STRUCTURAL ENGINEERING

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Final Report for:

Subsurface Exploration
SR 823 Bridge Over Morris Lane Blue Run Road (CR 54)
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

TranSystems Corporation
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Dublin, Ohio 43017

Ohio Department of Transportation
District 9

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DLZ Job No. 0121-3070.03

April 17, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

SR 823 BRIDGE OVER MORRIS LANE BLUE RUN ROAD (CR 54)

PROJECT SCI-823-10.13 PORTSMOUTH BYPASS (PID 79977)

SCIOTO COUNTY, OHIO

For:

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SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of evaluations for foundations of the proposed bridge at the above-referenced location of the project. The findings included in this report pertain to the structures at the proposed SR 823 bridge over Morris Lane Blue Run Road only. The findings of other structure evaluations for the Portsmouth bypass project will be submitted in separate documents.

The project consists in part of placing twin structures for the proposed SR 823 over Morris Lane Blue Run Road (CR 54) and a relocated stream. The two structures, as planned, are three-span structures using spill-through slopes at the abutments.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the structure foundations and the roadway approach embankments. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, considered the findings, and prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 GENERAL PROJECT INFORMATION

Based upon comments from ODOT's Office of Structural Engineering (OSE), it is understood that cast-in-place (CIP) pile foundations are recommended to support the abutments of the proposed structures. MSE walls were considered at the abutment locations during preliminary design phases. However, it is understood that spill-through slopes are now being considered at the abutments.

It is assumed that the maximum height of the embankment at stations 716+69 (Rear Abutment) and 719+96 (Forward Abutment) will be approximately 72.0 and 47.5 feet, respectively. These heights are based upon the maximum difference between the proposed grade of SR 823 and the existing grade, as indicated on the Structure Site Plan, presented in Appendix I.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concept are changed or differ from that assumed, DLZ should be informed of the changes so that recommendations and conclusions presented in this report may be revised as necessary.

3.0 FIELD EXPLORATION

The field exploration consisted of drilling a total of nine borings. Structure borings TR-4 through TR-6 were drilled for a previous design configuration. These borings were drilled on March 15 and 16, 2005. Borings B-18 through B-23 were drilled for the currently proposed structure, essentially consisting of proposed SR 823 passing over Morris Lane Blue Run Road (CR 54). These borings were drilled between September 6 and 15, 2006. The boring locations are presented on the Structure Site Plan, presented in Appendix I. Boring logs for all structural borings are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were determined by representatives of DLZ. The surveyed locations and ground surface elevations of the borings were determined by representatives from Lockwood, Lanier, Mathias & Noland, Inc. (2LMN).

4.0 FINDINGS

4.1 Geology of the Site

The area of this structure is characterized by gently sloping to steeply sloping topography. The project area is located in the Shawnee-Mississippian Plateau of the unglaciated portion of the Appalachian Plateau Physiographic Region. The Shawnee-Mississippian Plateau is characterized by Devonian aged to Pennsylvanian aged rocks and contains residual colluvial, glacial, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Soils at the rear abutment location are composed primarily of residual and colluvial soils. These soils are generally thin, covering moderate to steep slopes. At the forward abutment residual and lacustrine soils were encountered. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. Bedrock within the structure area is primarily sandstone of the Cuyahoga Formation of Mississippian age. Bedrock of the Mississippian Logan and the Pennsylvanian Breathitt Formation can be found on the slopes north and east of the structures roughly above elevations 800 and 1020, respectively. In the area of the structure, the bedrock was covered by a relatively thin soil overburden ranging in thickness between 5 and 32 feet.

4.2 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix II. Laboratory test results are presented on the boring logs and also in Appendix III.

4.2.1 Soil Conditions

Borings TR-4 and B-23 were drilled for the forward abutment. Borings B-18 and B-19 were drilled for the rear abutment, while borings TR-5, TR-6, and B-20 through B-23 were drilled for the piers.

All borings except B-18 and B-20 encountered surficial material consisting of 1 to 4 inches of topsoil. The topsoil, when encountered, was underlain by native soil deposits. All borings encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of stiff to hard silt and clay (A-6a), stiff to hard sandy silt (A-4a), stiff to hard silt (A-4b), medium stiff to hard silty clay (A-6b), and clay (A-7-6), while the granular soil deposits consisted mainly of loose to medium dense sandy silt (A-4a). The native soil deposits extended to an approximate depth ranging between 5 and 32 feet below the ground surface, where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. A layer of severely weathered rock, ranging in thickness from 1.3 to 8.0 feet was encountered above the more competent, cored bedrock. The bedrock consisted of soft to hard, slightly weathered to decomposed, slightly to highly fractured sandstone, siltstone, and shale. The amount of rock recovered in each core run varied between 88 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 15 and 100 percent with an average of 77 percent indicating good quality rock.

Unconfined compressive strength of tested cores ranged between 2,218 psi and 7,386 psi. The tested cores correspond to samples at depths between 19.5 feet and 33.0 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-20	19.5-19.9	157.2	7,386
B-21	32.6-33.0	155.6	2,218
B-22	20.6-21.0	151.9	3,056

4.2.3 Groundwater Conditions

Seepage was encountered only in borings TR-4, TR-5, and B-23. Where seepage was encountered, it was first observed at depths ranging from 2.5 to 20.7 feet below the ground surface. A measurable water level in the borings prior to rock coring was only encountered in boring B-23 at an approximate depth of 30.5 feet. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels, upon the completion of coring, were present in all borings except boring B-19 between approximate depths of 0.5 and 23.0 feet. It should be noted that the final water levels included drilling water, and consequently, may not be representative of actual groundwater conditions.

It should be noted that groundwater levels may fluctuate with seasonal variations and following periods of heavy or prolonged precipitation, and therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be needed to obtain a more accurate estimate of the groundwater table elevation.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is anticipated that the proposed bridges will have three spans and utilize spill-through abutment slopes. It is understood through comments from ODOT's Office of Structural Engineering (OSE) that cast-in place (CIP) reinforced concrete piles are recommended to support the abutments. The use of driven H-piles (forward abutment only), drilled shafts and spread footings has also been considered to support the abutments.

Additionally, spread footings and drilled shafts bearing on rock have been evaluated to support the piers. A summary of the bridge foundation recommendations is presented in Table 2. Detailed recommendations for the bridge foundations and embankment construction are presented in the following sections.

It should be noted that the bedrock surface varies widely across the project area. The approximate bearing elevations presented in Table 2 indicate the elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 2-Summary of Foundation Recommendations

Structural Element	Structure / Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-18	718.7	CIP Piles	701.8*	Pile Capacity++
			Drilled Shafts	701.8*	40 ksf+++
			Spread Footings	769.0+	3.5 ksf
	Right / B-19	722.1	CIP Piles	703.2*	Pile Capacity++
			Drilled Shafts	703.2*	40 ksf+++
			Spread Footings	769.0+	3.5 ksf
Pier 1	Left / TR-6	710.4	Drilled Shafts	696.4*	40 ksf+++
			Spread Footings	705.4	20 ksf
			Spread Footings	701.4	40 ksf
	Right / B-20	715.7	Drilled Shafts	700.1*	40 ksf+++
			Spread Footings	706.7	20 ksf
			Spread Footings	705.7	40 ksf
Pier 2	Left / B-21	725.2	Drilled Shafts	703.2*	40 ksf+++
			Spread Footings	714.2	20 ksf
			Spread Footings	708.2	40 ksf
	Right / B-22	726.3	Drilled Shafts	706.2*	40 ksf+++
			Spread Footings	718.8	20 ksf
			Spread Footings	711.2	40 ksf
Forward Abutment	Left / B-23	729.4	CIP Piles	688.9*	Pile Capacity++
			HP-12x53 Piles	705.4	70 tons
			Drilled Shafts	688.9*	40 ksf+++
			Spread Footings	764.0+	3.5 ksf
	Right / TR-4	733.0	CIP Piles	690.0*	Pile Capacity++
			HP-12x53 Piles	709.0	70 tons
			Drilled Shafts	690.0*	40 ksf+++
			Spread Footings	764.0+	3.5 ksf

* Includes 5-foot socket into competent rock.
\+ Spread footing founded on embankment fill.
++ Pile capacity should conform to ODOT BDM 202.2.3.2.
+++ End bearing capacity only.

5.1 Bridge Foundation Recommendations-Abutments

It is understood through comments from ODOT's Office of Structural Engineering (OSE) that cast-in-place (CIP) reinforced concrete piles are recommended to support the abutments. The CIP piles would be placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment (per OSE). Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment construction. The allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration. Recommended bearing elevations for the CIP pile foundations are presented in Table 2. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

The Contractor should anticipate the need for significant bracing of the prebored CIP piles to provide stability and ensure proper alignment of the abutment piles. The Contractor should be prepared to perform hand-compaction near the abutment piles as necessary during the construction of the approach embankment.

Due to the thin nature of the soil, driven piles are not being considered to support the rear abutment. Borings drilled for the forward abutment indicate that approximately 28 to 32 feet of soil overlies bedrock. Consequently, it may be difficult to install prebored CIP piles at this location. As an alternative to prebored CIP piles, H-piles driven to capacity near bedrock could be considered to support the forward abutment. Analyses indicate that HP-12x53, 70-ton piles could be driven to capacity at this location. The recommended pile tip elevations are presented in Table 2.

A settlement analysis has indicated that approximately 3 inches of settlement is anticipated at the forward abutment location. To prevent downdrag forces from reducing the allowable pile capacity at the forward abutment location, it is recommended that the foundation soils be allowed to consolidate to at least 85 percent of the total primary consolidation prior to driving piles. Analyses indicate that this would require a waiting period of approximately 120 days prior to driving piles at the forward abutment. Alternatively, piles could be driven to a higher capacity (may require the use of larger piles) to offset the reduction from downdrag forces. The downdrag forces have been estimated to be approximately 84 kips. Due to the likelihood of piles being driven near or to the top of rock, it is recommended that reinforced pile points be used to protect the piles while driving.

Due to the relatively low rigidity of the piles compared to drilled shafts, it is anticipated that the piles will provide low resistance to lateral forces. Therefore, the prebored and socketed CIP pile or driven pile foundation systems may be a concern if significant lateral loads are present.

As an alternative to pile foundations, drilled shafts could also be considered for the support of the abutments. It appears that drilled shafts socketed a minimum of 5 feet into competent rock will be well suited for the support of the proposed abutments. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 2. For additional recommendations on drilled shafts, refer to section 5.4.

Spread footings bearing in the embankment fill may also be considered to support the abutments. An allowable bearing capacity of 3.5 ksf may be used to design the footings. If spread footings are to be considered, an estimate of the elastic settlement/consolidation (from structure loading) of the embankment fill and foundation soils will be required to ensure that differential settlement is within allowable limits. This information can be provided upon request.

5. 2 Bridge Foundation Recommendations-Pier 1

Spread footings could be constructed on the rock encountered by the borings to support Pier 1. Bedrock was generally encountered within 5.0 to 6.5 feet below the ground surface. Spread footings bearing on the upper (more weathered) bedrock may be designed using an allowable bearing capacity of 20 ksf (10 tsf). More competent bedrock was generally encountered within 3.5 to 5.0 feet of the soil-rock interface. If a higher allowable bearing capacity is required, spread footings bearing on the lower (more competent) bedrock may be designed using an allowable bearing capacity of 40 ksf (20 tsf). Recommended bearing elevations for spread footings are presented in Table 2.

Currently, lateral loading and uplift is not anticipated to be a concern at this site. However, if spread footings cannot be used at the piers, drilled shafts may be considered for the support of the piers. If drilled shaft foundations are used to support the piers, a minimum of 5-foot deep socket into competent rock is required. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 2.

For additional recommendations on drilled shafts, refer to section 5.4.

5.3 Bridge Foundation Recommendations-Pier 2

Spread footings could be constructed on the rock encountered by the borings to support Pier 2. Bedrock was generally encountered within 7.5 to 11.0 feet below the ground surface. Spread footings bearing on the upper (more weathered) bedrock may be designed using an allowable bearing capacity of 20 ksf (10 tsf). More competent bedrock was generally encountered within 8.6 to 10.0 feet of the soil-rock interface. If a higher allowable bearing capacity is required, spread footings bearing on the lower (more competent) bedrock may be designed using an allowable bearing capacity of 40 ksf (20 tsf). Recommended bearing elevations for spread footings are presented in Table 2.

Currently, lateral loading and uplift is not anticipated to be a concern at this site. However, if spread footings cannot be used at the piers, drilled shafts may be considered for the support of the proposed piers. If drilled shaft foundations are used to support the piers, a minimum of 5-foot deep socket into competent rock is required. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 2.

For additional recommendations on drilled shafts, refer to section 5.4.

