-05	1145.7824	17276.0576
258.440	-.002946	3.026E+06	-286370.2751	5.452E-05	1064.7392	16499.3760
261.280	-.002775	2.279E+06	-240871.7339	6.449E-05	995.9471	15541.8501
264.120	-.002580	1.657E+06	-198285.9723	7.188E-05	938.6960	14448.1228
266.960	-.002367	1.152E+06	-158947.0441	7.716E-05	892.1742	13255.3478

269.800	-.002142	753888.5145	-123093.3742	8.074E-05	855.4965	11993.7155
272.640	-.001908	452339.7823	-90886.7181	8.301E-05	827.7264	10687.0283
275.480	-.001670	236981.4808	-62429.4411	8.431E-05	807.8937	9353.3076
278.320	-.001430	97059.6211	-37780.0419	8.493E-05	795.0080	8005.4242
281.160	-.001188	21704.8377	-16966.8686	8.516E-05	788.0685	6651.7401
284.000	-9.46E-04	0.0000	0.0000	8.520E-05	786.0697	5296.7589

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =      .87847629 in
Computed slope at pile head       =     -.00680218
Maximum bending moment            =   34272718.909 lbs-in
Maximum shear force               =    -721070.426 lbs
Depth of maximum bending moment   =        156.200 in
Depth of maximum shear force      =        227.200 in
Number of iterations              =              9
Number of zero deflection points  =              1
```

Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 52000.000	M= 2.67E+07	1.422E+06	.8785	3.427E+07	-721070.4257

The analysis ended normally.

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

              LPILE Plus for Windows, Version 4.0 (4.0.10)

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

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


This program is licensed to:

Portable
Flash Memory

Path to file locations:      F:\FRA\93496\geotechnical\engapps\lpile\BDP2L\BDP148\
Name of input data file:     RBigDarbyShaft_P1L_service.lpd
Name of output file:         RBigDarbyShaft_P1L_service.lpo
Name of plot output file:    RBigDarbyShaft_P1L_service.lpp
Name of runtime file:        RBigDarbyShaft_P1L_service.lpr


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

               Date:  February 18, 2015     Time:  22:18:17


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

FRA-71-0153 BigDarby P1L drilled shaft. Strength Limit State - post scour


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

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments           =          100
- Maximum number of iterations allowed =          100
```

- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 284.00 in
Depth of ground surface below top of pile = 66.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 6 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	54.000	417446.0000	2290.0000	2900000.000
2	66.0000	54.000	417446.0000	2290.0000	2900000.000
3	66.0000	54.000	417446.0000	2290.0000	2900000.000
4	188.0000	54.000	417446.0000	2290.0000	2900000.000
5	188.0000	48.000	260610.0000	1809.0000	2900000.000
6	284.0000	48.000	260610.0000	1809.0000	2900000.000

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 66.000 in
Distance from top of pile to bottom of layer = 140.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 140.000 in
Distance from top of pile to bottom of layer = 188.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 3 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer = 188.000 in
Distance from top of pile to bottom of layer = 212.000 in
Initial modulus of rock at top of layer = 1.4000E+06 lbs/in**2
Initial modulus of rock at bottom of layer = 1.4000E+06 lbs/in**2

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 212.000 in
Distance from top of pile to bottom of layer = 325.000 in

(Depth of lowest layer extends 41.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

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

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	66.00	.03600
2	140.00	.03600
3	140.00	.03600
4	188.00	.03600
5	188.00	.05100
6	212.00	.05100
7	212.00	.05600
8	325.00	.05600

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	66.000	.00000	36.00	------	------
2	140.000	.00000	36.00	------	------
3	140.000	6.90000	.00	.00700	.0
4	188.000	6.90000	.00	.00700	.0
5	188.000	500.00000	.00	.00050	15.0
6	212.000	500.00000	.00	.00050	15.0
7	212.000	5600.00000	.00	------	------
8	325.000	5600.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

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

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 49000.000 lbs
Bending moment at pile head = 13464000.000 in-lbs
Axial load at pile head = 1080000.000 lbs

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

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 49000.000 lbs
Specified bending moment at pile head = 13464000.000 in-lbs
Specified axial load at pile head = 1080000.000 lbs

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

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.504778	1.346E+07	49000.0000	-.003855	1342.4541	0.0000
2.840	.493876	1.361E+07	49000.0000	-.003823	1352.2164	0.0000
5.680	.483065	1.377E+07	49000.0000	-.003791	1361.9723	0.0000
8.520	.472346	1.392E+07	49000.0000	-.003758	1371.7218	0.0000
11.360	.461719	1.407E+07	49000.0000	-.003725	1381.4649	0.0000
14.200	.451186	1.422E+07	49000.0000	-.003692	1391.2014	0.0000
17.040	.440748	1.437E+07	49000.0000	-.003659	1400.9313	0.0000
19.880	.430405	1.452E+07	49000.0000	-.003625	1410.6545	0.0000
22.720	.420159	1.467E+07	49000.0000	-.003591	1420.3709	0.0000
25.560	.410011	1.482E+07	49000.0000	-.003556	1430.0805	0.0000
28.400	.399961	1.497E+07	49000.0000	-.003521	1439.7833	0.0000
31.240	.390012	1.512E+07	49000.0000	-.003486	1449.4790	0.0000
34.080	.380163	1.527E+07	49000.0000	-.003450	1459.1677	0.0000
36.920	.370416	1.542E+07	49000.0000	-.003414	1468.8493	0.0000
39.760	.360771	1.557E+07	49000.0000	-.003378	1478.5238	0.0000
42.600	.351230	1.572E+07	49000.0000	-.003341	1488.1910	0.0000
45.440	.341794	1.587E+07	49000.0000	-.003304	1497.8508	0.0000
48.280	.332464	1.602E+07	49000.0000	-.003267	1507.5033	0.0000
51.120	.323240	1.616E+07	49000.0000	-.003229	1517.1484	0.0000
53.960	.314124	1.631E+07	49000.0000	-.003191	1526.7859	0.0000
56.800	.305117	1.646E+07	49000.0000	-.003152	1536.4158	0.0000
59.640	.296219	1.661E+07	49000.0000	-.003113	1546.0381	0.0000
62.480	.287432	1.676E+07	49000.0000	-.003074	1555.6526	0.0000
65.320	.278757	1.691E+07	49000.0000	-.003035	1565.2593	0.0000
68.160	.270195	1.706E+07	48965.6579	-.002995	1574.8582	-24.1846
71.000	.261746	1.721E+07	48850.3762	-.002955	1584.4365	-56.9997
73.840	.253411	1.735E+07	48640.5510	-.002914	1593.9770	-90.7646
76.680	.245193	1.750E+07	48333.7990	-.002873	1603.4622	-125.2580
79.520	.237090	1.764E+07	47928.3627	-.002832	1612.8738	-160.2606
82.360	.229106	1.779E+07	47423.1039	-.002791	1622.1936	-195.5555
85.200	.221240	1.793E+07	46817.4969	-.002749	1631.4032	-230.9284
88.040	.213493	1.807E+07	46111.6208	-.002706	1640.4839	-266.1675

