Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
SR 823 Over Portsmouth-Minford Road (SR 139)
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

STRUCTURAL ENGINEERING

DEC 07 2006

WJK ☐ SM ☐ TJK ☐ JEM ☐
JAC ☐ RZ ☐ AW ☐ ☐
MT ☐ DAG ☐ JCR ☐ ☐
AD ☐ SS ☐ JS ☐ FILE ☐

Prepared for:

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DLZ Job No. 0121-3070.03

September 26, 2006

Prepared by:

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

SR 823 OVER PORTSMOUTH-MINFORD ROAD (SR 139)

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

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1.0 INTRODUCTION

This report includes the findings of evaluations of foundations and mechanically stabilized earth (MSE) retaining walls for the structure at the above-referenced location of the project. The findings included in this report pertain to the structure at the intersection of the proposed SR 823 and Portsmouth – Minford Road only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing two structures for the proposed SR 823 over Portsmouth – Minford Road (SR 139). The two structures as planned, are two-span structures using MSE walls to hold back the roadway embankments and contain 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, MSE walls, and the roadway 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

It is understood that the plan location of the bridge structure for proposed SR 823 over Portsmouth – Minford Road (SR 139) has not changed from the approved location, as shown on the Plan and Profile drawing in Appendix I. It is understood that MSE walls will be placed at approximate stations 483+97 and 486+15 to contain the abutments and hold back the roadway embankment for proposed SR 823. Furthermore, it is understood that pile foundations will be used to support the abutments of the proposed structures.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the embankment at stations 483+97 (Rear Abutment) and 486+15 (Forward Abutment) will be approximately 65 and 61 feet, respectively. Those heights are based upon the maximum difference between the proposed grade and the approximate existing grade along the Portsmouth – Minford Road (SR 139).

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 in part of three final and five preliminary structural borings. Borings B-10 through B-12 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over the Portsmouth – Minford Road (SR 139). The borings were drilled between June 20 and 28, 2006. Preliminary structural borings (TR-15 through TR-19) were drilled for a previous design configuration. The preliminary borings were drilled between July 9, 2004 and February 23, 2005. A boring plan is presented in Appendix I. Boring logs for borings TR-15 through TR-19, and B-10 through B-12 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 colluvial soils were also encountered. Lacustrine soils have also been encountered on this project. However, no lacustrine soils were encountered in borings near this proposed structure. Bedrock within the structure area is primarily sandstone of the Logan Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the north and south of the structures roughly above elevation 880. In the area of the structure, the bedrock was covered by a relatively thin soil overburden ranging in thickness between 4.0 and 9.2 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

The results of this investigation indicated that soil conditions at the site were somewhat uniform. In general, the subsoil stratigraphy consisted of shallow surficial materials consisting of topsoil underlain by native cohesive and granular soil deposits and sandstone.

Borings TR-15, TR-16, and B-10 were drilled for the west abutment. Borings TR-18 and TR-19 were drilled for the east abutment, while borings TR-17, B-11, and B-12 were drilled for the piers. Borings TR-16, TR-18, TR-19, and B-10 through B-12 are considered most representative of the soil and bedrock in the area of the proposed structures. However, borings TR-15 and TR-17 are included for informational purposes.

All borings except boring TR-16 encountered surficial material consisting of 2 to 12 inches of topsoil. Boring TR-16 encountered native soil at the ground surface level. All borings encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of medium stiff to very stiff sandy silt (A-4a) and medium stiff to stiff silt (A-4b), while the granular soil deposits consisted mainly of loose to medium dense gravel with sand (A-2-4), loose to very dense sandy silt (A-4a), and medium dense silt (A-4b). The native soil deposits extended to an approximate depth ranging between 4.0 and 9.2 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. The bedrock consisted mainly of medium hard to hard, slightly weathered, slightly to moderately fractured sandstone. The amount of rock recovered in each core run varied between 78 and 100 percent with an average of 95 percent. The rock quality designation (RQD) of the bedrock ranged between 57 and 97 percent with an average of 80 percent indicating good rock.

Unconfined compressive strength of tested cores ranged between 9,709 and 11,829 pounds per square inch. The tested cores correspond to samples at depths between 13.0 feet and 25.0 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1.

Table 1-Unconfined Compressive Strength Results

Boring	Depth (ft)	Unconfined Compressive Strength (psi)
B-10	16.5-17.0	10,393
B-11	13.5-14.0	10,537
B-12	24.5-25.0	9,709
B-12	13.0-13.5	11,829

4.2.3 Groundwater Conditions

Seepage was encountered only in borings TR-15, TR-16, and TR-17 between approximate depths of 6.0 and 7.0 feet. There were no measurable water levels in the borings prior to rock coring. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels were present in all test borings except borings B-11 and TR-15 upon the completion of coring between approximate depths of 1.6 and 28.5 feet.

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 existing bridge will be constructed as described in Sections 1 and 2 of this report. It is understood through comments from ODOT's Office of Structural Engineering that pipe piles will be used to support the abutments. The use of drilled shafts and spread footings has also been considered to support the abutments. In addition, to support the piers, spread footings and drilled shafts bearing on rock have been evaluated. Furthermore, the site is well suited for the use of MSE walls to contain the abutments and hold back the roadway embankment. Recommendations for the piles, drilled shafts, spread footings, and MSE walls are presented in the following sections.

5.1 Bridge Foundation Recommendations

5.1.1 Rear and Forward Abutments

It is understood through comments from the ODOT Office of Structural Engineering (OSE) that pipe piles are to be used to support the abutments. It is understood that the abutments will be supported by steel pipe piles placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel pipe 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 granular fill (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 and MSE wall construction. The allowable pile capacity as per ODOT BDM 202.2.3.2.b may be utilized in this configuration. 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.