5.4 General Drilled Shaft Recommendations

For end-bearing drilled shafts, it is recommended that skin friction in the overburden soil/fill and shallow rock socket be neglected. The bearing surface should be clean and free of loose material and water prior to placement of concrete. The drilled center-to-center spacing of drilled shafts should generally be no less than 2.5 times their diameter. A qualified representative of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets (friction-type shafts). Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 2,500 pounds per square foot may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance ignoring any end bearing.

Shafts that are installed as friction-type piles must have good sidewall contact with the concrete with preferably rough sides. If any shaft is allowed to sit over 12 hours filled with fluid (water or slurry), the potential for sidewall softening develops. This is especially true with the rock sockets and granular materials. The bedrock material encountered across the site contains shale and argillaceous sandstone that will deteriorate quickly when exposed to water or left to desiccate, losing its strength quickly. If it is anticipated that a drilled shaft excavation will be allowed to remain open for longer than 12 hours, the shaft excavation should be drilled at least 6 inches smaller in diameter and reamed to the design diameter immediately prior to placement of concrete. If a drilled shaft excavation does not have concrete placed within 12 hours of completion of the excavation, the shaft should be oversized 6 inches in diameter.

Drilled shafts that are end bearing and are allowed to remain open for more than 12 hours should be drilled short by at least 12 inches and reamed out to the design bearing depth immediately prior to placement of concrete to prevent softening of the bearing material. If a drilled shaft excavation does not have concrete placed within 12 hours of completion of the excavation, the shaft should be extended 12 inches in depth prior to the placement of concrete.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although significant seepage was encountered only in the borings drilled at the forward abutment (TR-4 and B-23) and between the piers (TR-5), water could flow into the drilled shaft excavations at other locations during installation particularly below the stream level and within wet zones that may be present in the rock or soil. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater.

Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

5.5 Embankment Stability Analysis

Slope stability analyses were performed for the proposed spill-through slopes using the existing and proposed grade elevations provided by TranSystems Corporation. The developed cross sections were characterized by 2H:1V side slopes. Based on the borings information, the embankment foundation at the forward abutment spill-through slope was assumed to consist of 9.5 feet of hard Sandy Silt (A-4a) underlain by stiff to very stiff Sandy Silt (A-4a) overlying Siltstone. At the rear abutment spill-through slope, the embankment foundation was assumed to consist of 7 feet of hard Silt and Clay (A-6a) underlain by Siltstone. The critical factors of safety, assuming end-of-construction (undrained) conditions, were found to be 2.07 and 3.09 for the forward and rear spill-through slopes, respectively. In addition, the critical factors of safety, assuming long-term (drained) conditions, were found to be 1.61 and 1.69 for the forward and rear spill-through slopes, respectively. These critical factors of safety are greater than the generally recommended minimum factor of safety of 1.5 for highway embankments. Based on these findings, the proposed spill-through slopes are considered stable. The results of the stability analyses are presented in Appendix IV.

5.6 Groundwater Considerations

Water seepage was encountered in borings B-23 and TR-4. Where seepage was encountered, it was first observed at depths ranging from approximately 2.5 to 20.7 feet below the ground surface. A measurable water level, prior to coring rock was measured only in boring B-23, at a depth of 30.5 feet. Representative final water levels could not be obtained due to the use of water during rock coring. Excavations for the pier foundations are anticipated to be approximately 9.0 to 17.0 feet deep. Foundation construction on the rock is expected to encounter only minor seepage. Excavations or shafts extending below the bedrock surface may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage, water flow, and precipitation that may enter any excavations.

5.7 General Earthwork Recommendations

The proposed alignment of SR 823 over Morris Lane Blue Run Road (CR 54) traverses a gently to moderately sloping area. Consequently, the placement of fill will be required to construct the approach embankments for the bridges. The maximum fill anticipated is approximately 72 feet, near the proposed rear abutment.

Between 0.5 and 4.0 inches of topsoil were encountered at the ground surface. All topsoil and vegetation within the footprint of the new embankment and roadway should be removed prior to new fill placement. All pavement, and organic soil within 3 feet of subgrade level should also be removed prior to placing fill or pavement materials.

Organic soils were not encountered in any of the borings. However, if organic soils are encountered, it is recommended that at least the top three feet of subgrade soil be removed prior to the construction of the new embankment. Overexcavation may need to be deeper if organic soils are encountered at depths greater than three feet.

The embankments should be constructed in accordance with ODOT Items 203. It is anticipated that the embankments will be constructed with side slopes of 2H:1V or flatter. Based on the materials encountered by the borings, the foundation soils are considered adequately stable under the proposed embankment loads.

Excavations for foundations of the culvert should be prepared in accordance with ODOT Item 503, "Excavation for Structures." Excavations deeper than 5.0 feet must be sloped or shored to protect workers entering the excavations. Refer to OSHA regulations (29CFR Part 1926) concerning sloping and shoring requirements for excavations.

Relative to the footing excavations, the following additional recommendations are presented:

1. All footings should be founded deep enough for frost protection, considered to be 36 inches in this area.

2. Excavation bottoms should be examined by the geotechnical engineer prior to placement of reinforcing steel and concrete in order to determine the suitability of the supporting soils.

3. Excavations should be undercut to suitable bearing material if such material is not encountered at the planned footing level. Such undercuts may be backfilled with a lean mix concrete (1,500 psi @ 28 days) or footing concrete.

4. All footing excavations should be cut to stable side walls and flat bottoms with the bottoms comprised of firm soil undisturbed by the method of excavation or softened by standing water. Concrete should be placed the same day that the footings are excavated.

While excavating for the footings, unsuitable soils may be encountered deeper than indicated by the borings. These unsuitable materials will need to be overexcavated until suitable bearing material is encountered. Overexcavations should be backfilled with compacted engineered fill.

It is understood that a portion of the new embankment will be constructed over an abandoned streambed. It is recommended that two feet of durable Type D riprap be placed within any abandoned stream channels being relocated from underneath the embankment. The rock within the channel should daylight out from underneath the embankment. If soil fill is placed above the riprap, geotextile fabric should be between the soil and the rock to separate the layers.

5.8 Scour Analysis

Particle-size analyses were performed on selected samples collected within the area of the currently proposed stream realignment. The proposed flow line is approximately elevation 705.69 (feet), as indicated on the structure site plan. It is anticipated that the bottom of the proposed stream will be in rock. Table 3 presents the sample locations and the D_{50} and D_{85} sizes from the particle-size analyses.

Table 3, Particle-size Analysis

Boring Number	Ground Surface Elevation (feet)	Sample Depth (feet)	ODOT Classification	D_{85} (mm)	D_{50} (mm)
B-20	715.7	1.0	A-4b	0.1033	0.0210
B-20	715.7	3.5	A-4a	15.9133	0.3539
TR-5	721.0	3.5	A-6a	0.0375	0.0088

6.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning our report.

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams

Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

sjr

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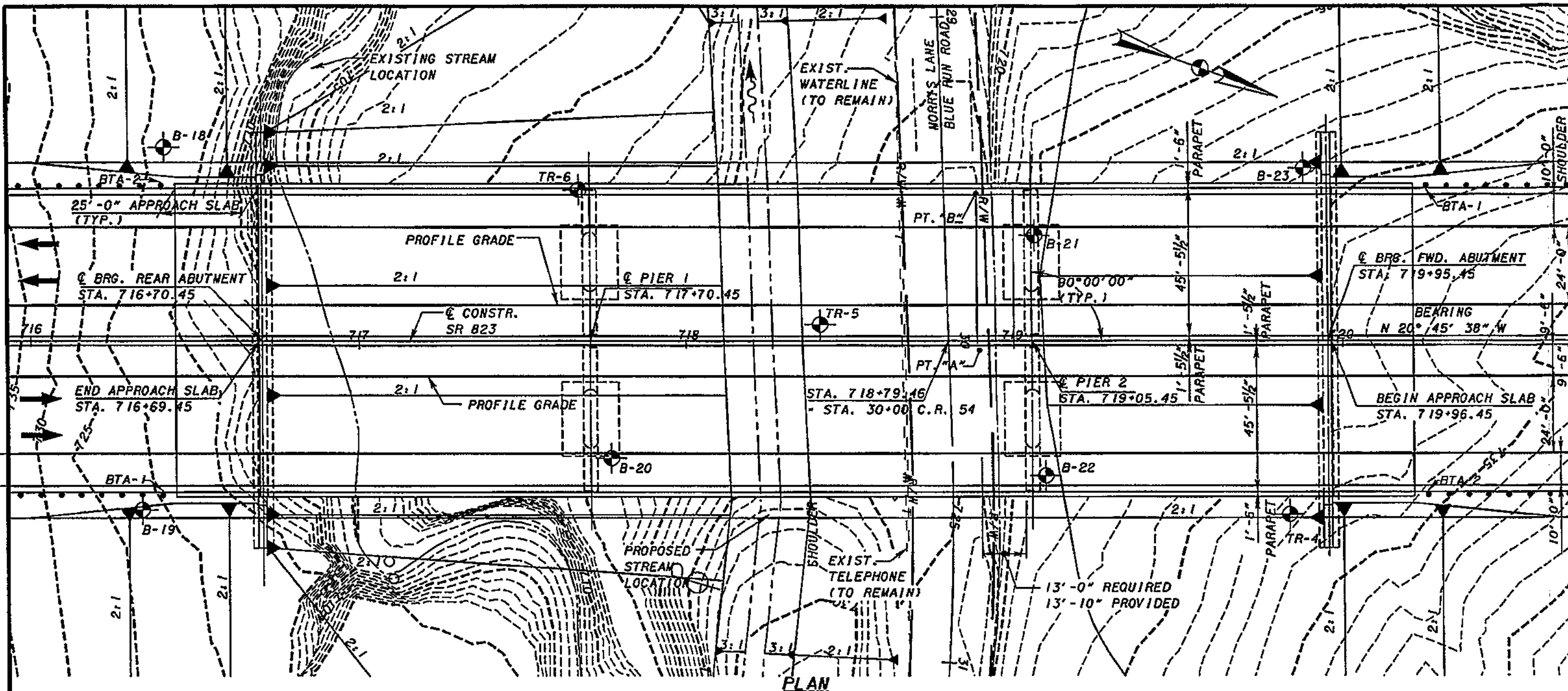
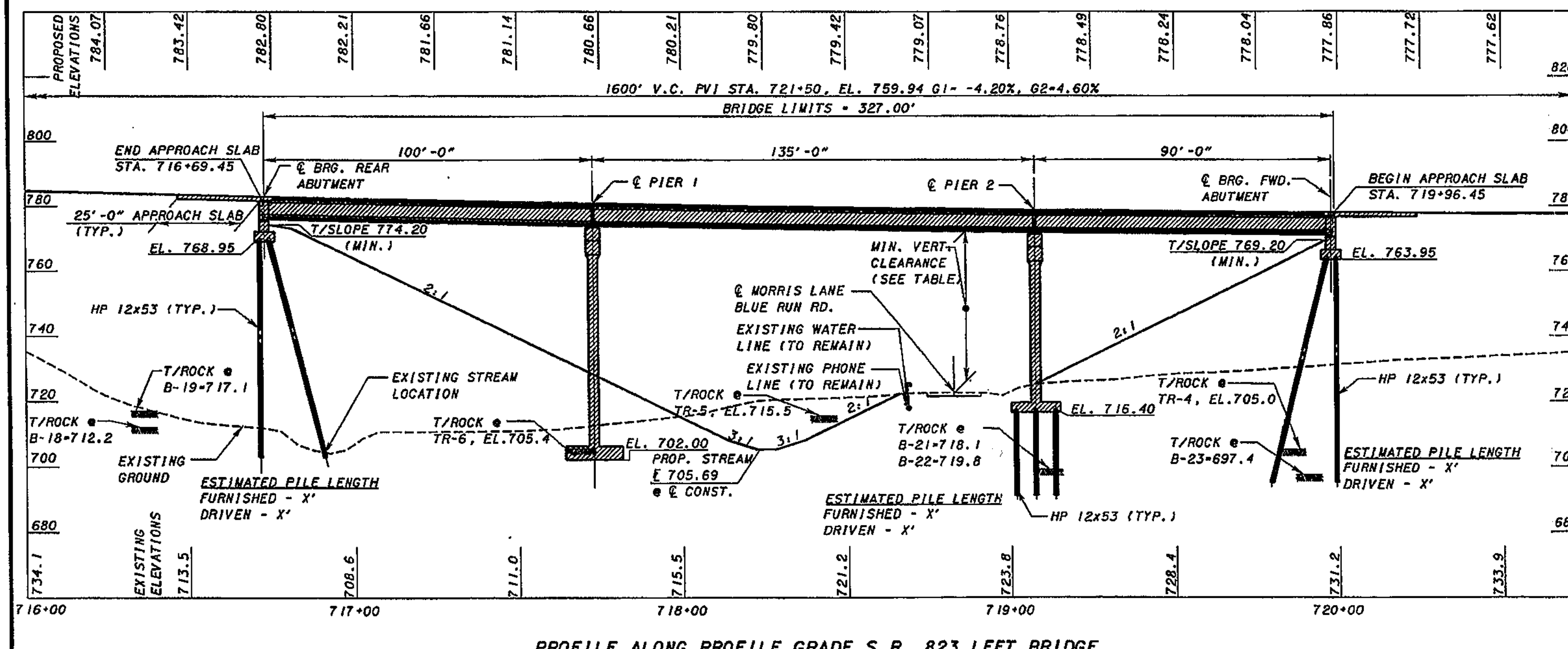
APPENDIX I

Structure Plan and Profile Drawing - 11”x17”





PROFILE ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

BENCHMARK 1	BENCHMARK 2
(TO BE PROVIDED LATER)	(TO BE PROVIDED LATER)

TRAFFIC DATA

(S.R. 823)

CURRENT YEAR ADT (2010) = 19,800
DESIGN YEAR ADT (2030) = 26,000
CURRENT YEAR ADTT (2010) = 2,772
DESIGN YEAR ADTT (2030) = 3,640

HYDRAULIC DATA

DRAINAGE AREA = 147.8 acres
Q_{50} = 294 cfs Q_{100} = 353 cfs
V_{50} = 12.7 fps V_{100} = 13.6 fps
EL_{50} = 709.53 EL_{100} = 709.74

FIRST GUARDRAIL POSTS OFF BRIDGE LOCATIONS

LOCATION	STA.	OFFSET
REAR ABUT.	716+70.17	RT.
REAR ABUT.	716+70.17	LT.
FWD. ABUT.	720+12.83	RT.
FWD. ABUT.	720+12.83	LT.