90.880	.205867	1.821E+07	45306.1516	-.002664	1649.4174	-301.0643
93.720	.198362	1.835E+07	44410.1594	-.002621	1658.1853	-329.9162
96.560	.190980	1.848E+07	43444.4224	-.002578	1666.7726	-350.1803
99.400	.183720	1.861E+07	42424.3575	-.002534	1675.1686	-368.1752
102.240	.176585	1.873E+07	41356.3175	-.002491	1683.3638	-383.9657
105.080	.169574	1.886E+07	40246.4701	-.002446	1691.3501	-397.6170
107.920	.162689	1.898E+07	39100.7969	-.002402	1699.1201	-409.1951
110.760	.155930	1.910E+07	37925.0917	-.002357	1706.6679	-418.7663
113.600	.149299	1.921E+07	36724.9589	-.002312	1713.9883	-426.3976
116.440	.142795	1.932E+07	35505.8125	-.002267	1721.0773	-432.1562
119.280	.136421	1.942E+07	34272.8747	-.002222	1727.9319	-436.1099
122.120	.130175	1.953E+07	33031.1747	-.002176	1734.5499	-438.3268
124.960	.124060	1.962E+07	31785.5474	-.002130	1740.9302	-438.8755
127.800	.118076	1.972E+07	30540.6327	-.002084	1747.0724	-437.8250
130.640	.112223	1.981E+07	29300.8739	-.002038	1752.9770	-435.2446
133.480	.106502	1.990E+07	28070.5172	-.001991	1758.6453	-431.2038
136.320	.100913	1.998E+07	26853.6105	-.001944	1764.0794	-425.7727
139.160	.095458	2.006E+07	25654.0030	-.001897	1769.2822	-419.0214
142.000	.090136	2.014E+07	23637.9668	-.001850	1774.2569	-1000.7224
144.840	.084949	2.021E+07	20837.4120	-.001803	1778.7003	-971.4992
147.680	.079896	2.027E+07	18120.0111	-.001755	1782.6275	-942.1634
150.520	.074978	2.032E+07	15486.0956	-.001708	1786.0537	-912.7067
153.360	.070195	2.037E+07	12936.0229	-.001660	1788.9943	-883.1191
156.200	.065549	2.041E+07	10470.1813	-.001612	1791.4647	-853.3891
159.040	.061038	2.044E+07	8088.9948	-.001564	1793.4805	-823.5028
161.880	.056663	2.046E+07	5792.9299	-.001516	1795.0571	-793.4442
164.720	.052425	2.048E+07	3582.5035	-.001468	1796.2103	-763.1941
167.560	.048323	2.049E+07	1458.2919	-.001420	1796.9558	-732.7295
170.400	.044358	2.050E+07	-579.0571	-.001372	1797.3096	-702.0233
173.240	.040529	2.050E+07	-2528.8105	-.001324	1797.2875	-671.0425
176.080	.036837	2.049E+07	-4390.1318	-.001276	1796.9059	-639.7472
178.920	.033281	2.048E+07	-6162.0586	-.001228	1796.1810	-608.0885
181.760	.029862	2.046E+07	-7843.4726	-.001180	1795.1293	-576.0059
184.600	.026579	2.044E+07	-9433.0616	-.001132	1793.7676	-543.4230
187.440	.023432	2.042E+07	-10929.2665	-.001084	1792.1130	-510.2425
190.280	.020422	2.039E+07	-27363.1250	-.001022	2474.4238	-11062.8973
193.120	.017629	2.027E+07	-59390.7334	-9.454E-04	2463.4364	-11491.7564
195.960	.015052	2.005E+07	-92525.7676	-8.697E-04	2443.8918	-11842.7747
198.800	.012689	1.975E+07	-126539.7621	-7.949E-04	2415.5293	-12110.7426
201.640	.010537	1.934E+07	-161187.6825	-7.214E-04	2378.1503	-12289.2013
204.480	.008591	1.884E+07	-196203.4873	-6.497E-04	2331.6227	-12369.8161
207.320	.006847	1.823E+07	-231293.2731	-5.801E-04	2275.8872	-12341.3007
210.160	.005296	1.753E+07	-266124.0918	-5.129E-04	2210.9654	-12187.4449
213.000	.003933	1.672E+07	-314708.4006	-4.485E-04	2136.9726	-22026.8570
215.840	.002749	1.574E+07	-367844.7087	-3.875E-04	2046.6010	-15393.0782
218.680	.001732	1.464E+07	-403476.9046	-3.305E-04	1944.7791	-9700.0175
221.520	8.72E-04	1.345E+07	-424182.7970	-2.777E-04	1835.7368	-4881.5969
224.360	1.55E-04	1.223E+07	-432345.8793	-2.294E-04	1723.0543	-867.0526
227.200	-4.32E-04	1.100E+07	-430145.3214	-1.858E-04	1609.7147	2416.7413
230.040	-9.01E-04	9.785E+06	-419552.0099	-1.468E-04	1498.1586	5043.3372
232.880	-.001265	8.614E+06	-402329.5861	-1.122E-04	1390.3382	7085.1303
235.720	-.001538	7.501E+06	-380039.5317	-8.192E-05	1287.7713	8612.0911
238.560	-.001730	6.456E+06	-354049.4528	-5.569E-05	1191.5932	9690.7813
241.400	-.001854	5.490E+06	-325543.8083	-3.325E-05	1102.6064	10383.6162
244.240	-.001919	4.607E+06	-295536.4244	-1.428E-05	1021.3264	10748.3443
247.080	-.001935	3.812E+06	-264884.2234	1.542E-06	948.0251	10837.7128
249.920	-.001911	3.103E+06	-234301.6781	1.453E-05	882.7698	10699.2910
252.760	-.001853	2.481E+06	-204375.5790	2.502E-05	825.4583	10375.4268
255.600	-.001768	1.942E+06	-175579.7688	3.333E-05	775.8507	9903.3128
258.440	-.001663	1.483E+06	-148289.5627	3.977E-05	733.5971	9315.1422
261.280	-.001543	1.099E+06	-122795.6240	4.462E-05	698.2609	8638.3358
264.120	-.001410	785358.7228	-99317.1156	4.816E-05	669.3399	7895.8251
266.960	-.001269	534988.1475	-78013.9863	5.064E-05	646.2829	7106.3787

269.800	-.001122	341928.6138	-58998.2861	5.229E-05	628.5037	6284.9595
272.640	-9.72E-04	199557.1084	-42344.4338	5.331E-05	615.3925	5443.1055
275.480	-8.20E-04	101085.2149	-28098.3820	5.387E-05	606.3240	4589.3253
278.320	-6.66E-04	39627.8185	-16285.6445	5.414E-05	600.6643	3729.5039
281.160	-5.12E-04	8250.6521	-6918.1628	5.423E-05	597.7747	2867.3142
284.000	-3.58E-04	0.0000	0.0000	5.424E-05	597.0149	2004.6314