Due to the relatively small rigidity of the steel pipe piles compared to drilled shafts, the steel pipe piles are anticipated to provide low lateral resistance to lateral earth pressures that can be induced in high embankment fills such as those at the proposed structure. Therefore, the prebored and socketed steel pipe piles foundation system may be a concern if significant lateral loads are present.

As mentioned above, drilled shafts have also been considered for the support of the abutments. Due to the large amount of embankment fill, it appears that drilled shafts socketed a minimum of 5 feet into competent rock will be well suited for the support of the proposed structural abutments. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

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 or 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. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 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.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Some borings did encounter significant seepage at this site. Water could flow into the drilled shafts 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.

Spread footings bearing in the MSE wall fill may also be considered to support the abutments. As per the Bridge Design Manual (BDM) 204.6.2.1 an allowable bearing capacity of 4 ksf may be used to design the footings. The MSE walls as proposed will be founded on or near bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated negligible.

5.1.2 Piers

Spread footings can be constructed on the rock encountered by the borings to support the piers. Competent bedrock was generally encountered within two to three feet of the soil-rock interface. Spread footings bearing on competent bedrock may be designed using an allowable bearing capacity of 80 ksf (40 tsf).

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 to support the piers. If drilled shafts are used to support the foundation of 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 80 ksf (40 tsf).

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 or 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. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 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.

Precautions should be taken to ensure appropriate drilled shaft construction practices are followed. See Section 5.1.1 for more information.

Table 2, on the following page summarizes the site conditions and foundation recommendations. It should be noted that the bedrock surface varies widely across the project area. The approximate bearing elevations presented below 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 / TR-19	633.0	Pipe Piles	618.3*	Pile Capacity[+]
			Drilled Shafts	618.3*	80 ksf[++]
			Spread Footings	MSE Fill**	4 ksf
	Right / TR-18	631.3	Pipe Piles	619.0*	Pile Capacity[+]
			Drilled Shafts	619.0*	80 ksf[++]
			Spread Footings	MSE Fill**	4 ksf
Pier	Left / B-11	632.7	Spread Footings	624.7***	80 ksf
			Drilled Shafts	619.7*	80 ksf[++]
	Right / B-12	632.5	Spread Footings	624.0***	80 ksf
			Drilled Shafts	619.0*	80 ksf[++]
Forward Abutment	Left / TR-16	631.9	Pipe Piles	617.7*	Pile Capacity[+]
			Drilled Shafts	617.7*	80 ksf[++]
			Spread Footings	MSE Fill**	4 ksf
	Right / B-10	632.6	Pipe Piles	617.6*	Pile Capacity[+]
			Drilled Shafts	617.6*	80 ksf[++]
			Spread Footings	MSE Fill**	4 ksf

* Includes 5-foot socket into competent rock.

** Bearing elevation should be determined by a qualified engineer as the foundation alternative is selected.

*** Assuming competent rock at the soil-rock interface.

[+] Pile capacity should conform to ODOT BDM 202.2.3.2.

[++] End bearing capacity only.

5.2 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls would be used to construct the embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. The MSE wall should be constructed per the recommendations presented in this report and in conformance with the manufacturer's specifications.

5.2.1 MSE Walls: General Information

An MSE retaining wall essentially consists of good quality backfill material with layers of metal or plastic reinforcing that are attached to concrete facing panels. The MSE wall and associated backfill should be constructed in accordance with the specifications of the manufacturer of the MSE wall.

A global stability analysis and bearing capacity analysis were performed for the MSE walls at this bridge location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding and overturning. At the time this report was prepared, it was understood that pipe piles socketed into bedrock would be used at this site to support the bridge abutments. If the foundation type should change, DLZ should be informed so that the analyses may be revised as necessary.

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability analyses are attached. Other external and internal stability analyses are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented in Table 3 below. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. However, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 3-Soil Parameters Used in MSE Wall Stability Analyses

Zone	**Soil Type**	**Unit Weight (pcf)**	**Strength Parameters**			
			Undrained		**Drained**	
			c	**ϕ**	**c'**	**ϕ'**
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear Abutment)	Medium Dense Sandy Silt	120	0	29	0	29
Foundation Soil (Forward Abutment)	Very Soft to Stiff Sandy Silt	120	1000	0	0	29
Foundation Soil (Undercut and Replace)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Wall Evaluations and Recommendations

The MSE wall at the rear abutment (station 483+97) is understood to be approximately 65 feet high. The minimum required embedment depth for this wall is or 3.0 feet assuming that the wall will be bearing on the native soil deposits.

Borings TR-18 and TR-19 were drilled for the rear abutment location. These borings generally encountered cohesionless silt (A-4b) and sandy silt (A-4a) to a depth of 7.3 to 8.7 feet below the ground surface.

Bearing capacity, stability, and global stability calculations have been performed assuming the above parameters. All calculated factors of safety for bearing capacity, sliding, overturning, and global stability were above the minimum recommended values. Therefore, it is recommended that the MSE wall at the rear abutment be built using a minimum embedment of 3.0 feet. Alternatively, soils may be overexcavated to shallow bedrock and replaced with compacted, granular fill to the leveling pad elevation. If soft or highly compressible soils are encountered while excavating for the leveling pad, these soils should be removed and replaced with compacted granular fill. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by the depth of the aggregate base as per ODOT BDM Figure 330. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204. For stability, calculations have indicated that a minimum reinforcement length of 0.8H, or 54.8 feet, is required for stability of the proposed MSE wall at this location.

It should be noted that variations in the topography will be encountered within the proposed footprint of the proposed MSE wall, causing the bedrock elevation to vary. Significant rock excavations may be required to accommodate the reinforcing straps for the MSE wall panels. In areas where bedrock is to be excavated, compacted granular fill is to be placed on bedrock, and a level bench must be cut into the rock to place the fill for stability purposes.