NOTES:

1. EARTHWORK LIMITS ARE SHOWN APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
2. THE PROPOSED PROFILE GRADE IS WITHIN BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

FOUNDATION DATA:

ALL NEW PILES SHALL BE HP 12 x 53 PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE.

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	49.08'	46.28'
PREFERRED	15.0'	15.0'

PROPOSED STRUCTURE

TYPE: 3 SPAN CONTINUOUS 72" TYPE 4 MODIFIED AASHTO PRESTRESSED CONCRETE I-BEAM WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY T-TYPE PIERS AND SEMI-INTEGRAL ABUTMENT SUBSTRUCTURES ON PILES.

SPANS: 100'-0", 135'-0", 90'-0" C/C SUBSTRUCTURES

ROADWAY: 2 - 42'-0" T/T PARAPETS

LOADING: HS-25 AND ALTERTNATE MILITARY LOADING, FWS=60 PSF

SKEW: NONE

CROWN: NORMAL 0.016 FT/FT

ALIGNMENT: TANGENT

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (25' LONG)

LATITUDE: 82° 56' 56" N

LONGITUDE: 38° 52' 59" W

DESIGN AGENCY: Systems

DESIGNED	CHECKED	DRAWN	REVISED	REVIEWED	DATE
PJP	MSL	CAS	MTN	JRC	07/20/06

STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 716+69.45
STA. 719+96.45

SITE PLAN

BRIDGE NO. SCI-823-1357 L&R
S.R. 823 OVER MORRIS LANE BLUE RUN (C.R. 54)

SCI-823-10.31
PID 79977

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Nine (9) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

Drilling and sampling were conducted in accordance with procedures generally recognized and accepted as standardized methods of investigation of subsurface conditions concerning geotechnical engineering considerations. Borings were drilled with either a truck-mounted or ATV-mounted drill rig.

Drive split-barrel sampling was performed in 1.5 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

In borings where rock was cored, NXM or NQ size diamond coring tools were used.

In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses and plasticity characteristics tests. The results of these tests are shown on the boring logs.

The boring logs included in the Appendix have been prepared on the basis of the field record of drilling and sampling, and the results of the laboratory examination and testing of samples. Stratification lines on the boring logs indicating changes in soil stratigraphy represent depths of changes approximated by the driller, by sampling effort and recovery, and by laboratory test results. Actual depths to changes may differ somewhat from the estimated depths, or transitions may occur gradually and not be sharply defined. The boring logs presented in this report therefore contain both factual and interpretative information and are not an exact copy of the field log.

Although it is considered that the borings have disclosed information generally representative of site conditions, it should be expected that between borings conditions may occur which are not precisely represented by any one of the borings. Soil deposition processes and natural geologic forces are such that soil and rock types and conditions may change in short vertical intervals and horizontal distances.

Soil/rock samples will be stored at our laboratory for a period of six months. After this period of time, they will be discarded, unless notified to the contrary by the client.

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

1. Depth (in feet) – refers to distance below the ground surface.

2. Elevation (in feet) – is referenced to mean sea level, unless otherwise noted.

3. Standard Penetration (N) – the number of blows required to drive a 2-inch O.D., 1-3/8 inch I.D., split-barrel sampler, using a 140-pound hammer with a 30-inch free fall. The blows are recorded in 6-inch drive increments. Standard penetration resistance is determined from the total number of blows required for one foot of penetration by summing the second and third 6-inch increments of an 18-inch drive.

   50/n – indicates number of blows (50) to drive a split-barrel sampler a certain number of inches (n) other than the normal 6-inch increment.

4. The length of the sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.

5. Sample recovery from each drive is indicated numerically in the column headed "Recovery".

6. The drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.

7. The length of hydraulically pressed "Undisturbed" samples is indicated graphically by horizontal lines across the "Press" column.

8. Sample numbers are designated consecutively, increasing in depth.

9. Soil Description

   a. The following terms are used to describe the relative compactness and consistency of soils:

   Granular Soils – Compactness

Term	Blows/Foot Standard Penetration
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	over 50

   Cohesive Soils – Consistency

Term	Unconfined Compression tons/sq.ft.	Blows/Foot Standard Penetration	Hand Manipulation
Very Soft	less than 0.25	below 2	Easily penetrated by fist
Soft	0.25 – 0.50	2 – 4	Easily penetrated by thumb
Medium Stiff	0.50 – 1.0	4 – 8	Penetrated by thumb with moderate pressure
Stiff	1.0 – 2.0	8 – 15	Readily indented by thumb but not penetrated
Very Stiff	2.0 – 4.0	15 – 30	Readily indented by thumb nail
Hard	over 4.0	over 30	Indented with difficulty by thumb nail

   b. Color – If a soil is a uniform color throughout, the term is single, modified by such adjective as light and dark. If the predominant color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled".

   c. Texture is based on the Ohio Department of Transportation Classification System. Soil particle size definitions are as follows:

Description		Size	Description		Size
Boulders		Larger than 8"	Sand	– Coarse	2.0 mm to 0.42 mm
Cobbles		8" to 3"		– Fine	0.42 mm to 0.074 mm
Gravel	– Coarse	3" to ¾"	Silt		0.074 mm to 0.005 mm
	– Fine	¾" to 2.0 mm	Clay		smaller than 0.005 mm

d. The main soil component is listed first. The minor components are listed in order of decreasing percentage of particle size.

e. Modifiers to main soil descriptions are indicated as a percentage by weight of particle sizes.

trace	0 to 10%
little	10 to 20%
some	20 to 35%
"and"	35 to 50%

f. Moisture content of **cohesionless soils** (sands and gravels) is described as follows:

Term	Relative Moisture or Appearance
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Wet	Voids filled with free water

g. The moisture content of **cohesive soils** (silts and clays) is expressed relative to plastic properties.

Term	Relative Moisture or Appearance
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit but below liquid limit
Wet	Moisture content above liquid limit

10. Rock Hardness and Rock Quality Designation

a. The following terms are used to describe the relative hardness of the **bedrock**.

Term	Description
Very Soft	Permits denting by moderate pressure of the fingers. Resembles hard soil but has rock structure. (Crushes under pressure of fingers and/or thumb)
Soft	Resists denting by fingers, but can be abraded and pierced to shallow depth by a pencil point. (Crushes under pressure of pressed hammer)
Medium Hard	Resists pencil point, but can be scratched with a knife blade. (Breaks easily under single hammer blow, but with crumbly edges.)
Hard	Can be deformed or broken by light to moderate hammer blows. (Breaks under one or two strong hammer blow, but with resistant sharp edges.)
Very Hard	Can be broken only by heavy and in some rocks repeated hammer blows.

b. Rock Quality Designation, RQD – This value is expressed in percent and is an indirect measure of rock soundness. It is obtained by summing the total length of all core pieces which are at least four inches long, and then dividing this sum by the total length of the core run.

11. Gradation – when tests are performed, the percentage of each particle size is listed in the appropriate column (defined in Item 9c).

12. When a test is performed to determine the natural moisture content, liquid limit moisture content, or plastic limit moisture content, the moisture content is indicated graphically.

13. The standard penetration (N) value in blows per foot is indicated graphically.

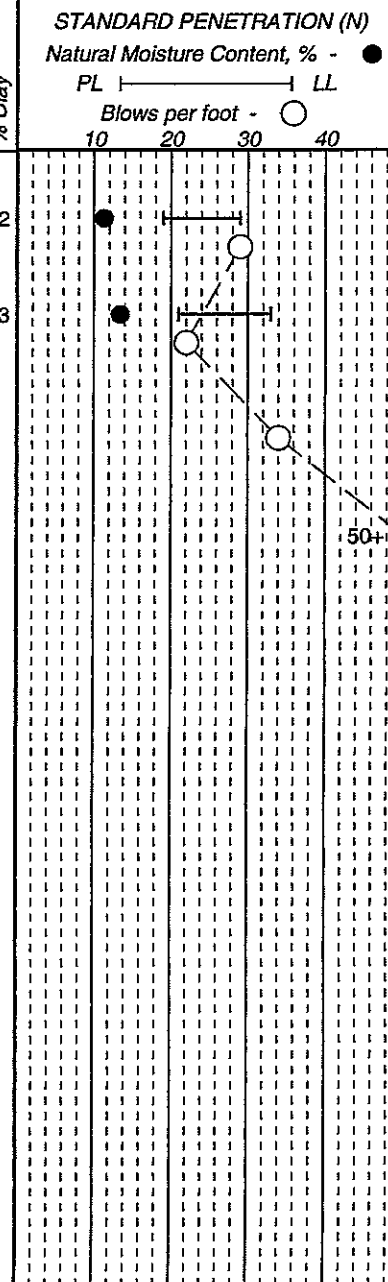
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-18 | Location: Sta. 716+41.2, 59.8 ft. LT of SR 823 CL | Date Drilled: 09/13/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
0.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	718.7												
0.4	718.3						Topsoil - 5"						
		7 12 17		1		4.5+	Hard mottled brown and gray SILT (A-4b), some fine to coarse sand, trace gravel; dry to damp.	5	15	--	8	50	22
3.0	715.7	9 7 15		2		4.5+	Hard mottled brown and gray SILT AND CLAY (A-6a), little to some fine to coarse sand, little gravel; dry to damp.	14	13	--	7	43	23
6.5	712.2	6 13 21		3		-	Severely weathered light brown and gray SILTSTONE, arenaceous.						
		20 49 50/3		4		-							
10.0	708.7	Core 60"	Rec 59"	RQD 85%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, argillaceous, broken, contains abundant argillaceous laminations. @ 10.1', 10.6', 10.8', 10.9', iron stained, low angle fractures. @ 11.3'-11.6', iron stained, high angle fracture.						
11.9	706.8						Medium hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, highly fractured, contains abundant argillaceous laminations.						
15.0	703.7						Bottom of Boring - 15.0'						

Depth scale: 0, 5, 20, 25, 30



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Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

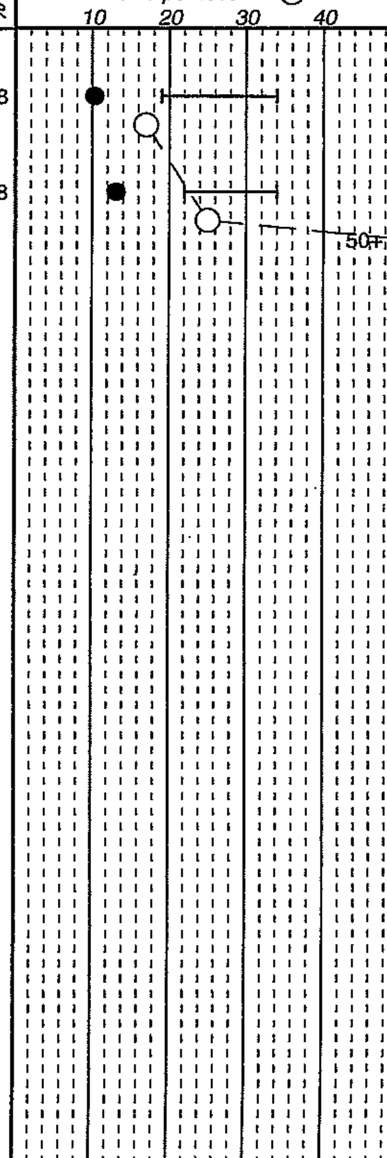
LOG OF: Boring B-19 | *Location:* Sta. 716+34.0, 52.5 ft. RT of SR 823 CL | *Date Drilled:* 09/08/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	722.1												
0.3	721.8						Topsoil - 3"						
		5 6 11	18	1		4.5+	Hard mottled brown and gray SILT AND CLAY (A-6a), some gravel, little to some fine to coarse sand; dry to damp.	32	15	--	9	26	18
		4 10 15	15	2		4.5+		0	12	--	6	54	28
5.0	717.1	50/2	2	3		-	Severely weathered brown SANDSTONE, micaceous.						
6.0	716.1	Core 60"	Rec 16"	RQD 15%	R-1		Very soft to soft light brown gray SANDSTONE; very fine to fine grained, decomposed, arenaceous, highly fractured to broken, contains abundant argillaceous laminations. @ 7.3'-11.0', possible core loss. @ 11.7'-12.8', typical iron stained, low angle fractures.						
13.9	708.2	Core 60"	Rec 60"	RQD 63%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, micaceous, highly fractured to broken, contains abundant argillaceous laminations. @ 15.0'-15.4', qu=2,963 psi.						
16.0	706.1						Bottom of Boring - 16.0'						