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection            =      .50477842 in
Computed slope at pile head     =    -.00385450
Maximum bending moment          =   20496503.315 lbs-in
Maximum shear force             =    -432345.879 lbs
Depth of maximum bending moment =        170.400 in
Depth of maximum shear force    =        224.360 in
Number of iterations            =             10
Number of zero deflection points =             1
```

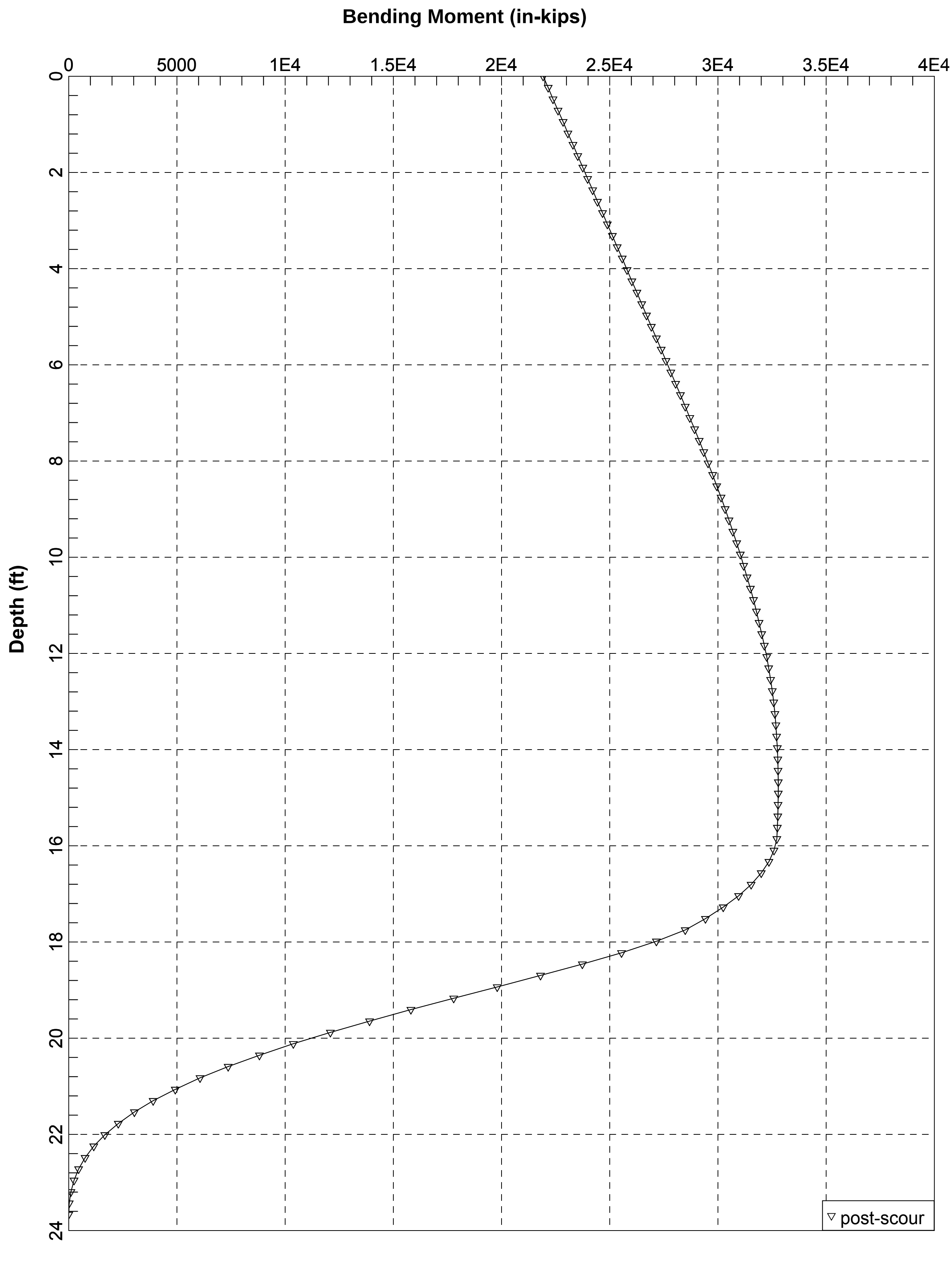
Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

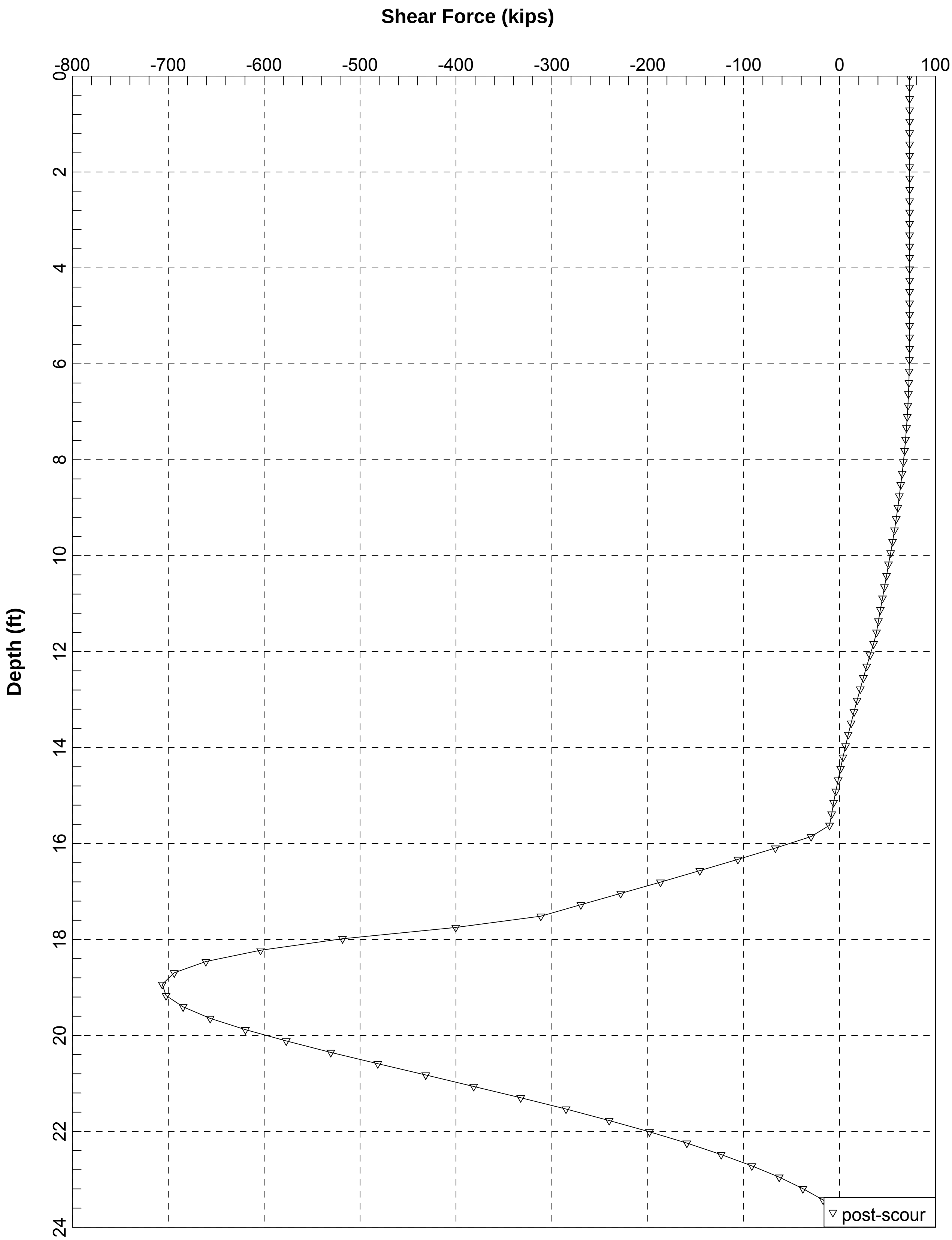
y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 49000.000	M= 1.35E+07	1.080E+06	.5048	2.050E+07	-432345.8793

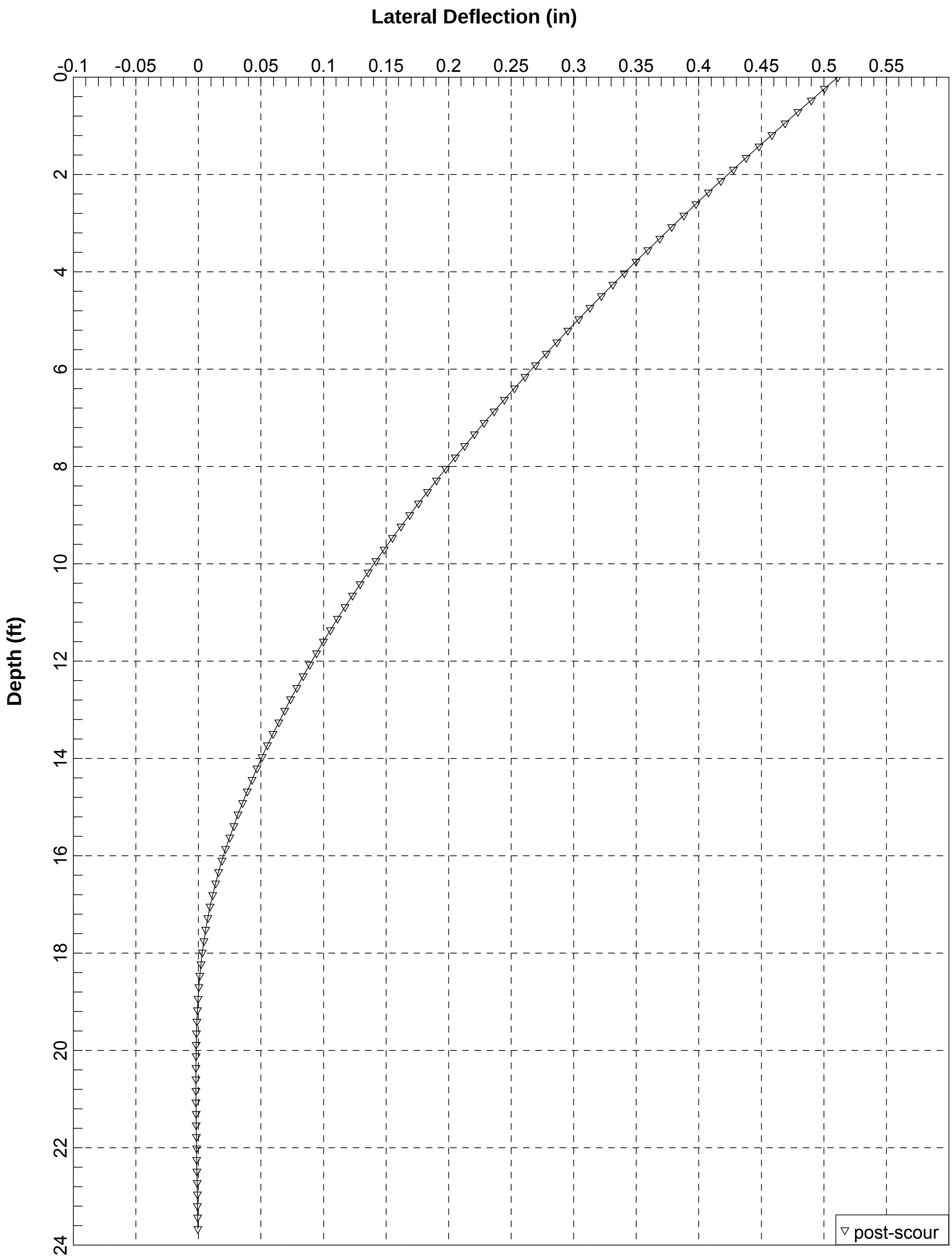
The analysis ended normally.



Big Darby Pier 1L Strength - Max Longitudinal Load



Big Darby Pier 1L Strength - Maximum Longitudinal_Load (Fz)



Big Darby Pier 1L Case3 service Maximum Longitudinal Load (Fz)