In addition, the foundation leveling pad of the MSE wall at the rear abutment is in close proximity to Long Run Creek, which is running essentially parallel to Portsmouth-Minford Road (SR 139). The approximate elevation of bedrock under the MSE wall is 624 feet, which is near the bottom of the creek. If scour and erosion near the toe of the MSE wall are a concern, then slope protection should be provided with riprap. Alternatively, to mitigate the threat of scour the MSE wall may be founded on bedrock, which is approximately 9 feet below the existing ground surface.

The MSE wall at the forward abutment (station 486+15) is understood to be approximately 61 feet high. The minimum required embedment depth for this wall is 3.0 feet.

Borings B-10 and TR-16 were drilled for the forward abutment. These borings generally encountered cohesive silt (A-4b) and sandy silt (A-4a) to a depth of approximately 9.0 feet below the ground surface.

Initial analyses for the MSE wall bearing on natural soils at this location yielded inadequate factors of safety for undrained bearing capacity, undrained sliding, and undrained global stability. Consequently, it is recommended that the soils beneath the proposed MSE wall be overexcavated to bedrock and replaced with compacted, granular fill to the leveling pad elevation. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by the depth of the aggregate base as per ODOT BDM Figure 330. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204.

It should be anticipated that variations in the topography may be encountered within the footprint of the proposed MSE wall, causing the bedrock elevations to vary significantly. In areas where compacted granular fill is to be placed on bedrock, a level bench must be cut into the rock to place the fill for stability purposes. A minimum reinforcing length of 0.8H, or 51.3 feet, is required for the MSE wall at this location.

Settlement calculations are not necessary for the MSE walls at this site. The MSE walls will bear on compacted granular fill or bedrock resulting in negligible settlement.

Calculations for bearing capacity, overturning, and sliding are attached for both the native soil and compacted granular fill foundations. A drawing showing the results of the global stability analyses is also attached. Tables 4 and 5, on the following pages summarize the MSE retaining wall parameters and results of analyses.

Table 4-MSE Retaining Wall Parameters and Analyses Results (Rear Abutment) *Natural Soil foundation*

Retained Soil (New Embankment)
Unit Weight = 120 pcf
Coefficient of Active Earth Pressure (K_a) = 0.33
(Based on $\Phi = 30^{\circ}$)

Sliding along base of MSE wall
Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37
Use (μ)(0.67) = 0.35 as a maximum value as per AASHTO, BDM, 303.4.1.1

Allowable Bearing Capacity – Undrained Condition
q_{all} = 11,126 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 11,126 psf

Global Stability
Factor of Safety – Undrained Condition = NA (Sandy Silt – Drained Condition)
Factor of Safety – Drained Condition = 1.9
Factor of Safety – Seismic Condition = 1.8

Estimated Settlement of MSE volume
Total settlement = 0 inches
Differential settlement = 0 <1/100

Full Height of MSE Wall = 65.5 feet
Minimum Embedment Depth = 3.0 feet
Minimum Length of Reinforcement for External Stability = 54.8 feet

Table 5-MSE Retaining Wall Parameters and Analyses Results
(Forward Abutment) ***Compacted Granular Fill Foundation on Bedrock***

Retained Soil (New Embankment)
Unit Weight = 120 pcf
Coefficient of Active Earth Pressure (K_a) = 0.33
(Based on $\Phi = 30^{\circ}$)

Sliding along base of MSE wall
Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45
Use (μ)(0.67) = 0.55 as a maximum value as per AASHTO, BDM, 303.4.1.1

Allowable Bearing Capacity – Undrained Condition
q_{all} = 21,873 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 21,873 psf

Global Stability *(Without undercut) [With "remove and replace", on bedrock]*
Factor of Safety – Undrained Condition = (**1.1**) [>1.5]
Factor of Safety – Drained Condition = (1.8) [>1.5]
Factor of Safety – Seismic Condition = (1.7) [>1.3]

Estimated Settlement of MSE volume
Total settlement = 0 inches
Differential settlement = 0 < 1/100

Full Height of MSE Wall = 61.1 feet
Minimum Embedment Depth = 3.0 feet
Minimum Length of Reinforcement for External Stability = 51.3 feet

5.3 Groundwater Considerations

Water seepage was not encountered in any of the borings. Groundwater was not noted prior to adding drill water. Representative final water levels could not be obtained due to the use of water during rock coring. Excavation for the pier foundation is expected to be limited to seven feet or less. Foundation construction on the rock is expected to encounter only minor seepage. Excavations or shafts extending below ground level may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage and water flow that may enter any excavations.

5.4 Anticipated Sequence of Construction

It is understood through comments from ODOT Office of Structural Engineering (OSE) that pipe piles are to be used to support the abutment. It is also understood that MSE walls will be used to retain the roadway embankment and contain the abutments. A brief outline of the anticipated construction sequence is provided here. This outline is general and is in no way inclusive of all of the procedures and precautions required during the construction process. The contractor is ultimately responsible for implementing sound construction practices to build the MSE wall and pile foundations as per plan and in accordance with ODOT specifications.

- Drill a 5-foot deep socket for each pile into competent bedrock.
- Place the pile into socket and grout or cement annular space in the socket. The unsupported length of piling shall be determined by the contractor. Stability of the unsupported pile must be maintained throughout the construction process. If the full length of the pile isn't installed initially, then splices shall be used.
- Although no appreciable consolidation is anticipated at this site, 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 and MSE wall construction.
- Contractor is responsible for controlling the locations of the piles and ensuring that the locations conform to the plan location. This may be accomplished through bracing or other means.
- Place layers of select fill and/or MSE reinforcing straps per ODOT specifications and the MSE wall supplier's recommendations.
  Splice additional lengths of piling onto "in-place" piles as necessary.