Depth scale: 10, 15, 20, 25, 30

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



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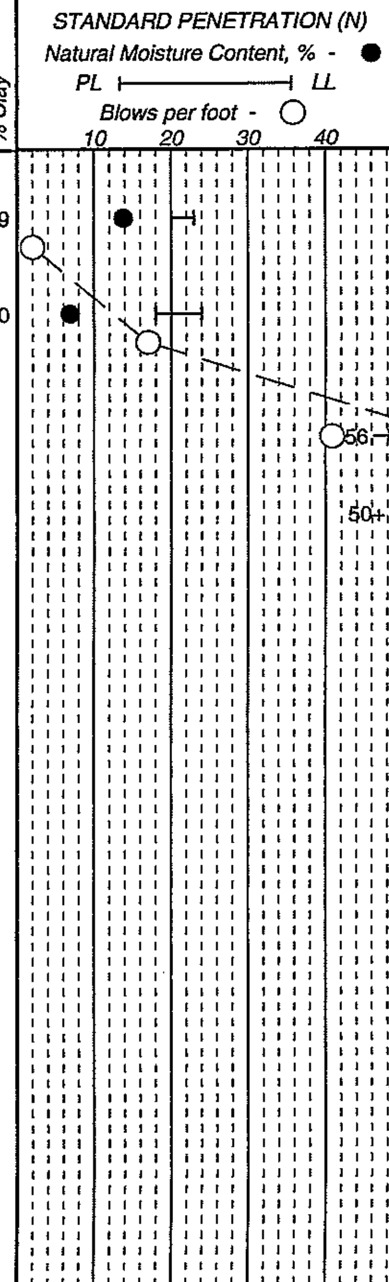
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-20 | Location: Sta. 717+76.8, 36.4 ft. RT of SR 823 CL | Date Drilled: 09/14/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (Prior to coring)
7.0' (Includes drilling water, with augers removed)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	715.7												
0.2	715.5						Topsoil - 2"						
		2 1 1	12	1		0.75	Medium stiff brown SILT (A-4b), little fine to coarse sand, trace gravel; dry to damp.	5	6	–	8	62	19
3.0	712.7	2 7 10	13	2		-	Medium dense brown SANDY SILT (A-4a), some gravel, trace to little clay; dry to damp. @ 4.5'-5.0', trace to little gravel.	30	18	–	13	29	10
6.5	709.2	9 24 32	15	3		-	Severely weathered gray SILTSTONE, arenaceous.						
		17 50/5	10	4		-							
10.0	705.7	Core 48"	Rec 43"	RQD 85%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered, argillaceous, micaceous, highly fractured, contains abundant argillaceous laminations. @ 14.2', 15.3', high angle fractures.						
15.6	700.1	Core 60"	Rec 60"	RQD 96%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, highly fractured, contains abundant argillaceous laminations. @ 19.5'-19.9', qu=7,386 psi.						
		Core 60"	Rec 58"	RQD 96%	R-3								
		Core 60"	Rec 60"	RQD 95%	R-4								
30.0	685.7	Core	Rec	RQD	R-5		Bottom of Boring - 30.0'						



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Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-20 | *Location:* Sta. 717+76.8, 36.4 ft. RT of SR 823 CL | *Date Drilled:* 09/14/06

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: None (Prior to coring)
7.0' (Includes drilling water, with augers removed)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ 10 20 30 40
30	685.7	12"	12"	100%										
35														
40														
45														
50														
55														
60														

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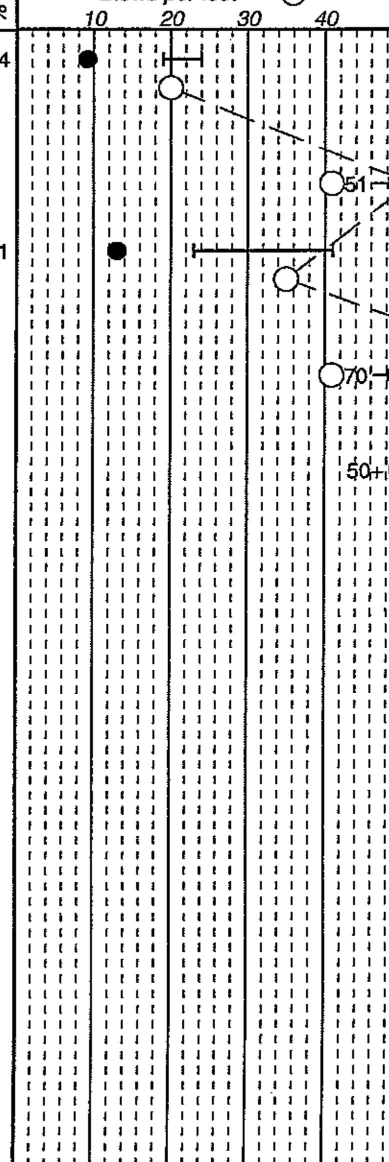
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-21 | Location: Sta. 719+06.4, 32.6 ft. LT of SR 823 CL | Date Drilled: 9/6/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (Prior to coring)
23.0' (Includes drilling water, with augers removed.

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	725.2												
0.3	724.9	2 5 15	10	1		4.5+	Topsoil - 4"						
							Hard brown SILT (A-4b), little clay, little fine to coarse sand, little gravel; damp.	18	9	--	4	55	14
3.0	722.2	12 24 27	18	2		-	Severely weathered brown SANDSTONE; argillaceous.						
4.5	720.7	11 13 22	18	3		4.5+	Hard mottled brown and gray CLAY (A-7-6), "and" silt, trace fine to coarse sand; dry to damp.	0	1	--	2	56	41
7.0	718.2	13 20 50	14	4		-	Severely weathered brown SILTSTONE.						
10		22 35 50/4	12	5		-							
15.0	710.2	Core 54"	Rec 52"	RQD 67%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, micaceous, highly fractured to broken, contains abundant argillaceous laminations. @ 16.5'-16.8', brown, clay filled fractures.						
20		Core 60"	Rec 60"	RQD 75%	R-2		@ 20.1'-22.0', highly weathered.						
24.9	700.3	Core 60"	Rec 56"	RQD 87%	R-3		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, micaceous, moderatly to highly fractured, contains abundant argillaceous laminations.						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



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Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-21 | *Location:* Sta. 719+06.4, 32.6 ft. LT of SR 823 CL | *Date Drilled:* 9/6/06

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: None (Prior to coring)
23.0' (Includes drilling water, with augers removed.

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	695.2	Core 60"	Rec 56"	RQD 83%	R-4		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, micaceous, moderatly to highly fractured, contains abundant argillaceous laminations. @ 32.6'-33.0', qu=2,218 psi. @ 33.9'-34.1', broken.							
34.5	690.7						Bottom of Boring - 34.5'							
35														
40														
45														
50														
55														
60														

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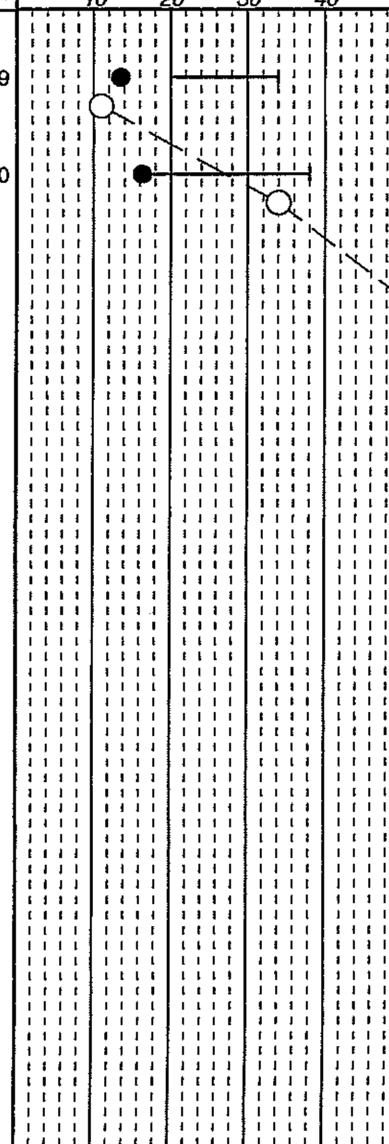
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-22 | Location: Sta. 719+09.4, 41.9 ft. RT of SR 823 CL | Date Drilled: 09/15/06

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: None (Prior to coring)
2.5' (Includes drilling water, with augers removed)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.0	726.3												
0.3	726.0						Topsoil - 4"						
		5 5 6	15	1		-	Stiff brown SILT AND CLAY (A-6a), little to some fine to coarse sand, little gravel; dry to damp.	12	10	–	9	40	29
4.0	722.3	7 14 20	14	2		-	Hard mottled brown and gray SILTY CLAY (A-6b), trace fine to coarse sand, trace gravel; damp to moist.	3	1	–	5	51	40
6.5	719.8	12 25 25	18	3		-	Severely weathered brown SANDSTONE, argillaceous.						
7.5	718.8						Medium hard to hard SANDSTONE; fine to medium grained moderately weathered, micaceous, highly fractured.						
7.9	718.4	Core 24"	Rec 21"	RQD 54%	R-1		Soft brown SILTSTONE; decomposed, highly fractured.						
		Core 60"	Rec 60"	RQD 58%	R-2		@ 13.8'-14.0', iron stained, high angle fractures, broken.						
15.1	711.2	Core 60"	Rec 60"	RQD 70%	R-3		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered, argillaceous, micaceous, highly fractured, contains abundant argillaceous laminations.						
		Core 60"	Rec 60"	RQD 96%	R-4		@ 20.6'-21.0', qu=3,056 psi. @ 23.1'-23.6', high angle fractures.						
		Core 36"	Rec 36"	RQD 83%	R-5								
27.5	698.8						Bottom of Boring - 27.5'						



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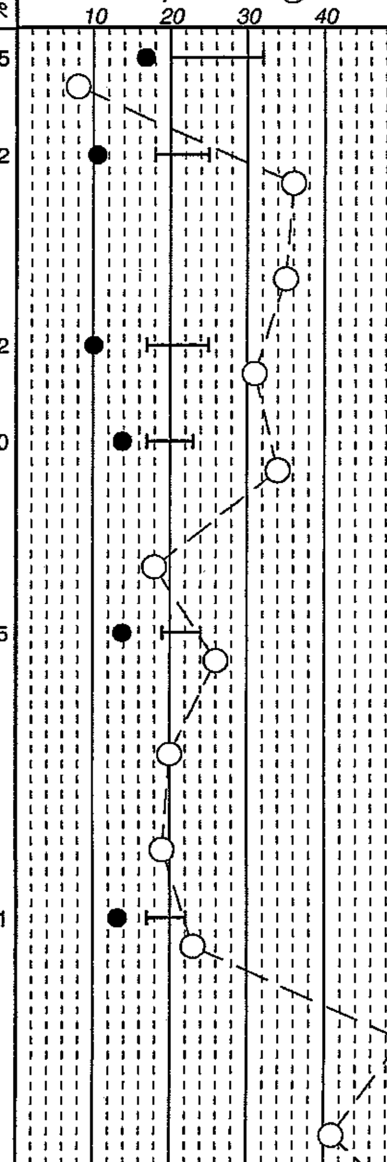
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-23 | *Location:* Sta. 719+88.3, 53.4 ft. LT of SR 823 CL | *Date Drilled:* 9/6/06