```
================================================================================
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                LPILE Plus for Windows, Version 4.0 (4.0.10)

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

                  (c) Copyright ENSOFT, Inc., 1985-2003
                           All Rights Reserved

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This program is licensed to:

Portable
Flash Memory

Path to file locations:      F:\FRA\93496\geotechnical\engapps\lpile
\BDP2L\BDP148\
Name of input data file:     RBigDarbyShaft_P1L_case2.lpd
Name of output file:         RBigDarbyShaft_P1L_case2.lpo
Name of plot output file:    RBigDarbyShaft_P1L_case2.lpp
Name of runtime file:        RBigDarbyShaft_P1L_case2.lpr


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

              Date:  February 20, 2015     Time:  13:21:53


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

FRA-71-0153 BigDarby P1L drilled shaft. Strength Limit State -post
scour - Case2

Fy=1241 kips; Fr=73 kips; Mr=1827 kip-ft


--------------------------------------------------------------------------------
```

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 284.00 in
Depth of ground surface below top of pile = 66.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 6 points

Point	Depth X	Pile Diameter	Moment of Inertia	Pile Area	Modulus of Elasticity

```
            in           in          in**4       Sq.in      lbs/Sq.in
-----  ---------   -----------   ----------   ----------   -----------
  1       0.0000       54.000  417446.0000    2290.0000   2900000.000
  2      66.0000       54.000  417446.0000    2290.0000   2900000.000
  3      66.0000       54.000  417446.0000    2290.0000   2900000.000
  4     188.0000       54.000  417446.0000    2290.0000   2900000.000
  5     188.0000       48.000  260610.0000    1809.0000   2900000.000
  6     284.0000       48.000  260610.0000    1809.0000   2900000.000


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

The soil profile is modelled using  4 layers

Layer  1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =       66.000 in
Distance from top of pile to bottom of layer =      140.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer    =      140.000 in
Distance from top of pile to bottom of layer =      188.000 in
p-y subgrade modulus k for top of soil layer =      500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      500.000 lbs/in**3