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

WKasawneh

Wael Alkasawneh, P.E.
Geotechnical Engineer

sjr

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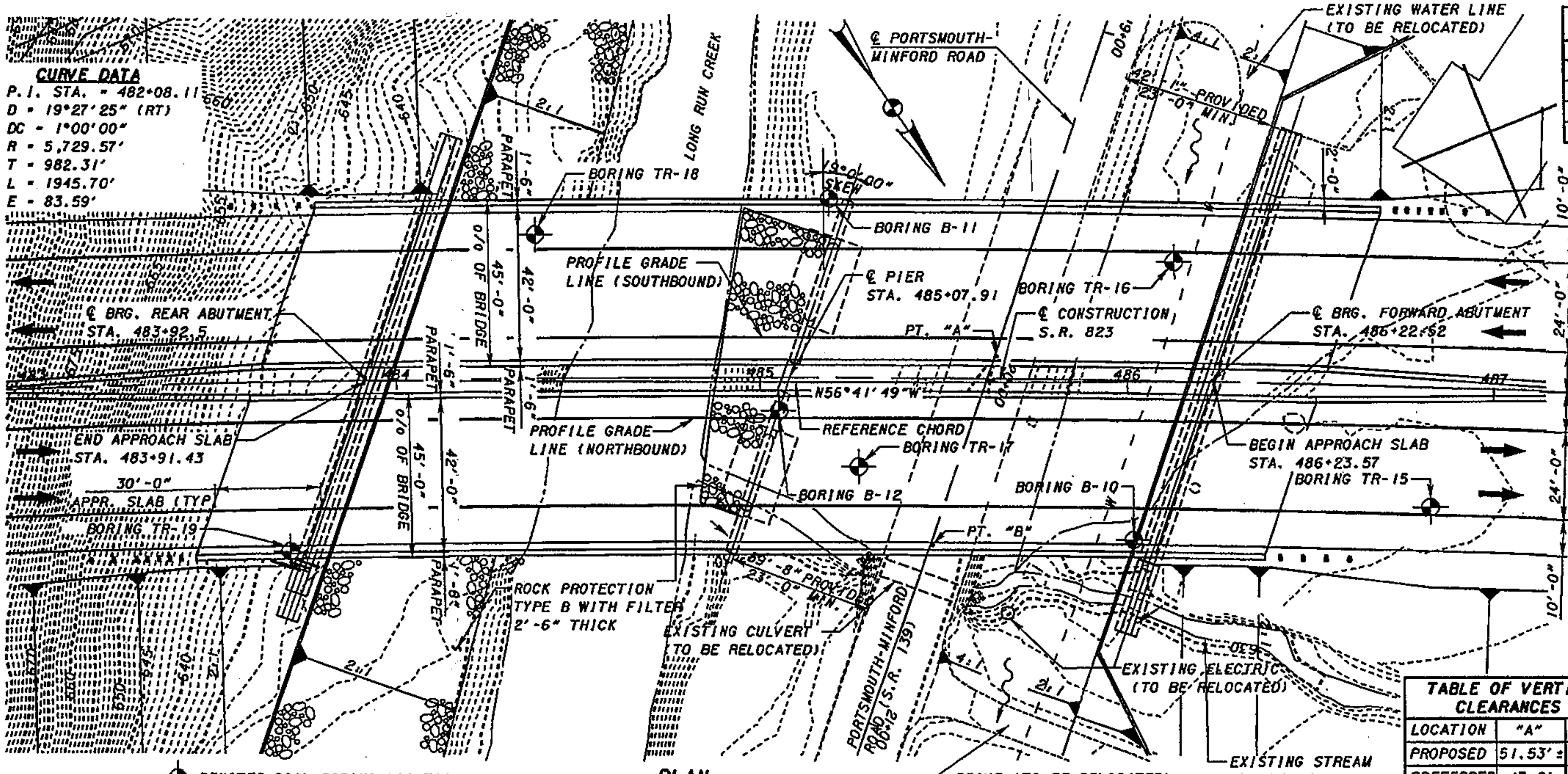
APPENDIX I

Structure Plan and Profile Drawing – 11"x17"
Boring Plan - 11"x17"