WATER OBSERVATIONS:
Water seepage at: 20.7'
Water level at completion: 30.5' (Prior to coring)
10.0' (Includes drilling water, inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	729.4												
0.1	729.3	1 2 6	14	1		2.0	Topsoil - 0.5"	16	11	--	7	41	25
							Stiff to very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, little gravel; damp.						
2.0	727.4	11 15 21	18	2		-	Hard mottled brown and gray SANDY SILT (A-4a), some clay, little gravel; damp.	18	7	--	5	48	22
5		11 12 23	18	3		-							
7.5	721.9	15 14 17	18	4		-	Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), little clay; damp.	49	11	--	12	16	12
9.5	719.9	12 15 19	18	5		1.75	Stiff to very stiff mottled brown and gray SANDY SILT (A-4a), little to some clay, little to some fine to coarse sand, trace to little gravel; damp.	7	8	--	16	49	20
10		5 6 12	18	6		3.0							
15		4 8 18	18	7		3.0		11	12	--	20	42	15
		6 9 11	18	8		3.0							
20		5 7 12	18	9		2.0	@ 20.0'-29.0', brown, some to "and" gravel.						
		4 8 15	18	10		-		31	18	--	12	28	11
25		12 23 27	18	11		-	@ 25.0'-30.0', hard.						
		13 20 21	18	12		-							
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



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Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-23 | Location: Sta. 719+88.3, 53.4 ft. LT of SR 823 CL | Date Drilled: 9/6/06

WATER OBSERVATIONS: Water seepage at: 20.7'
Water level at completion: 30.5' (Prior to coring)
10.0' (Includes drilling water, inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N): Natural Moisture Content, % - ●; PL ├─┤ LL; Blows per foot - ○
30	699.4	15 25 29	15	13		-	Stiff to very stiff brown SANDY SILT (A-4a), little clay, some gravel; damp to moist.							54
32.0	697.4	10 15 20	18	14		-	Severely weathered brown SILTSTONE, arenaceous.							
35.5	693.9	50/5	5	15		-								50+
		Core 60"	Rec 60"	RQD 53%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, decomposed, argillaceous, micaceous, highly fractured, contains abundant argillaceous laminations. @ 37.5', 39.8', 40.0', 40.2', clay filled, low angle fractures. @ 38.3'-38.5', high angle fracture.							
40.5	688.9						Bottom of Boring - 40.5'							

Depth scale (ft): 30, 35, 40, 45, 50, 55, 60

Blows per foot scale: 10, 20, 30, 40

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

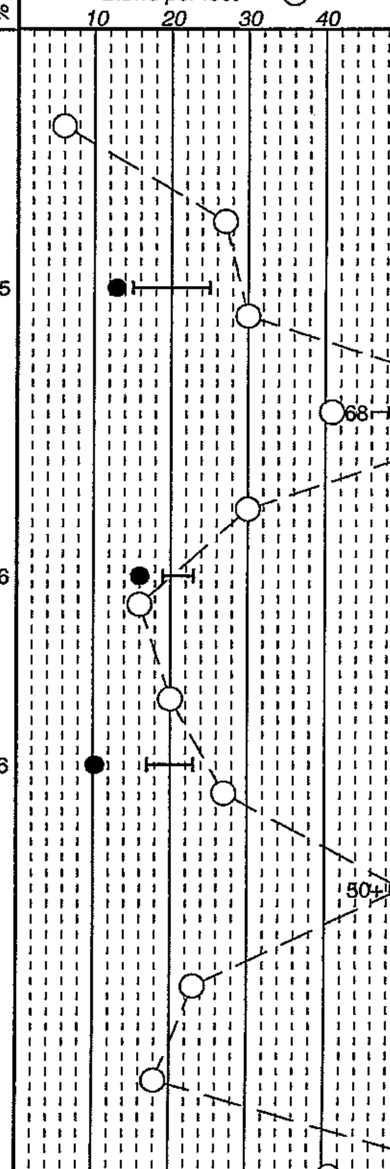
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-4 | Location: Sta. 719+83.4, 53.9 ft. RT of SR 823 CL | Date Drilled: 03/16/05

WATER OBSERVATIONS: Water seepage at: 2.5'
Water level at completion: 9.4' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	733.0												
0.3	732.7						Topsoil - 3"						
		1 2 4	18	1		0.75	Medium stiff brown SILTY CLAY (A-6b), trace fine sand; damp.						
3.0	730.0	4 10 17	18	2		1.5	Stiff brown and red SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; damp.						
5													
5.5	727.5	7 13 17	18	3		--	Very stiff to hard reddish brown SANDY SILT (A-4a), little clay, trace to little gravel; damp.	10	6	--	21	48	15
		9 35 33	18	4		--							
10													
		7 13 17	17	5		--							
13.0	720.0	6 7 9	16	6		--	Very stiff brown and gray SILT (A-4b), some fine to coarse sand, little clay; damp.	0	6	--	18	60	16
15													
17.0	716.0	12 12 8	18	7A 7B		4.5+	Hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace to little gravel; damp.						
18.0	715.0	7 12 15	8	8		--	Very stiff brown SANDY SILT (A-4a), little clay, trace gravel; damp.	4	21	--	23	36	16
20													
		16 25 50/5	16	9		--	@ 21.5', possible decomposed boulder.						
23.0	710.0	5 11 12	18	10		2.75	Very stiff brown and gray SILTY CLAY (A-6b), trace to little fine to coarse sand; damp.						
25													
		8 9 9	18	11		3.25							
28.0	705.0	12 28 33	18	12			Severely weathered gray SHALE.						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-4 | Location: Sta. 719+83.4, 53.9 ft. RT of SR 823 CL | Date Drilled: 03/16/05

WATER OBSERVATIONS: Water seepage at: 2.5'
Water level at completion: 9.4' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	703.0						Severely weathered gray SHALE.							61
		16 37 50/4	16	13										50+
35.0	698.0						Soft to medium hard gray SHALE; thinly laminated to laminated, moderately to highly weathered, moderately fractured to broken, contains ferric bands. @ 35.8'-36.1',37.3'-37.6', decomposed zones. @ 37.6'-38.0', high angle fracture.							
40		Core 120"	Rec 120"	RQD 63%	R-1		@ 39.6', high angle fracture. @ 42.3', thin clay seam.							
45.0	688.0						Bottom of Boring - 45.0'							
50														
55														
60														

Natural Moisture Content, % - ● ; PL |——| LL ; Blows per foot - ○ ; scale 10 20 30 40

FILE: 0121-3070-03 [5/16/2007 6:23 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

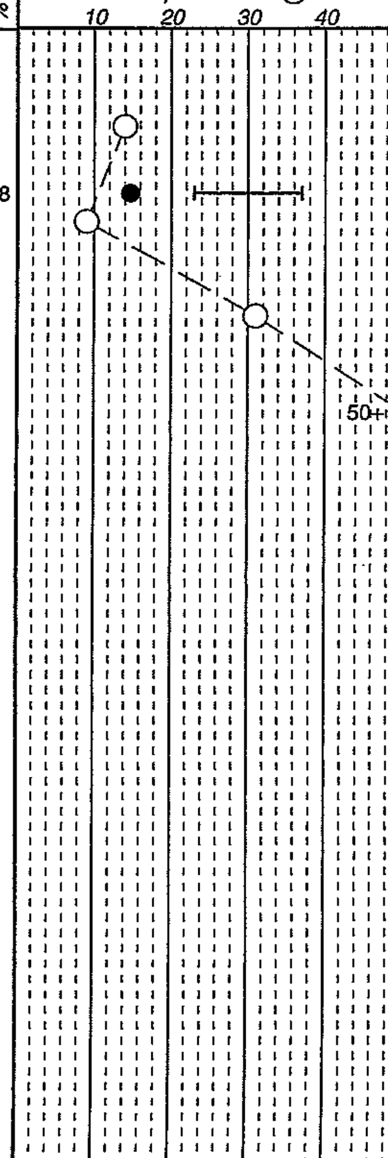
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-5 | *Location:* Sta. 718+40.7, 5.1 ft. LT of SR 823 CL | *Date Drilled:* 3/15/05

WATER OBSERVATIONS: Water seepage at: 2.5'-3.5'
Water level at completion: None (prior to coring)
3.2' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	721.0												
0.3	720.7						Topsoil - 3"						
		5 6 8	18	1		4.5+	Hard brown and gray SILT AND CLAY (A-6a), trace fine to coarse sand; damp.						
		3 5 4	16	2		1.0	@ 3.5', medium stiff.	0	3	--	3	56	38
5.5	715.5	5 11 20	18	3			Severely weathered brown and gray SHALE.						
		20 33 50/1	10	4									
10.0	711.0	Core 36"	Rec 36"	RQD 89%	R-1		Soft brown and gray SHALE; highly weathered to decomposed, broken.						
12.4	708.6	Core 120"	Rec 120"	RQD 100%	R-2		Soft to medium hard gray SHALE; moderately to highly weathered, arenaceous, thinly laminated to laminated, slightly to moderately fractured. @ 19.8'-19.9', decomposed.						
23.0	698.0						Bottom of Boring - 23.0'						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL |——| LL
Blows per foot - ○



FILE: 0121-3070-03 [5/16/2007 6:23 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-6 | Location: Sta. 717+67.1, 46.6 ft. LT of SR 823 CL | Date Drilled: 03/15/05 to 03/16/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
9.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○
	710.4													
0.2	710.2						Topsoil - 2"							
		1 3 4	18	1			Loose brown SANDY SILT (A-4a), some gravel, trace clay; contains sandstone fragments; damp.							
3.0	707.4	4 8 18	16	2			Very stiff gray SILTY CLAY (A-6b), trace to little fine sand; dry to damp.							
5.0	705.4	Core 48"	Rec 48"	RQD 54%	R-1		Soft to medium hard brownish gray SANDSTONE; very fine to fine grained, decomposed, slightly fractured.							
9.0	701.4	Core 120"	Rec 120"	RQD 83%	R-2		Medium hard gray SANDSTONE; very fine grained, moderately weathered, argillaceous, micaceous, laminated to thinly bedded, slightly to highly fractured. @ 9.2'-9.4', 15.4'-15.9', clay seams @ 11.9', high angle fracture. @ 12.5'-12.7', decomposed zone.							
19.0	691.4						Bottom of Boring - 19.0'							

Depth scale: 10, 15, 20, 25, 30

Blows per foot scale: 10, 20, 30, 40

FILE: 0121-3070-03 [5/16/2007 6:23 PM]

APPENDIX III

Laboratory Test Results

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

DLZ Project No.: 0121-3070.03 | **Client: TranSystems**

Project Name: SCI-823-0.00 | **Date:** 12/19/2006

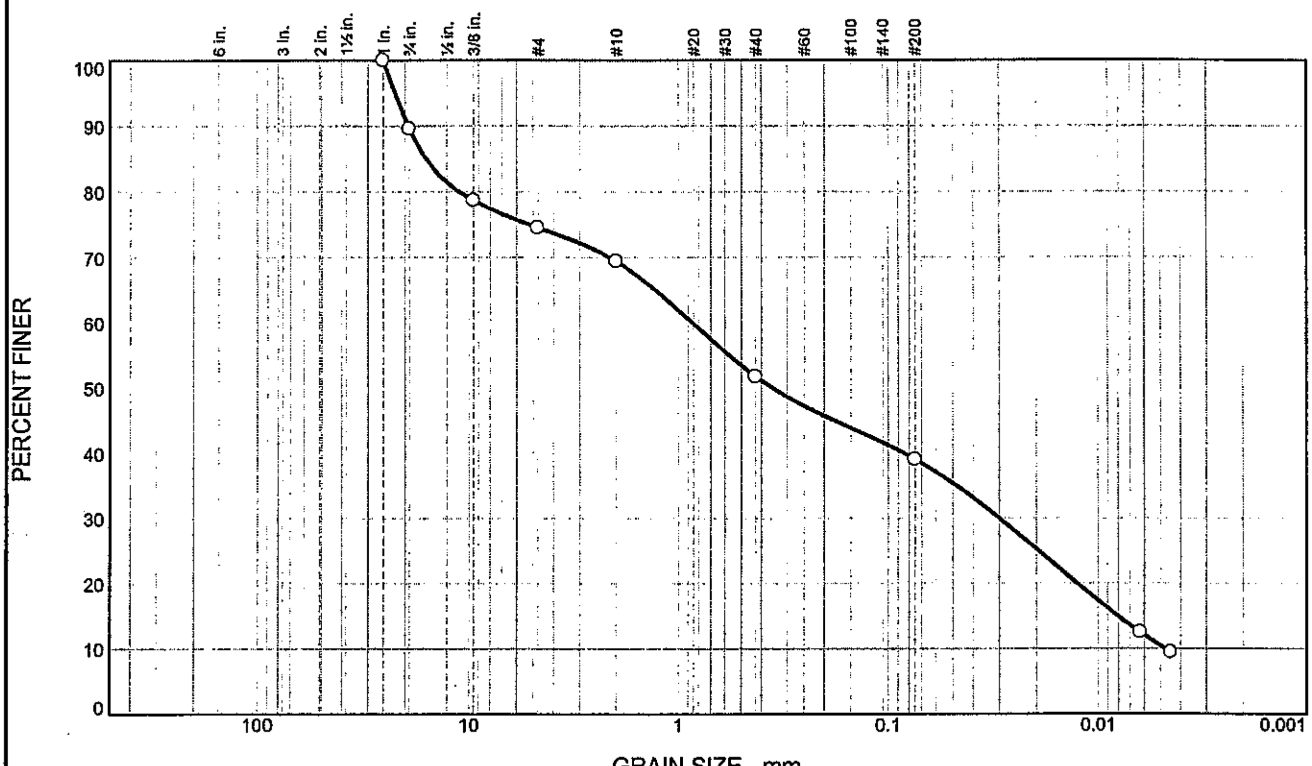
Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-21	R-4	32.6-33.0	1.961	1.958	1.971	1.963	4.665	4.669	4.668	4.667	2.378	0.0081678	576.59	155.63	6,710	**2,218**
			1.956	1.965	1.965											
B-22	R-4	30.6-21.0	1.937	1.938	1.955	1.942	4.532	4.533	4.532	4.532	2.334	0.0077641	535.16	151.96	9,050	**3,056**
			1.943	1.938	1.940											
B-20	R-3	19.5-19.9	1.972	1.967	1.966	1.969	4.445	4.449	4.444	4.446	2.259	0.0078268	558.27	157.25	22,480	**7,386**
			1.970	1.969	1.967											