Layer  3 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer    =      188.000 in
Distance from top of pile to bottom of layer =      212.000 in
Initial modulus of rock at top of layer      =   1.4000E+06 lbs/in**2
Initial modulus of rock at bottom of layer   =   1.4000E+06 lbs/in**2

Layer  4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer    =      212.000 in
Distance from top of pile to bottom of layer =      325.000 in


(Depth of lowest layer extends   41.00 in below pile tip)


----------------------------------------------------------------------
--------
                   Effective Unit Weight of Soil vs. Depth
----------------------------------------------------------------------
--------
```

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

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	66.00	.03600
2	140.00	.03600
3	140.00	.03600
4	188.00	.03600
5	188.00	.05100
6	212.00	.05100
7	212.00	.05600
8	325.00	.05600

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	66.000	.00000	36.00	------	------
2	140.000	.00000	36.00	------	------
3	140.000	6.90000	.00	.00700	.0
4	188.000	6.90000	.00	.00700	.0
5	188.000	500.00000	.00	.00050	15.0
6	212.000	500.00000	.00	.00050	15.0
7	212.000	5600.00000	.00	------	------
8	325.000	5600.00000	.00	------	------

Notes:

```
(1)  Cohesion = uniaxial compressive strength for rock materials.
(2)  Values of E50 are reported for clay strata.
(3)  Default values will be generated for E50 when input values are 0.
(4)  RQD and k_rm are reported only for weak rock strata.



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

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



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

Number of loads specified =  1

Load Case Number  1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head    =       73000.000 lbs
Bending moment at pile head =    21924000.000 in-lbs
Axial load at pile head     =     1241000.000 lbs

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



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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       73000.000 lbs
```

Specified bending moment at pile head = 21924000.000 in-lbs
Specified axial load at pile head = 1241000.000 lbs

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

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.838322	2.192E+07	73000.0000	-.006312	1959.9443	0.0000
2.840	.820468	2.215E+07	72999.9999	-.006261	1974.7867	0.0000
5.680	.802762	2.238E+07	73000.0000	-.006208	1989.6171	0.0000
8.520	.785205	2.261E+07	73000.0000	-.006156	2004.4356	0.0000
11.360	.767798	2.284E+07	73000.0000	-.006102	2019.2421	0.0000
14.200	.750544	2.307E+07	73000.0000	-.006048	2034.0363	0.0000
17.040	.733443	2.330E+07	73000.0000	-.005994	2048.8181	0.0000
19.880	.716498	2.353E+07	73000.0000	-.005939	2063.5875	0.0000
22.720	.699709	2.375E+07	73000.0000	-.005884	2078.3443	0.0000
25.560	.683079	2.398E+07	73000.0000	-.005828	2093.0885	0.0000
28.400	.666608	2.421E+07	73000.0000	-.005771	2107.8198	0.0000
31.240	.650299	2.444E+07	73000.0000	-.005714	2122.5381	0.0000
34.080	.634153	2.467E+07	73000.0000	-.005656	2137.2434	0.0000
36.920	.618171	2.489E+07	73000.0000	-.005598	2151.9355	0.0000
39.760	.602354	2.512E+07	73000.0000	-.005540	2166.6142	0.0000
42.600	.586705	2.535E+07	73000.0000	-.005480	2181.2796	0.0000
45.440	.571225	2.557E+07	73000.0000	-.005421	2195.9314	0.0000

48.280	.555916	2.580E+07	73000.0000	-.005360	2210.5695	0.0000
51.120	.540778	2.603E+07	73000.0000	-.005300	2225.1938	0.0000
53.960	.525813	2.625E+07	73000.0000	-.005238	2239.8042	0.0000
56.800	.511024	2.648E+07	73000.0000	-.005177	2254.4006	0.0000
59.640	.496411	2.670E+07	73000.0000	-.005114	2268.9828	0.0000
62.480	.481976	2.693E+07	73000.0000	-.005051	2283.5507	0.0000
65.320	.467720	2.715E+07	73000.0000	-.004988	2298.1042	0.0000
68.160	.453645	2.738E+07	72960.7099	-.004924	2312.6432	-27.6691
71.000	.439752	2.760E+07	72828.6847	-.004859	2327.1531	-65.3064
73.840	.426044	2.782E+07	72588.0601	-.004794	2341.6142	-104.1475
76.680	.412521	2.805E+07	72235.7626	-.004729	2356.0061	-143.9493
79.520	.399184	2.827E+07	71769.4064	-.004663	2370.3079	-184.4705
82.360	.386036	2.849E+07	71187.2889	-.004596	2384.4983	-225.4714
85.200	.373078	2.871E+07	70488.3851	-.004529	2398.5559	-266.7144
88.040	.360311	2.892E+07	69672.3415	-.004461	2412.4590	-307.9642
90.880	.347737	2.913E+07	68739.4699	-.004393	2426.1860	-348.9876
93.720	.335357	2.934E+07	67690.2770	-.004325	2439.7153	-389.8806
96.560	.323172	2.955E+07	66524.2067	-.004256	2453.0256	-431.2956
99.400	.311184	2.975E+07	65241.2124	-.004186	2466.0950	-472.2215
102.240	.299394	2.995E+07	63842.9431	-.004116	2478.9022	-512.4751
105.080	.287804	3.014E+07	62331.5631	-.004046	2491.4261	-551.8770
107.920	.276415	3.033E+07	60709.7408	-.003975	2503.6459	-590.2515
110.760	.265228	3.051E+07	58980.6371	-.003903	2515.5415	-627.4272
113.600	.254244	3.069E+07	57147.8943	-.003832	2527.0936	-663.2368
116.440	.243465	3.087E+07	55215.6235	-.003759	2538.2832	-697.5173

119.280	.232891	3.103E+07	53188.3922	−.003687	2549.0925	−730.1103
122.120	.222524	3.119E+07	51087.6551	−.003614	2559.5043	−749.2820
124.960	.212365	3.135E+07	48956.8842	−.003540	2569.5085	−751.2609
127.800	.202414	3.150E+07	46824.3098	−.003467	2579.1040	−750.5520
130.640	.192674	3.164E+07	44697.4083	−.003393	2588.2912	−747.2659
133.480	.183144	3.177E+07	42583.3411	−.003318	2597.0716	−741.5138
136.320	.173826	3.190E+07	40488.9534	−.003244	2605.4481	−733.4071
139.160	.164721	3.203E+07	38420.7731	−.003169	2613.4250	−723.0579
142.000	.155829	3.214E+07	35525.6033	−.003093	2621.0076	−1315.7941
144.840	.147151	3.225E+07	31841.5177	−.003018	2627.8866	−1278.6323
147.680	.138688	3.235E+07	28263.1861	−.002942	2634.0813	−1241.3196
150.520	.130440	3.243E+07	24791.0528	−.002866	2639.6111	−1203.8448
153.360	.122409	3.251E+07	21425.5961	−.002790	2644.4956	−1166.1951
156.200	.114594	3.257E+07	18167.3344	−.002713	2648.7543	−1128.3554
159.040	.106996	3.263E+07	15016.8320	−.002637	2652.4070	−1090.3083
161.880	.099616	3.268E+07	11974.7073	−.002560	2655.4734	−1052.0331
164.720	.092453	3.272E+07	9041.6422	−.002484	2657.9736	−1013.5058
167.560	.085508	3.275E+07	6218.3930	−.002407	2659.9275	−974.6979
170.400	.078782	3.277E+07	3505.8045	−.002330	2661.3554	−935.5757
173.240	.072274	3.278E+07	904.8264	−.002253	2662.2777	−896.0990
176.080	.065984	3.279E+07	−1583.4657	−.002176	2662.7151	−856.2194
178.920	.059913	3.279E+07	−3957.8441	−.002099	2662.6882	−815.8780
181.760	.054060	3.278E+07	−6216.8948	−.002022	2662.2182	−775.0028
184.600	.048425	3.277E+07	−8358.9747	−.001946	2661.3263	−733.5041
187.440	.043009	3.275E+07	−10382.1526	−.001869	2660.0343	−691.2690

190.280	.037811	3.272E+07	-29688.4652	-.001769	3699.4590	-12904.7258
193.120	.032962	3.259E+07	-67095.1218	-.001646	3687.4558	-13437.9901
195.960	.028461	3.235E+07	-105897.0481	-.001524	3665.4314	-13887.3101
198.800	.024306	3.200E+07	-145848.4683	-.001403	3633.0524	-14247.4929
201.640	.020491	3.153E+07	-186687.3544	-.001284	3590.0518	-14512.2860
204.480	.017014	3.095E+07	-228131.7826	-.001166	3536.2333	-14673.9311
207.320	.013867	3.025E+07	-269874.6400	-.001051	3471.4776	-14722.4473
210.160	.011042	2.942E+07	-311575.5827	-9.393E-04	3395.7496	-14644.4138
213.000	.008532	2.848E+07	-400213.2536	-8.305E-04	3309.1082	-47776.4812
215.840	.006325	2.716E+07	-518352.1186	-7.259E-04	3186.9450	-35419.9026
218.680	.004408	2.554E+07	-603702.9532	-6.269E-04	3038.4396	-24686.3189
221.520	.002764	2.373E+07	-660738.2119	-5.343E-04	2871.5667	-15479.3563
224.360	.001373	2.180E+07	-693639.7226	-4.488E-04	2693.1672	-7690.7217
227.200	2.15E-04	1.980E+07	-706271.1387	-3.706E-04	2509.0286	-1204.6417
230.040	-7.32E-04	1.779E+07	-702162.0278	-3.000E-04	2323.9712	4098.3817
232.880	-.001489	1.581E+07	-684501.7372	-2.369E-04	2141.9362	8338.4427
235.720	-.002077	1.390E+07	-656141.3314	-1.811E-04	1966.0755	11633.6740
238.560	-.002518	1.208E+07	-619602.0679	-1.323E-04	1798.8390	14098.2016
241.400	-.002829	1.038E+07	-577089.0339	-9.005E-05	1642.0596	15840.5547
244.240	-.003029	8.807E+06	-530508.7302	-5.400E-05	1497.0334	16962.4761
247.080	-.003135	7.369E+06	-481489.5381	-2.361E-05	1364.5959	17558.0816
249.920	-.003163	6.072E+06	-431404.1516	1.648E-06	1245.1910	17713.3174
252.760	-.003126	4.918E+06	-381393.1868	2.230E-05	1138.9357	17505.6719
255.600	-.003036	3.905E+06	-332389.3103	3.888E-05	1045.6772	17004.1003
258.440	-.002905	3.030E+06	-285141.3331	5.191E-05	965.0441	16269.1232