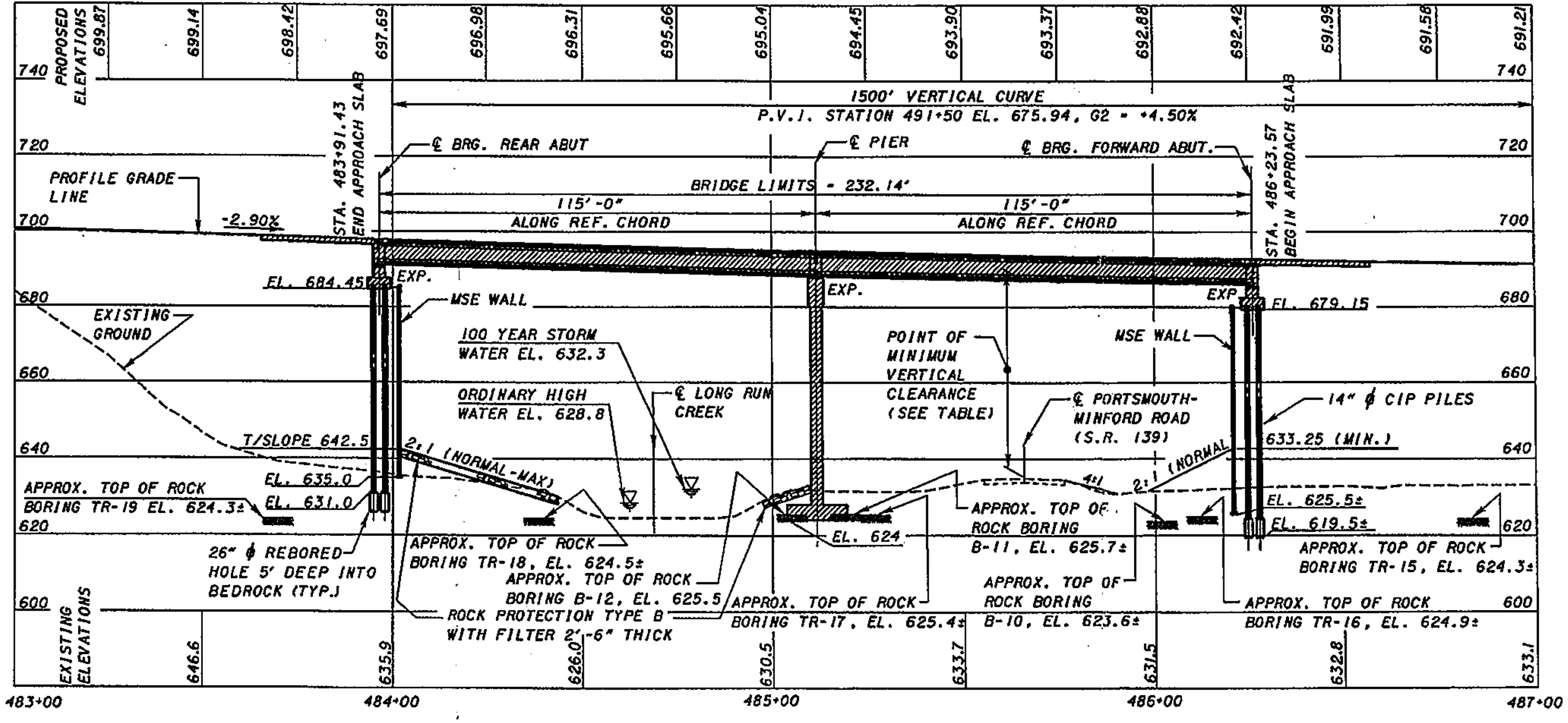
CURVE DATA

P.I. STA. = 482+08.11
D = 19°27'25" (RT)
DC = 1°00'00"
R = 5,729.57'
T = 982.31'
L = 1945.70'
E = 83.59'

⊕ DENOTES SOIL BORING LOCATION

PLAN

TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	51.53'±	50.47±
PREFERRED	17.0'	17.0'



PROFILE ALONG LEFT PROFILE GRADE LINE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	483+39.53	47.00 RT.
REAR ABUT.	483+74.57	47.00 LT.
FWD. ABUT.	486+42.87	47.00 RT.
FWD. ABUT.	486+72.41	47.00 LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-15	486+82.23	36.94' RT.
TR-16	486+12.38	32.33' LT.
TR-17	485+26.88	46.98' RT.
TR-18	484+42.66	46.98' LT.
TR-19	483+60.89	42.99' RT.
B-10	486+01.99	43.79' RT.
B-11	485+18.99	48.57' LT.
B-12	485+04.99	8.98' RT.

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

TRAFFIC DATA

(SR 823)
CURRENT YEAR ADT (2010) = 19,800
DESIGN YEAR ADT (2030) = 26,000
CURRENT YEAR ADTT (2010) = 2,770
DESIGN YEAR ADTT (2030) = 3,640

HYDRAULIC DATA

DRAINAGE AREA = 13.424 sq.mi. = 8591 acres

Q_{50} = 2230 cfs	Q_{100} = 2572 cfs
V_{50} = 6.8 fps	V_{100} = 7.1 fps
EL_{50} = 631.7	EL_{100} = 632.1

OHWM: EL. 628.8
AREA BELOW OHWM: 0.13 ACRES
TEMP. FILL BELOW OHWM: 835 CY

NOTES:

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

FOUNDATION DATA:

ALL NEW PILES SHALL BE 14" ϕ CIP PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE. SPREAD FOOTINGS SHALL HAVE AN ALLOWABLE BEARING CAPACITY OF 15 TSF.

PROPOSED STRUCTURE

TYPE: 2 SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS, T-TYPE PIERS, AND MSE WALL SUPPORTED EMBANKMENT.

SPANS: 115'-0", 115'-0" C/C BEARINGS (MEASURED ALONG REF. CHORD)

ROADWAY: 2-42'-0" TOE TO TOE OF PARAPETS.

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

SKEW: 19°00'00" (LF) WITH RESPECT TO REF. CHORD.

SUPER ELEVATION: 0.036 FT/FT.

ALIGNMENT: 1°00'00" CURVE TO THE RIGHT.

WEARING SURFACE: 1" MONOLITHIC CONCRETE.

APPROACH SLABS: AS-1-81 (30'-0" LONG).