DLZ

Engineers * Architects * Scientists

6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.4	15.1	5.1	17.7	12.6	28.7	10.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00	100.0		
0.75	89.6		
0.375	78.7		
#4	74.5		
#10	69.4		
#40	51.7		
#200	39.1		

* (no specification provided)

Material Description

Silty, clayey sand with gravel

Atterberg Limits (ASTM D 4318)

PL= 18 LL= 24 PI= 6

Classification

USCS= SC-SM AASHTO= A-4(0)

Coefficients

D_{85}= 15.9133 D_{60}= 0.8584 D_{50}= 0.3539
D_{30}= 0.0301 D_{15}= 0.0081 D_{10}= 0.0048
C_u= 179.15 C_c= 0.22

Date Tested: **Tested By:**

Remarks

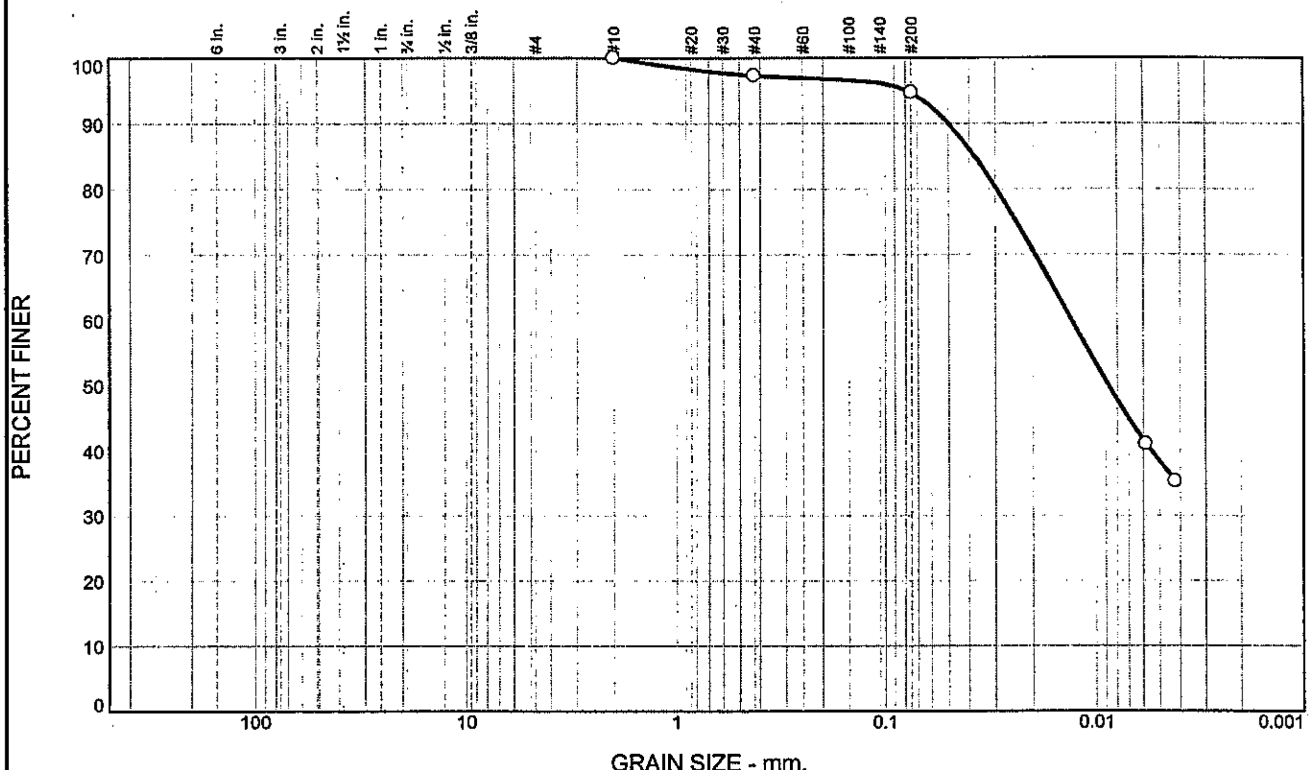
Moisture Content= 6.9%

Sample No.: 2 **Source of Sample:** B-20 **Date Sampled:** 12/15/06
Location: **Elev./Depth:** 3.5
Checked By: **Title:**

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	2.7	2.6	56.5	38.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	97.3		
#200	94.7		

* (no specification provided)

Material Description

Lean clay

Atterberg Limits (ASTM D 4318)

PL= 23 LL= 37 PI= 14

Classification

USCS= CL AASHTO= A-6(14)

Coefficients

D_{85}= 0.0375 D_{60}= 0.0132 D_{50}= 0.0088
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Date Tested: **Tested By:**

Remarks

Moisture Content= 14.7%

Sample No.: 2 **Source of Sample:** TR-5 **Date Sampled:** 4/6/05
Location: **Elev./Depth:** 3.5
Checked By: **Title:**

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

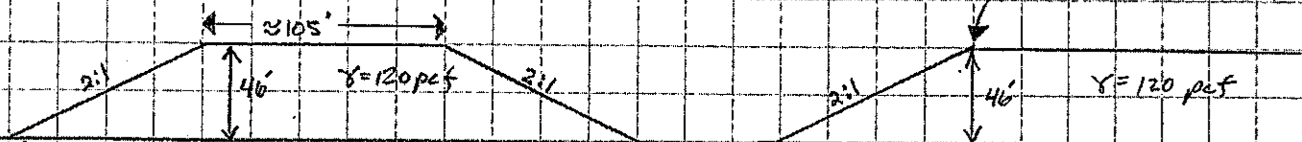
APPENDIX IV

Settlement Calculations
Drilled Shaft – End Bearing and Side Resistance Calculations
Driven-Pile Analysis
Slope Stability Analysis

* Soil profile encountered in boring B-23 is assumed to be the most critical.

X-Section View — Profile View

Forward Abutment location



9.5' — ① $C_c = 0.11$ $C_r = 0.011$ $e_o = 0.531$; $\gamma = 120$ pcf; Sandy Silt (A-4a) — Parameters estimated as per NHI-00-045. * See sample calculations below

12.5' — ② $C_c = 0.14$ $C_r = 0.014$ $e_o = 0.575$; $\gamma = 120$ pcf; Sandy Silt (A-4a) — Parameters as per NHI-00-045

13.5' — ③ $\bar{N} \approx 41.5$ $\bar{N}' \approx 0.78(41.5) = 32.4$; $C' \approx 72$ $\gamma = 120$ pcf; Sandy Silt (A-4a) — Parameters as per NHI-00-045

Sample Calculation for Layer 1

$\bar{\omega} \simeq 11\%$ $\qquad C_c = \frac{\omega}{100} = \frac{11}{100} = 0.11 \qquad C_r = \frac{\omega}{1000} = \frac{11}{1000} = 0.011$

*Not Saturated $\qquad e_o = \frac{G_s \cdot \gamma_w}{\gamma_d} - 1$ where; $\gamma_d = \frac{\gamma}{1+\omega} = \frac{120 \text{ pcf}}{1+0.11} = 108$ pcf

$$e_o = \frac{(2.65)(62.4)}{108} - 1 = 0.531$$

Estimate Coefficient of Consolidation

$LL \approx 24\%$ $\qquad C_v \simeq 0.7$ ft²/day [Ref: NHI-00-045 & NAVFAC, DM-7.1, 1982]

Primary Consolidation has been estimated to be approximately 2.6".

At least 85% of primary consolidation should be achieved prior to driving piles. - To prevent downdrag forces.

$$t = \frac{T_v \cdot H^2}{C_v}$$

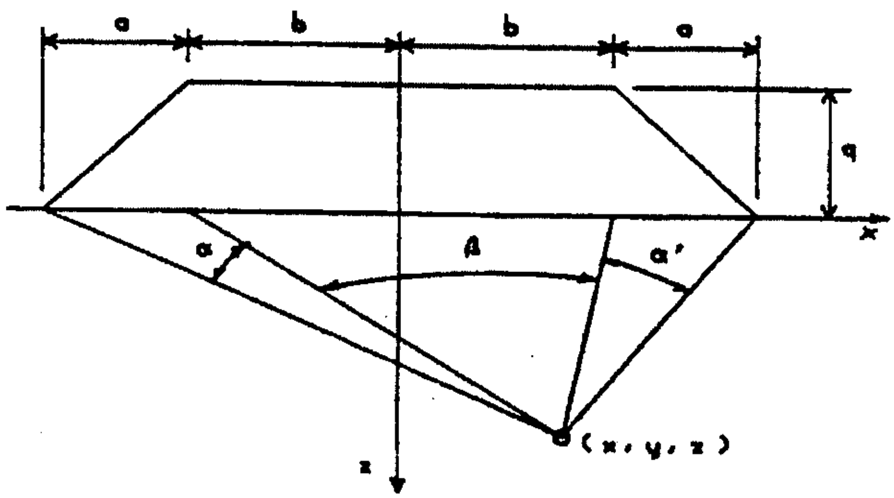
*Assume double drainage $H_{dr} = 11'$
*$U = 85\% \rightarrow T_v = 0.685$

$$t_{85} = \frac{(0.685)(11')^2}{0.7 \text{ ft}^2/\text{day}} = 118 \text{ days}$$

DLZ SUBJECT

Client	TranSystems / ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	3 OF 14
Item	Consolidation Parameters	COMP. BY	SJR DATE 4-11-07
823 over Morris Lane - Blue Run Road		CHECKED BY	DAA DATE 4-11-07

INCREASE IN VERTICAL STRESS DUE TO EMBANKMENT LOADING



q = 5520 load

a = 92 width of slope

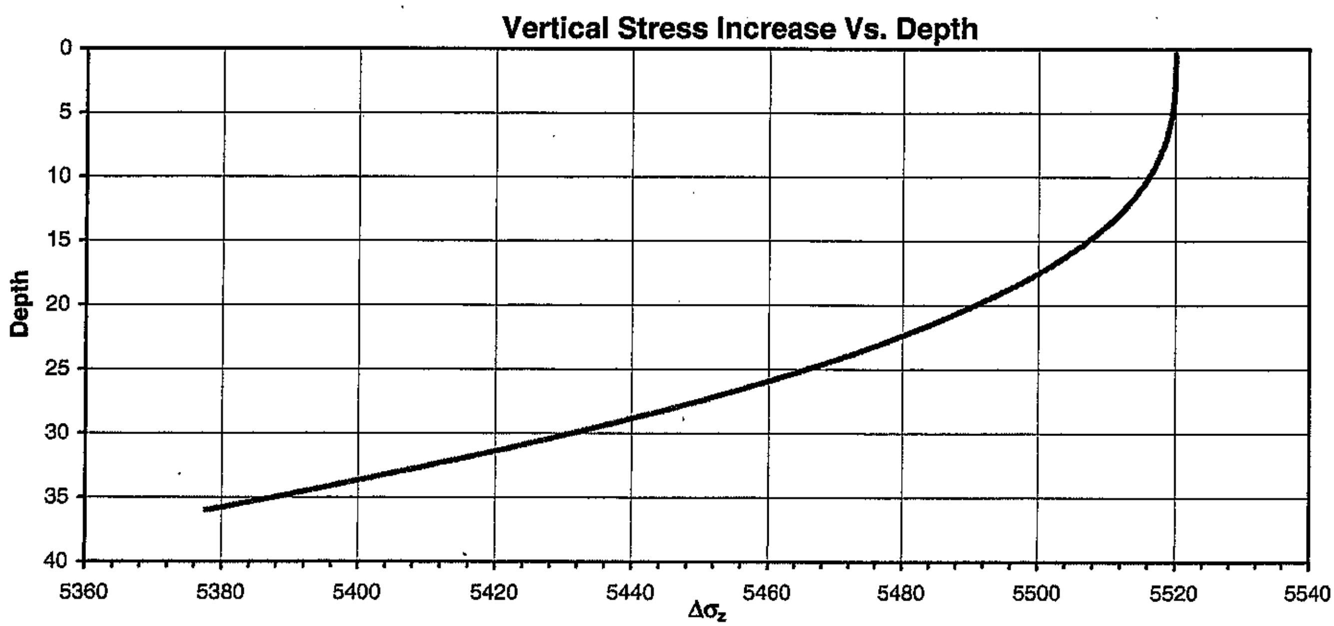
b = 52.5 top half-width of embankment

x = 0 distance from CL

z = 0 to 36 depth range

$$\sigma_v(z) := \left(\frac{q}{\pi\, a}\right)\left(a\cdot(\alpha(z) + \beta(z) + \alpha'(z)) + b\cdot(\alpha(z) + \alpha'(z)) + x\cdot(\alpha(z) - \alpha'(z))\right)$$