```
 261.280  -.002742   2.286E+06-240237.8217   6.189E-05    896.4916
15353.0679
 264.120  -.002554   1.665E+06-198129.8658   6.932E-05    839.3402
14300.4221
 266.960  -.002348   1.160E+06-159152.7165   7.462E-05    792.8088
13148.2746
 269.800  -.002130 760413.8233-123546.0792   7.823E-05    756.0421
11926.8221
 272.640  -.001904 457360.6928 -91472.8989   8.052E-05    728.1335
10659.9246
 275.480  -.001672 240280.1932 -63036.5208   8.183E-05    708.1422
9365.6937
 278.320  -.001439  98736.4509 -38296.1496   8.247E-05    695.1072
8057.1029
 281.160  -.001204  22176.7699 -17280.5556   8.269E-05    688.0567
6742.6112
 284.000 -9.69E-04      0.0000      0.0000   8.273E-05    686.0144
5426.7941
```

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =      .83832197 in
Computed slope at pile head       =     -.00631228
Maximum bending moment            =   32789512.264 lbs-in
Maximum shear force               =    -706271.139 lbs
Depth of maximum bending moment   =        176.080 in
Depth of maximum shear force      =        227.200 in
Number of iterations              =              9
Number of zero deflection points  =              1
```


Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs

S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

```
 BC    Boundary     Boundary       Axial    Pile Head    Maximum    
Maximum
Type  Condition    Condition       Load     Deflection    Moment    
Shear
          1            2            lbs         in        in-lbs    
lbs
---- ------------ ------------ ----------- ----------- -----------
-----------
  1  V= 73000.000 M=  2.19E+07   1.241E+06       .8383   3.279E
+07-706271.1387
```

The analysis ended normally.

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

                LPILE Plus for Windows, Version 4.0 (4.0.10)

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

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                             All Rights Reserved

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This program is licensed to:

Portable
Flash Memory

Path to file locations:      F:\FRA\93496\geotechnical\engapps\lpile
\BDP2L\BDP148\
Name of input data file:     RBigDarbyShaft_P1L_case3_service.lpd
Name of output file:         RBigDarbyShaft_P1L_case3_service.lpo
Name of plot output file:    RBigDarbyShaft_P1L_case3_service.lpp
Name of runtime file:        RBigDarbyShaft_P1L_case3_service.lpr


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

               Date:  February 20, 2015     Time:  13:50:54


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

FRA-71-0153 BigDarby P1L drilled shaft. Strength Limit State -post
scour - Case2


---------------------------------------------------------------------------
--------
                               Program Options
```

```
--------------------------------------------------------------------------------

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant
EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft
action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments            =          100
- Maximum number of iterations allowed =          100
- Deflection tolerance for convergence =   1.0000E-05 in
- Maximum allowable deflection         =   1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) =  1


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

Pile Length                                 =     284.00 in
Depth of ground surface below top of pile =      66.00 in
Slope angle of ground surface               =        .00 deg.