LATITUDE: 38° 51' 30"

LONGITUDE: 82° 53' 00"

DESIGN AGENCY
Systems
5411 POINTER WOODS, SUITE 2A
DUBLIN, OHIO 43017

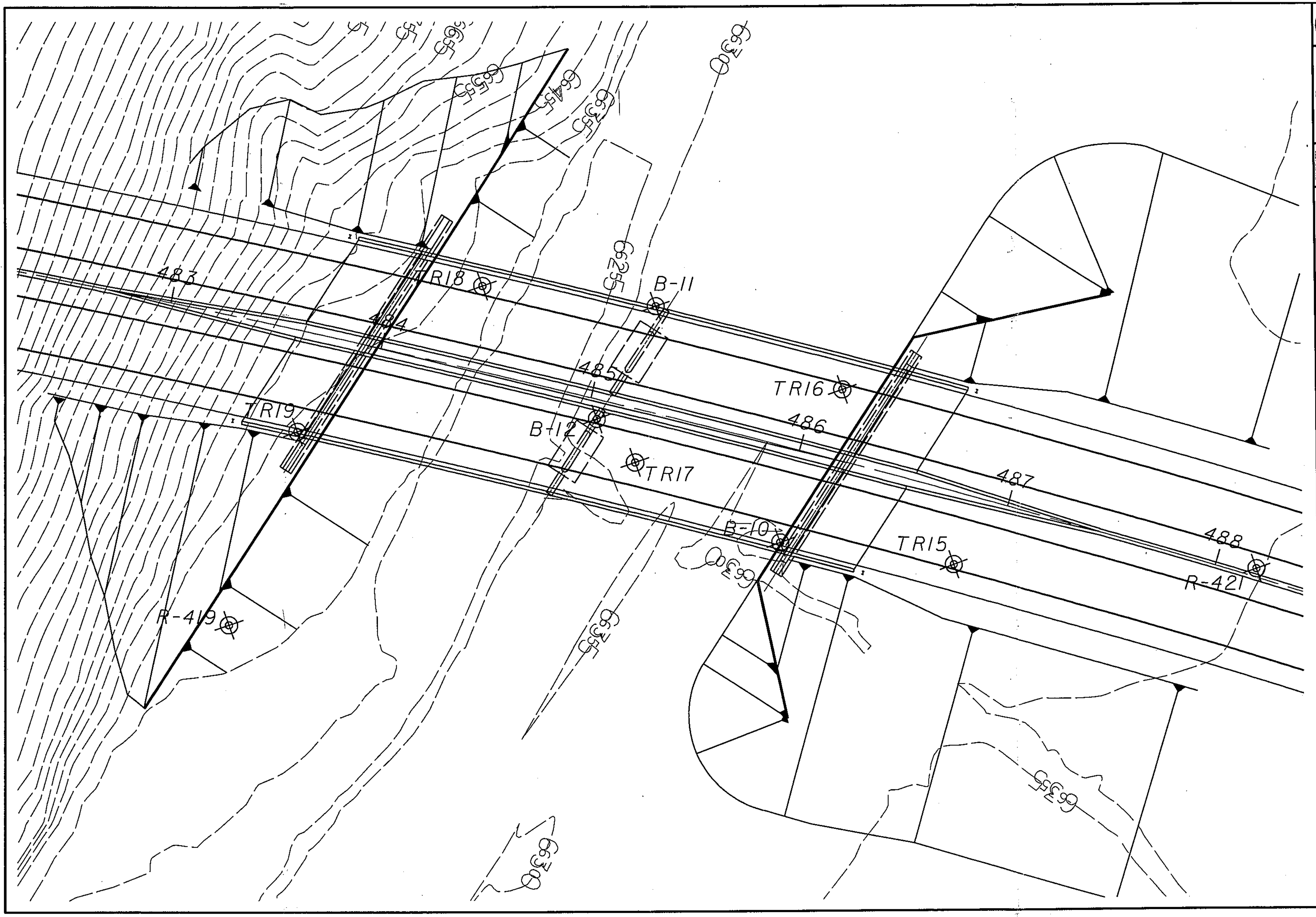
DESIGNED	DRAWN	REVIEWED	DATE
PJP	AWB	JRC	4/13/06
CHECKED MSL	REVISED	STRUCTURE FILE NUMBER	

SCIOTO COUNTY
STA. 483+91.43
STA. 486+23.57

SITE PLAN

BRIDGE NO. SCI-0814-R
S.R. 823 OVER PORTSMOUTH-MINFORD RD. (S.R. 139)

SCI-823-0.00
PID 19415



BORING PLAN

SR 823 OVER PORTSMOUTH-MINFORD ROAD

SCI-823

HORIZONTAL SCALE IN FEET
0 10 20 40

CALCULATED

CHECKED

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.

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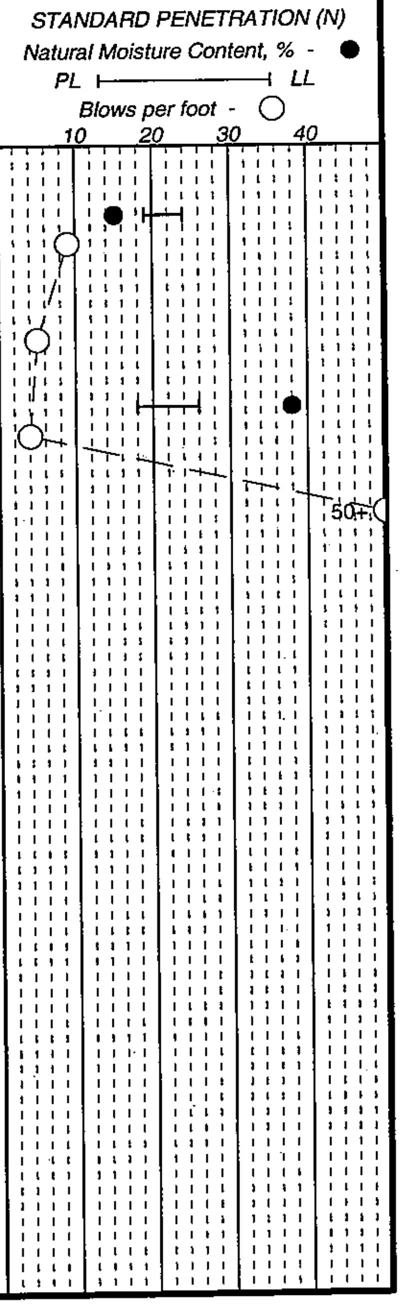
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-10 | Location: Sta. 486+01.5, 43.8' RT of SR 823 CL | Date Drilled: 06/28/06