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right] + \operatorname{atan}\left[\frac{(b+x)}{z}\right] \;;\quad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right] - \operatorname{atan}\left[\frac{(b-x)}{z}\right] \quad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right] - \operatorname{atan}\left[\frac{(b+x)}{z}\right]$$



Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\ML-BR.DVN
Project Name: SCI-823 Project Date: 04/06/2007
Project Client: TranSystems
Computed By: SJR
Project Manager: Nix

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 5.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	30.00 ft
	- Driving/Restrike	30.00 ft
	- Ultimate:	30.00 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	22.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel
2	Cohesionless	13.50 ft	0.00%	120.00 pcf	36.0/36.0	Nordlund

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
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	1165.00 psf	18.55 Kips
18.01 ft	Cohesive	N/A	N/A	1199.55 psf	61.97 Kips
21.99 ft	Cohesive	N/A	N/A	1245.81 psf	84.05 Kips
22.01 ft	Cohesionless	2640.60 psf	27.11	N/A	84.16 Kips
29.99 ft	Cohesionless	3119.40 psf	27.11	N/A	134.55 Kips
30.01 ft	Cohesionless	3600.29 psf	27.11	N/A	134.70 Kips
35.49 ft	Cohesionless	3758.11 psf	27.11	N/A	176.38 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
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
21.99 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
22.01 ft	Cohesionless	2641.20 psf	77.60	149.38 Kips	139.89 Kips
29.99 ft	Cohesionless	3598.80 psf	77.60	149.38 Kips	149.38 Kips
30.01 ft	Cohesionless	3600.58 psf	77.60	149.38 Kips	149.38 Kips
35.49 ft	Cohesionless	3916.22 psf	77.60	149.38 Kips	149.38 Kips

DRIVING - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	17.74 Kips	17.74 Kips
9.01 ft	18.55 Kips	17.74 Kips	36.29 Kips
18.01 ft	61.97 Kips	17.74 Kips	79.71 Kips
21.99 ft	84.05 Kips	17.74 Kips	101.78 Kips
22.01 ft	84.16 Kips	139.89 Kips	224.05 Kips
29.99 ft	134.55 Kips	149.38 Kips	283.93 Kips
30.01 ft	134.70 Kips	149.38 Kips	284.08 Kips
35.49 ft	176.38 Kips	149.38 Kips	325.76 Kips

ULTIMATE - SKIN FRICTION

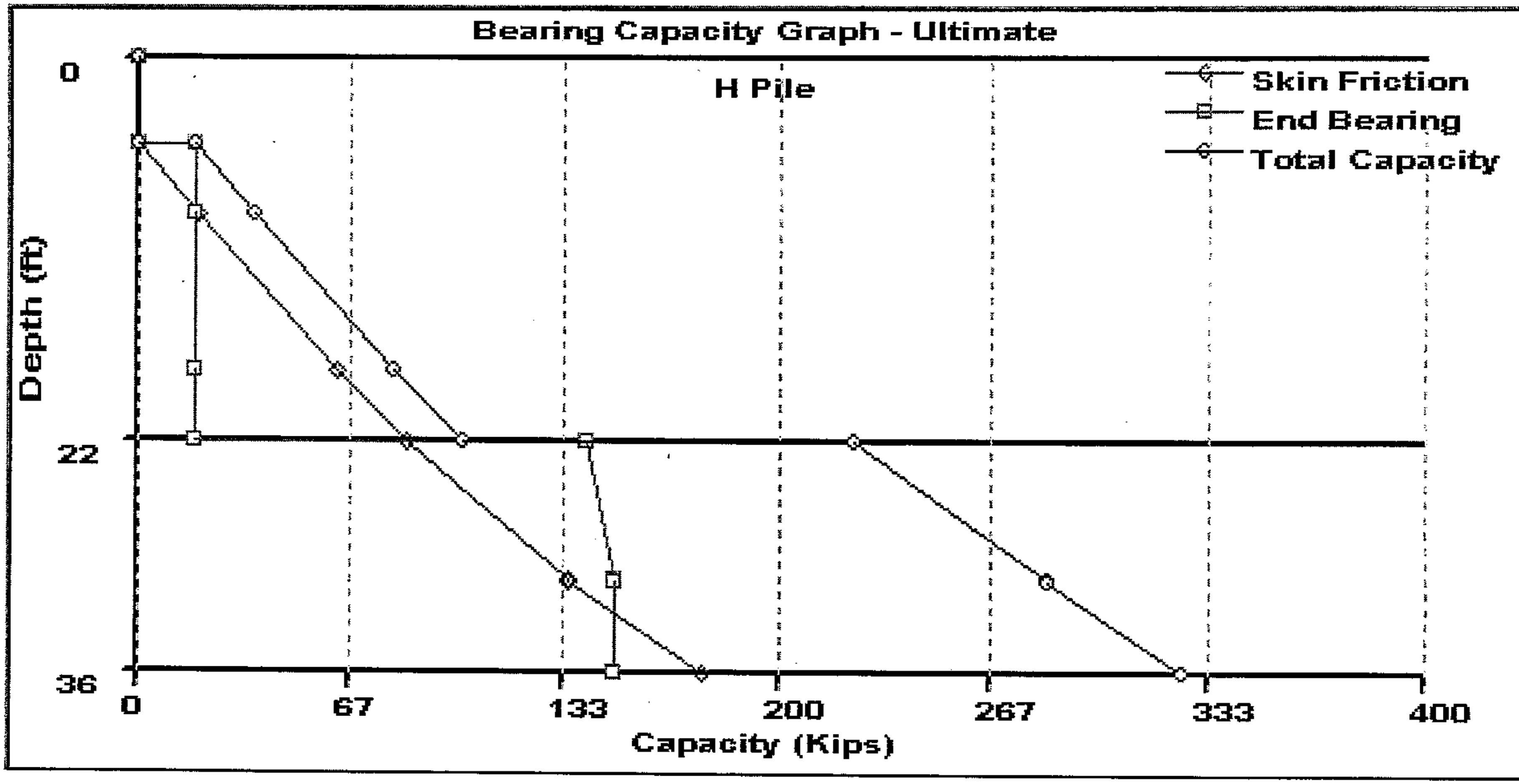
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	1165.00 psf	18.55 Kips
18.01 ft	Cohesive	N/A	N/A	1199.55 psf	61.97 Kips
21.99 ft	Cohesive	N/A	N/A	1245.81 psf	84.05 Kips
22.01 ft	Cohesionless	2640.60 psf	27.11	N/A	84.16 Kips
29.99 ft	Cohesionless	3119.40 psf	27.11	N/A	134.55 Kips
30.01 ft	Cohesionless	3600.29 psf	27.11	N/A	134.70 Kips
35.49 ft	Cohesionless	3758.11 psf	27.11	N/A	176.38 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
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
21.99 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
22.01 ft	Cohesionless	2641.20 psf	77.60	149.38 Kips	139.89 Kips
29.99 ft	Cohesionless	3598.80 psf	77.60	149.38 Kips	149.38 Kips
30.01 ft	Cohesionless	3600.58 psf	77.60	149.38 Kips	149.38 Kips
35.49 ft	Cohesionless	3916.22 psf	77.60	149.38 Kips	149.38 Kips

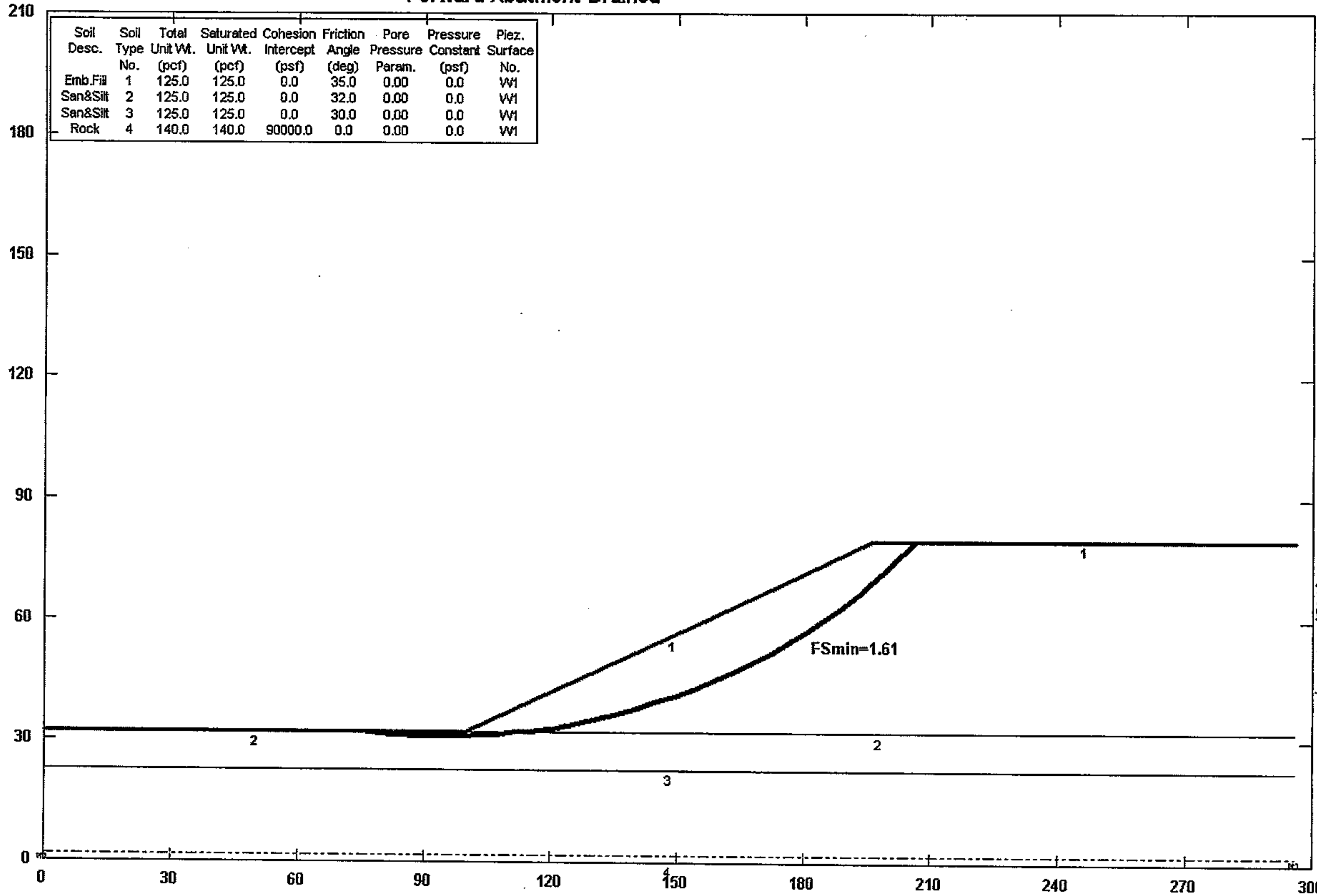
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	17.74 Kips	17.74 Kips
9.01 ft	18.55 Kips	17.74 Kips	36.29 Kips
18.01 ft	61.97 Kips	17.74 Kips	79.71 Kips
21.99 ft	84.05 Kips	17.74 Kips	101.78 Kips
22.01 ft	84.16 Kips	139.89 Kips	224.05 Kips
29.99 ft	134.55 Kips	149.38 Kips	283.93 Kips
30.01 ft	134.70 Kips	149.38 Kips	284.08 Kips
35.49 ft	176.38 Kips	149.38 Kips	325.76 Kips


Bearing Capacity Graph - Ultimate
H Pile
Skin Friction
End Bearing
Total Capacity
Depth (ft)
0
22
36
0
67
133
200
267
333
400
Capacity (Kips)