Structural properties of pile defined using  6 points

Point    Depth        Pile      Moment of       Pile      Modulus of
           X        Diameter     Inertia        Area      Elasticity
           in          in         in**4         Sq.in     lbs/Sq.in
-----  ---------   -----------  ----------   ----------   -----------
```

1	0.0000	54.000	417446.0000	2290.0000	2900000.000
2	66.0000	54.000	417446.0000	2290.0000	2900000.000
3	66.0000	54.000	417446.0000	2290.0000	2900000.000
4	188.0000	54.000	417446.0000	2290.0000	2900000.000
5	188.0000	48.000	260610.0000	1809.0000	2900000.000
6	284.0000	48.000	260610.0000	1809.0000	2900000.000

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

```
Layer  1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =       66.000 in
Distance from top of pile to bottom of layer =      140.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  2 is stiff clay with water-induced erosion
Distance from top of pile to top of layer    =      140.000 in
Distance from top of pile to bottom of layer =      188.000 in
p-y subgrade modulus k for top of soil layer =      500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      500.000 lbs/in**3

Layer  3 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer    =      188.000 in
Distance from top of pile to bottom of layer =      212.000 in
Initial modulus of rock at top of layer      =   1.4000E+06 lbs/in**2
Initial modulus of rock at bottom of layer   =   1.4000E+06 lbs/in**2

Layer  4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer    =      212.000 in
Distance from top of pile to bottom of layer =      325.000 in
```

(Depth of lowest layer extends 41.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

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

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	66.00	.03600
2	140.00	.03600
3	140.00	.03600
4	188.00	.03600
5	188.00	.05100
6	212.00	.05100
7	212.00	.05600
8	325.00	.05600

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	66.000	.00000	36.00	------	------
2	140.000	.00000	36.00	------	------
3	140.000	6.90000	.00	.00700	.0
4	188.000	6.90000	.00	.00700	.0
5	188.000	500.00000	.00	.00050	15.0
6	212.000	500.00000	.00	.00050	15.0
7	212.000	5600.00000	.00	------	------
8	325.000	5600.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.

(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

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

----

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head   =   64000.000 lbs
Bending moment at pile head =   11316000.000 in-lbs
Axial load at pile head   =   944000.000 lbs

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

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

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head   =   64000.000 lbs
Specified bending moment at pile head =   11316000.000 in-lbs
Specified axial load at pile head   =   944000.000 lbs

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

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.511230	1.132E+07	64000.0000	-.003785	1144.1349	0.0000
2.840	.500520	1.151E+07	63999.9999	-.003758	1156.5449	0.0000
5.680	.489886	1.170E+07	64000.0000	-.003731	1168.9502	0.0000
8.520	.479330	1.189E+07	64000.0000	-.003703	1181.3508	0.0000
11.360	.468854	1.208E+07	64000.0000	-.003675	1193.7465	0.0000
14.200	.458458	1.227E+07	64000.0000	-.003646	1206.1373	0.0000
17.040	.448143	1.247E+07	64000.0000	-.003617	1218.5231	0.0000
19.880	.437912	1.266E+07	64000.0000	-.003588	1230.9039	0.0000
22.720	.427765	1.285E+07	64000.0000	-.003558	1243.2795	0.0000
25.560	.417704	1.304E+07	64000.0000	-.003527	1255.6499	0.0000
28.400	.407730	1.323E+07	64000.0000	-.003497	1268.0149	0.0000
31.240	.397844	1.342E+07	64000.0000	-.003465	1280.3746	0.0000
34.080	.388047	1.361E+07	64000.0000	-.003434	1292.7288	0.0000
36.920	.378341	1.380E+07	64000.0000	-.003401	1305.0775	0.0000
39.760	.368727	1.400E+07	64000.0000	-.003369	1317.4206	0.0000
42.600	.359206	1.419E+07	64000.0000	-.003336	1329.7579	0.0000
45.440	.349779	1.438E+07	64000.0000	-.003302	1342.0895	0.0000
48.280	.340449	1.457E+07	64000.0000	-.003268	1354.4153	0.0000

51.120	.331215	1.476E+07	64000.0000	-.003234	1366.7351	0.0000
53.960	.322080	1.495E+07	64000.0000	-.003199	1379.0490	0.0000
56.800	.313045	1.514E+07	64000.0000	-.003164	1391.3567	0.0000
59.640	.304110	1.533E+07	64000.0000	-.003128	1403.6583	0.0000
62.480	.295277	1.552E+07	64000.0000	-.003092	1415.9536	0.0000
65.320	.286548	1.571E+07	64000.0000	-.003055	1428.2427	0.0000
68.160	.277923	1.590E+07	63965.4054	-.003018	1440.5253	-24.3624
71.000	.269405	1.609E+07	63849.2568	-.002981	1452.7888	-57.4324
73.840	.260993	1.628E+07	63637.8073	-.002943	1465.0158	-91.4757
76.680	.252690	1.647E+07	63328.6075	-.002904	1477.1884	-126.2707
79.520	.244497	1.665E+07	62919.8349	-.002865	1489.2885	-161.5973
82.360	.236415	1.684E+07	62410.2886	-.002826	1501.2974	-197.2381
85.200	.228445	1.702E+07	61799.3823	-.002786	1513.1967	-232.9777
88.040	.220588	1.720E+07	61087.1371	-.002746	1524.9674	-268.6034
90.880	.212846	1.738E+07	60274.1745	-.002706	1536.5911	-303.9055
93.720	.205219	1.756E+07	59361.1737	-.002665	1548.0491	-339.0528
96.560	.197710	1.774E+07	58364.9388	-.002623	1559.3231	-362.5211
99.400	.190319	1.791E+07	57308.5721	-.002582	1570.4008	-381.3990
102.240	.183047	1.807E+07	56201.8006	-.002539	1581.2723	-398.0175
105.080	.175896	1.824E+07	55050.9510	-.002497	1591.9287	-412.4400
107.920	.168866	1.840E+07	53862.1682	-.002454	1602.3626	-424.7310
110.760	.161958	1.856E+07	52641.4135	-.002410	1612.5674	-434.9554
113.600	.155175	1.871E+07	51394.4627	-.002367	1622.5377	-443.1790
116.440	.148516	1.886E+07	50126.9040	-.002323	1632.2693	-449.4679
119.280	.141982	1.901E+07	48844.1368	-.002278	1641.7587	-453.8893

122.120	.135576	1.915E+07	47551.3690	-.002233	1651.0036	-456.5106
124.960	.129297	1.929E+07	46253.6162	-.002188	1660.0025	-457.3998
127.800	.123146	1.943E+07	44955.6997	-.002143	1668.7550	-456.6258
130.640	.117125	1.956E+07	43662.2451	-.002097	1677.2614	-454.2577
133.480	.111234	1.969E+07	42377.6807	-.002051	1685.5228	-450.3652
136.320	.105475	1.981E+07	41106.2359	-.002005	1693.5413	-445.0185
139.160	.099847	1.993E+07	39851.9404	-.001958	1701.3196	-438.2882
142.000	.094352	2.005E+07	37775.6921	-.001911	1708.8611	-1023.8585
144.840	.088991	2.016E+07	34909.8440	-.001864	1715.8603	-994.3444
147.680	.083764	2.026E+07	32128.0000	-.001817	1722.3326	-964.7007
150.520	.078672	2.035E+07	29430.5402	-.001769	1728.2935	-934.9189
153.360	.073715	2.043E+07	26817.8706	-.001721	1733.7583	-904.9892
156.200	.068895	2.051E+07	24290.4279	-.001673	1738.7427	-874.9000
159.040	.064211	2.058E+07	21848.6845	-.001625	1743.2623	-844.6376
161.880	.059665	2.064E+07	19493.1552	-.001577	1747.3329	-814.1859
164.720	.055256	2.070E+07	17224.4048	-.001528	1750.9704	-783.5257
167.560	.050984	2.075E+07	15043.0574	-.001480	1754.1908	-752.6344
170.400	.046851	2.079E+07	12949.8081	-.001431	1757.0100	-721.4848
173.240	.042857	2.083E+07	10945.4367	-.001382	1759.4445	-690.0443
176.080	.039001	2.086E+07	9030.8255	-.001333	1761.5104	-658.2734
178.920	.035285	2.089E+07	7206.9818	-.001284	1763.2245	-626.1235
181.760	.031707	2.091E+07	5475.0668	-.001235	1764.6035	-593.5349
184.600	.028269	2.093E+07	3836.4334	-.001186	1765.6643	-560.4323
187.440	.024970	2.094E+07	2292.6771	-.001137	1766.4242	-526.7200
190.280	.021811	2.094E+07	-14425.1725	-.001073	2450.6541	-11246.4135

193.120	.018875	2.086E+07	-46994.4969	-9.945E-04	2442.9598	-11689.7304
195.960	.016162	2.068E+07	-80712.5419	-9.165E-04	2426.5633	-12055.3717
198.800	.013670	2.041E+07	-115351.4903	-8.393E-04	2401.1932	-12338.2539
201.640	.011395	2.003E+07	-150667.4339	-7.633E-04	2366.6396	-12532.1288
204.480	.009334	1.956E+07	-186396.2628	-6.889E-04	2322.7589	-12629.0184
207.320	.007482	1.898E+07	-222247.3986	-6.165E-04	2269.4795	-12618.2603
210.160	.005833	1.830E+07	-257893.7287	-5.465E-04	2206.8100	-12484.7891
213.000	.004378	1.752E+07	-310438.9574	-4.792E-04	2134.8502	-24518.8931
215.840	.003111	1.654E+07	-369994.2647	-4.152E-04	2044.6621	-17421.4642
218.680	.002020	1.542E+07	-410796.1862	-3.552E-04	1941.5184	-11312.2833
221.520	.001094	1.420E+07	-435556.2977	-2.995E-04	1829.9580	-6124.4149
224.360	3.19E-04	1.294E+07	-446788.3212	-2.485E-04	1713.8353	-1785.4609
227.200	-3.18E-04	1.167E+07	-446796.1650	-2.023E-04	1596.3745	1779.9371
230.040	-8.30E-04	1.041E+07	-437668.4855	-1.608E-04	1480.2248	4648.0061
232.880	-.001231	9.183E+06	-421278.6633	-1.240E-04	1367.5180	6894.1222
235.720	-.001534	8.015E+06	-399289.1834	-9.166E-05	1259.9234	8591.4270
238.560	-.001752	6.916E+06	-373159.5192	-6.361E-05	1158.7029	9809.7449
241.400	-.001895	5.896E+06	-344156.7171	-3.954E-05	1064.7624	10614.7636
244.240	-.001976	4.961E+06	-313367.9792	-1.914E-05	978.7008	11067.4462
247.080	-.002004	4.116E+06	-281714.6324	-2.088E-06	900.8552	11223.6431
249.920	-.001988	3.361E+06	-249966.9600	1.196E-05	831.3424	11133.8726
252.760	-.001936	2.696E+06	-218759.4522	2.334E-05	770.0964	10843.2456
255.600	-.001856	2.118E+06	-188606.1023	3.238E-05	716.9021	10391.5079
258.440	-.001752	1.624E+06	-159915.4451	3.942E-05	671.4243	9813.1803
261.280	-.001632	1.210E+06	-133005.0871	4.474E-05	633.2339	9137.7760