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: none (prior to coring)
6.0' (inside hollowstem augers, 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	632.6												
0.3	632.3						Topsoil - 3"						
		4 5 4	14	1		1.5	Stiff brown SILT (A-4b), little clay, trace to little fine sand; damp.	0	0	--	10	74	16
		3 3 2	17	2		--							
		2 2 2	13	3		2.0	@ 6.0'-7.5', soft, wet.	0	0	--	11	74	15
8.5	624.1	10 50/2	6	4			Severely weathered gray SANDSTONE.						
9.5	623.1	Core 120"	Rec 116"	RQD 87%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured. @ 16.5', qu = 10,393 psi.						
19.5	613.1						Bottom of Boring - 19.5'						



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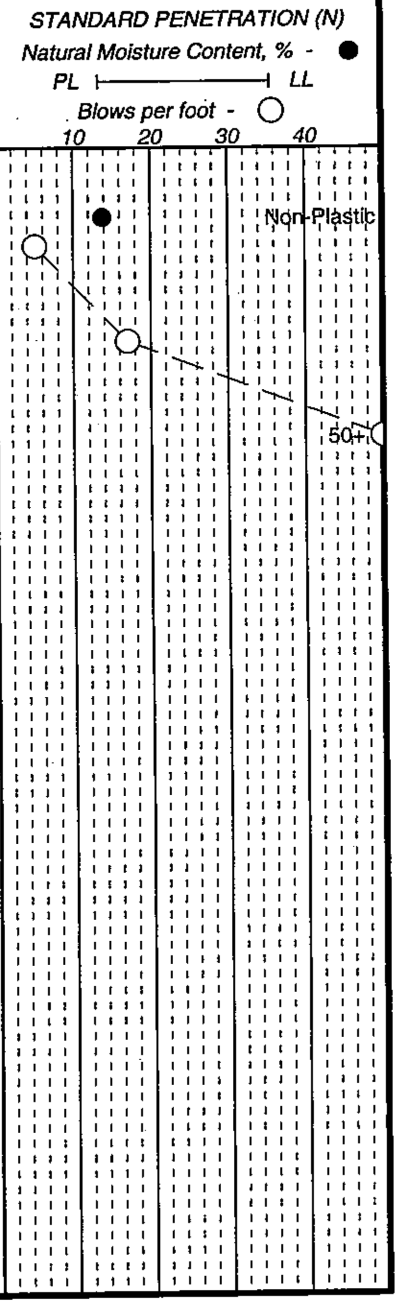
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring B-11
Location: Sta. 485+19.1, 48.6 ft. LT of SR 823 CL
Date Drilled: 6/20/06

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: not reported

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
0.2	632.7 632.5						Topsoil - 2"						
		3 3 2	13	1			Loose brown SANDY SILT (A-4a), trace clay; damp.	0	1	--	17	82	
4.0	628.7	4 8 9	15	2			Medium dense gray SANDY SILT (A-4a) ; damp. (Decomposed Rock)						
		7 15 50/3	15	3			@ 8.5', auger refusal.						
8.5	624.2	Core 120"	Rec 93"	RQD 57%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured. @ 8.5' to 10.0', highly fractured to broken. @ 13.5', qu = 10,537 psi.						
		Core 120"	Rec 116"	RQD 97%	R-2								
28.5	604.2						Bottom of Boring - 28.5'						



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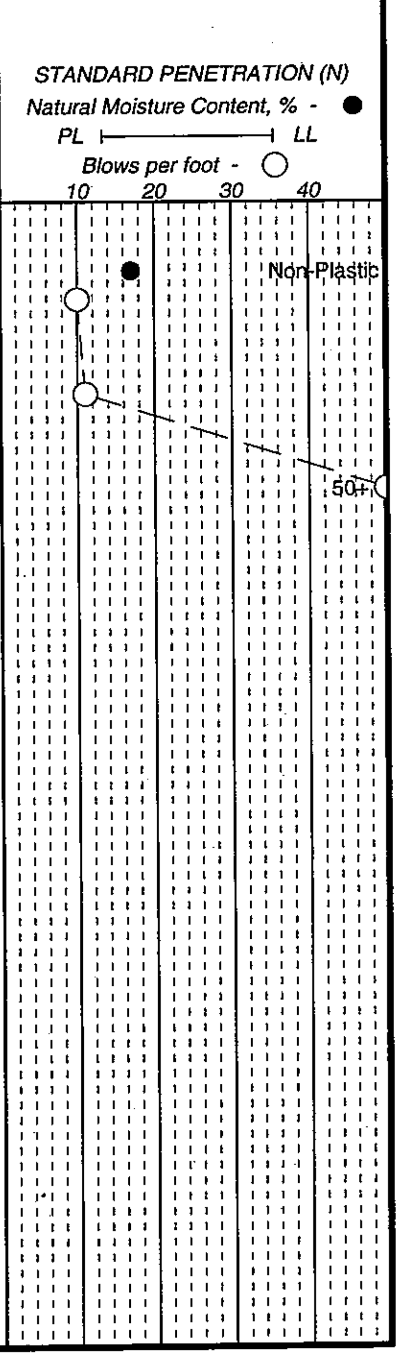
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring B-12
Location: Sta. 485+04.7, 9.0 ft. RT of SR 823 CL
Date Drilled: 6/20/06