Portsmouth Bypass-Morris Lane Blue Run
Forward Abutment-Drained

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Emb.Fill	1	125.0	125.0	0.0	35.0	0.00	0.0	W1
San&Silt	2	125.0	125.0	0.0	32.0	0.00	0.0	W1
San&Silt	3	125.0	125.0	0.0	30.0	0.00	0.0	W1
Rock	4	140.0	140.0	90000.0	0.0	0.00	0.0	W1

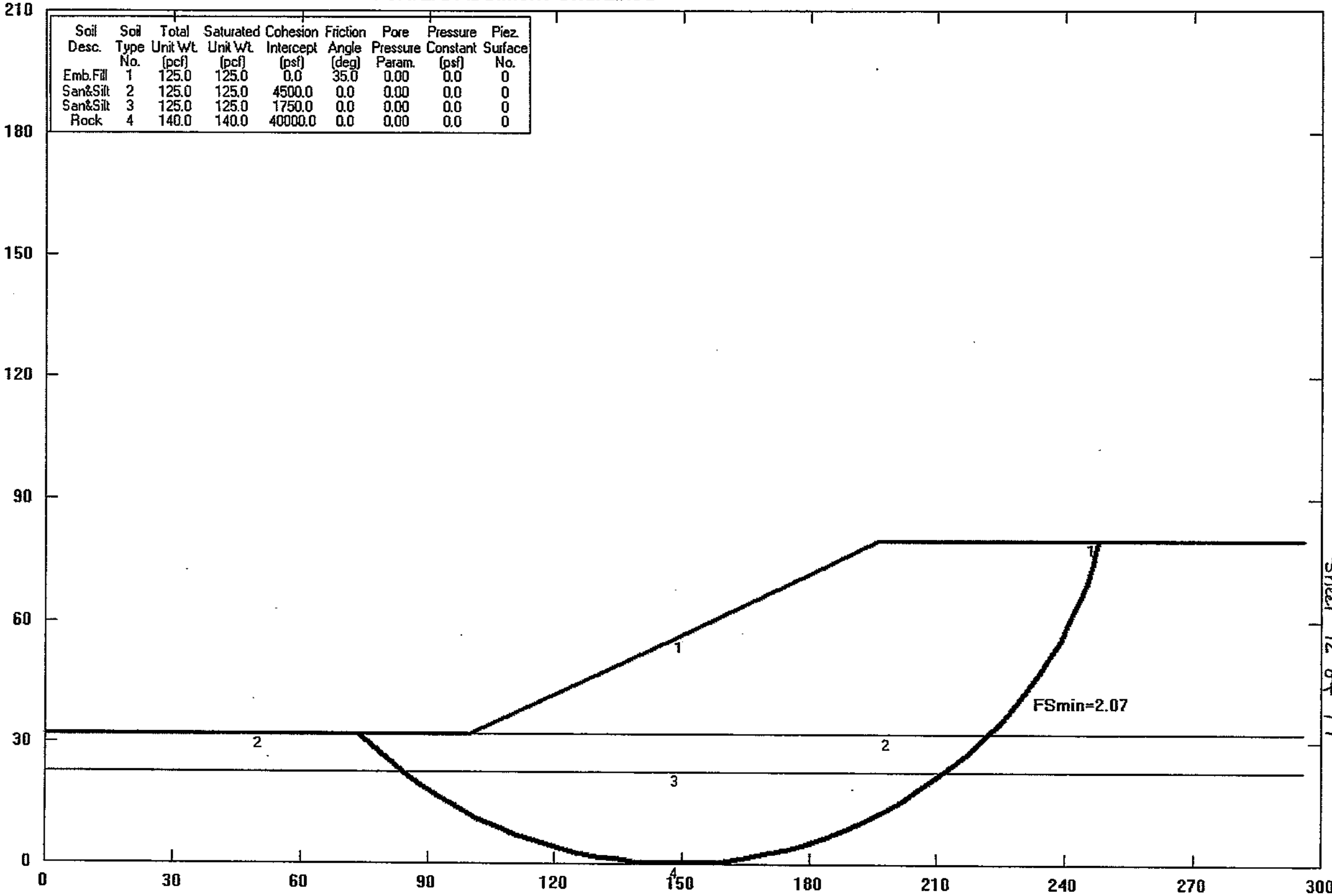


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Portsmouth Bypass-Morris Lane Blue Run
Forward Abutment-Undrained

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Emb.Fill	1	125.0	125.0	0.0	35.0	0.00	0.0	0
San&Silt	2	125.0	125.0	4500.0	0.0	0.00	0.0	0
San&Silt	3	125.0	125.0	1750.0	0.0	0.00	0.0	0
Rock	4	140.0	140.0	40000.0	0.0	0.00	0.0	0

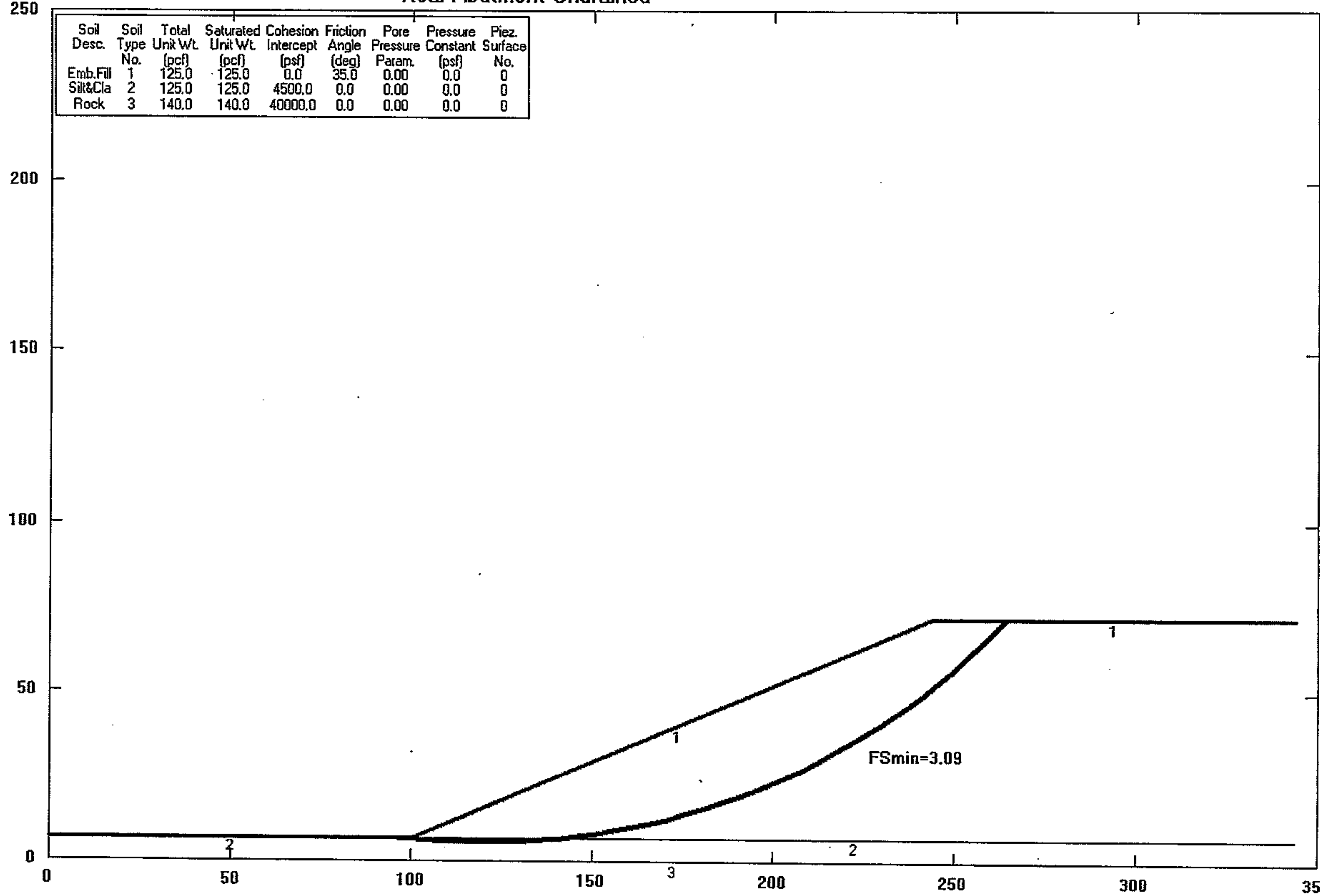


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Portsmouth Bypass-Morris Lane Blue Run
Rear Abutment-Undrained

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Emb.Fill	1	125.0	125.0	0.0	35.0	0.00	0.0	0
Silt&Cla	2	125.0	125.0	4500.0	0.0	0.00	0.0	0
Rock	3	140.0	140.0	40000.0	0.0	0.00	0.0	0

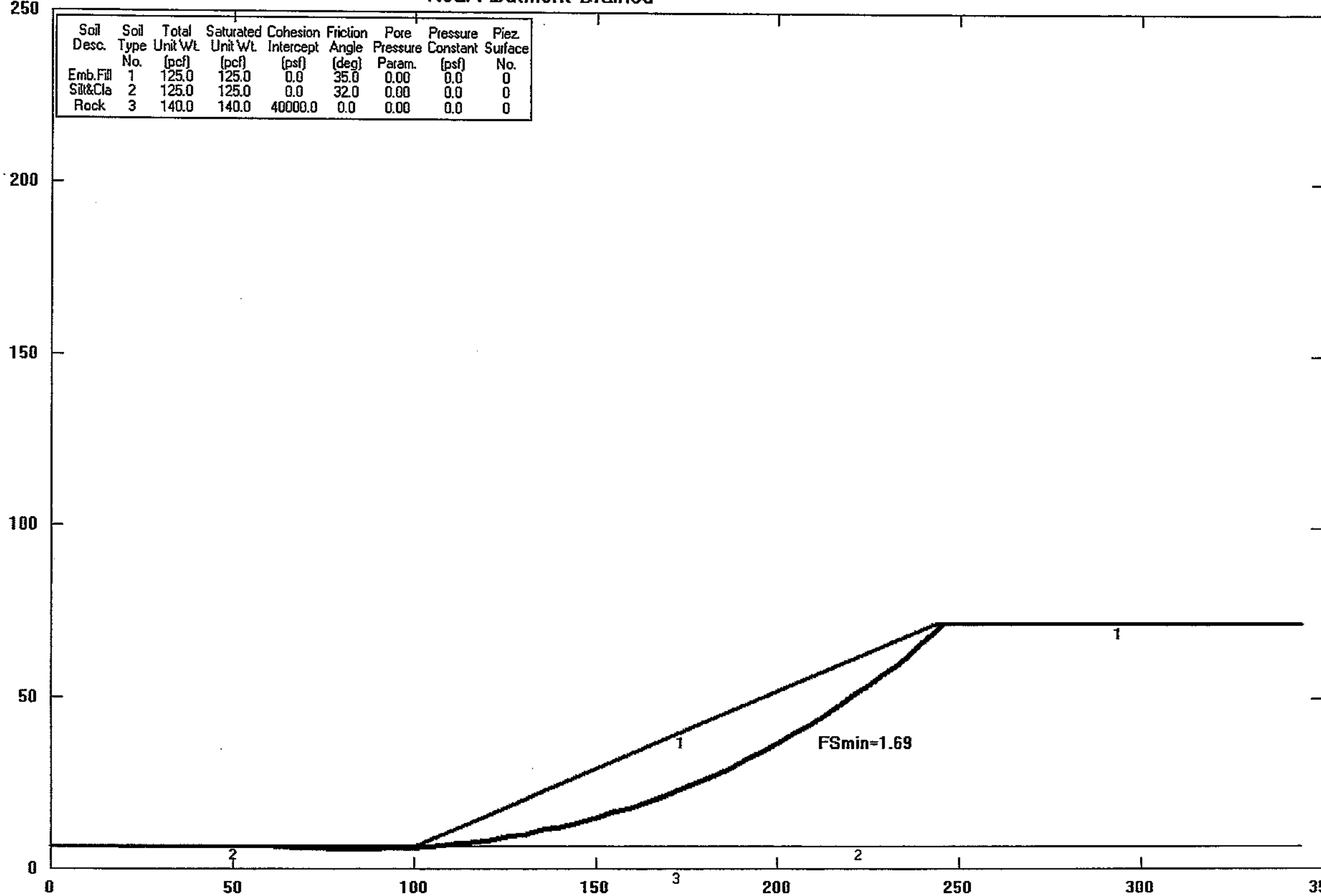


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Portsmouth Bypass-Morris Lane Blue Run
Rear Abutment-Drained

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Emb.Fill	1	125.0	125.0	0.0	35.0	0.00	0.0	0
Silt&Cla	2	125.0	125.0	0.0	32.0	0.00	0.0	0
Rock	3	140.0	140.0	40000.0	0.0	0.00	0.0	0



Safety Factors Are Calculated By The Modified Bishop Method

DLZ