```
 264.120  -.001498 868640.7627-108115.5331   4.865E-05    601.8298
8390.0789
 266.960  -.001355 595293.3677 -85423.1553   5.140E-05    576.6568
7590.4688
 269.800  -.001206 383161.6604 -65052.1894   5.323E-05    557.1213
6755.2819
 272.640  -.001053 225511.4947 -47085.6708   5.438E-05    542.6030
5897.1959
 275.480 -8.97E-04 115423.4807 -31575.2542   5.502E-05    532.4648
5025.6326
 278.320 -7.41E-04  45869.0465 -18549.8727   5.532E-05    526.0594
4147.1712
 281.160 -5.83E-04   9763.5745  -8023.2142   5.543E-05    522.7344
3265.9685
 284.000 -4.26E-04      0.0000      0.0000   5.544E-05    521.8353
2384.1823
```

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      .51122990 in
Computed slope at pile head      =     -.00378453
Maximum bending moment           =   20944561.462 lbs-in
Maximum shear force              =    -446796.165 lbs
Depth of maximum bending moment  =        190.280 in
Depth of maximum shear force     =        227.200 in
Number of iterations             =             10
Number of zero deflection points =              1
```


Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

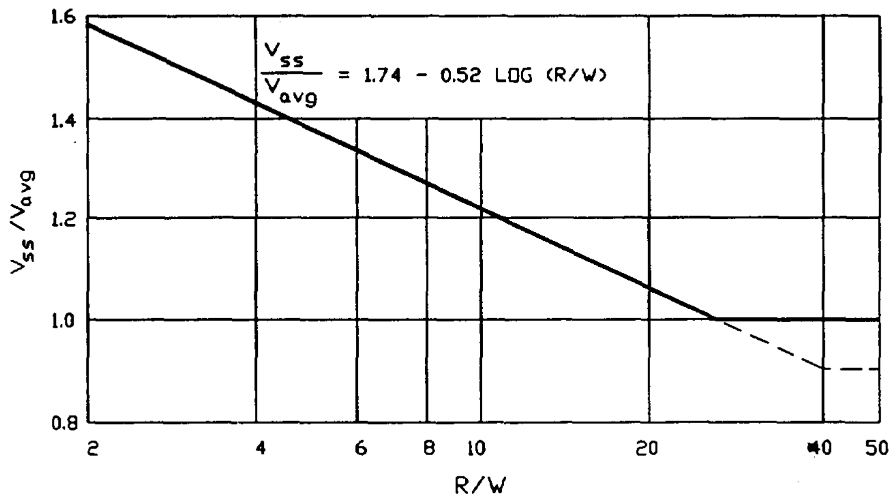
BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 64000.000	M= 1.13E+07	944000.0000	.5112	2.094E+07	-446796.1650

The analysis ended normally.

APPENDIX B-5

RIPRAP DESIGN

*



NOTE: V_{ss} IS DEPTH-AVERAGED VELOCITY AT 20 PERCENT OF SLOPE LENGTH UP FROM TOE

RIPRAP DESIGN VELOCITIES
NATURAL CHANNEL

Plate B-33
(Sheet 1 of 2)

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