WATER OBSERVATIONS:
Water seepage at: none
Water level at completion: none (prior to coring)
4.0' (inside hollowstem augers 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	632.5												
0.3	632.2						Topsoil - 3"						
		3 4 6	15	1			Loose to medium dense brown SANDY SILT (A-4a), trace clay; contains sandstone fragments; damp.	0	0	--	8	92	
4.0	628.5	10 7 4	9	2			Loose to medium dense reddish brown GRAVEL WITH SAND AND SILT (A-2-4); contains sandstone fragments; damp.						
5													
		4 5 50/3	15	3									
8.5	624.0						Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured.						
10													
		Core 120"	Rec 97"	RQD 67%	R-1		@ 13.0', qu = 11,829 psi.						
15													
20													
		Core 120"	Rec 120"	RQD 85%	R-2		@ 24.5', qu = 9,709 psi.						
25													
28.5	604.0						Bottom of Boring - 28.5'						
30													



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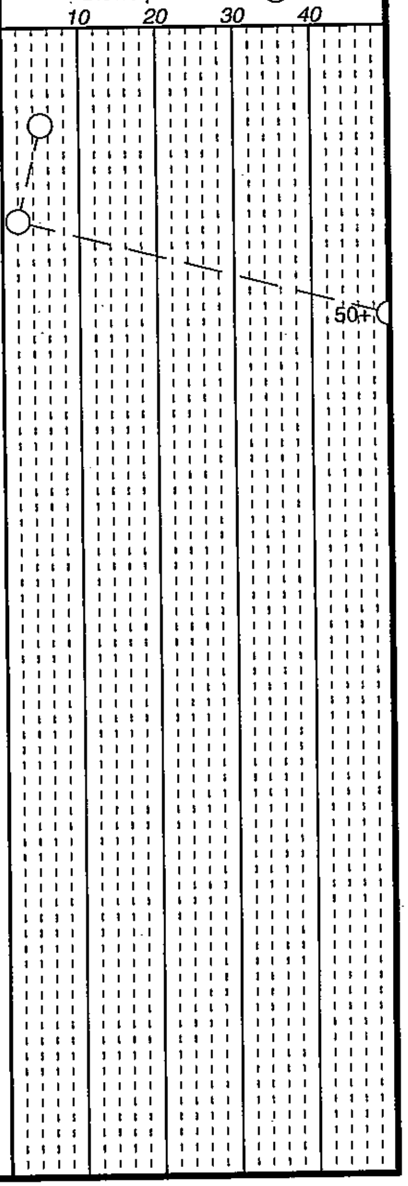
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-15
Location: Sta. 486+83.3, 32.9 ft. RT of SR 823 CL
Date Drilled: 7/9/2004

WATER OBSERVATIONS: Water seepage at: 6.0'
Water level at completion: None

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
	631.3												
0.2	631.1						Topsoil - 2"						
		4 2 3	14	1		1.0	Stiff to very stiff brown SANDY SILT (A-4a), trace gravel; moist.						
		2 1 1	13	2		<0.25	@ 3.5'-5.0', very soft.						
5													
7.0	624.3	3 20 50/2	13	3A 3B		3.25	Severely weathered brownish-gray SILTSTONE fragments.						
8.0	623.3						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured.						
10													
		Core 120"	Rec 99"	RQD 70%	R-1		@ 8.0'-9.0', probable core loss.						
15													
18.0	613.3						Bottom of Boring - 18.0'						
20													
25													
30													

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



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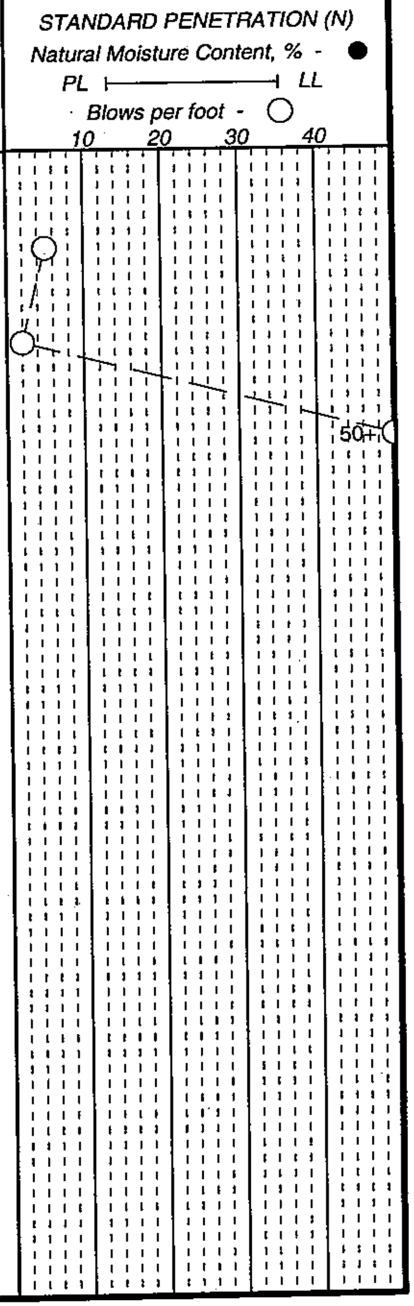
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-16
Location: Sta. 486+12.4, 32.3 ft. LT of SR 823 CL
Date Drilled: 7/9/04

WATER OBSERVATIONS: Water seepage at: 6.0'
Water level at completion: 6.5'

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
	631.9												
0.2	631.7						Topsoil - 2"						
		2 3 2	16	1		1.0	Medium stiff brown SANDY SILT (A-4a); moist.						
		1 1 1	15	2		0.75							
5													
		4 10 50/5	12	3		--	@ 6.0' to 7.4', contains rock fragments.						
8.5	623.4						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, argillaceous, massively bedded, slightly fractured.						
10													
		Core 120"	Rec 118"	RQD 85%	R-1								
15													
							@ 17.0', contains few argillaceous laminations.						
18.5	613.4						Bottom of Boring - 18.5'						
20													
25													
30													



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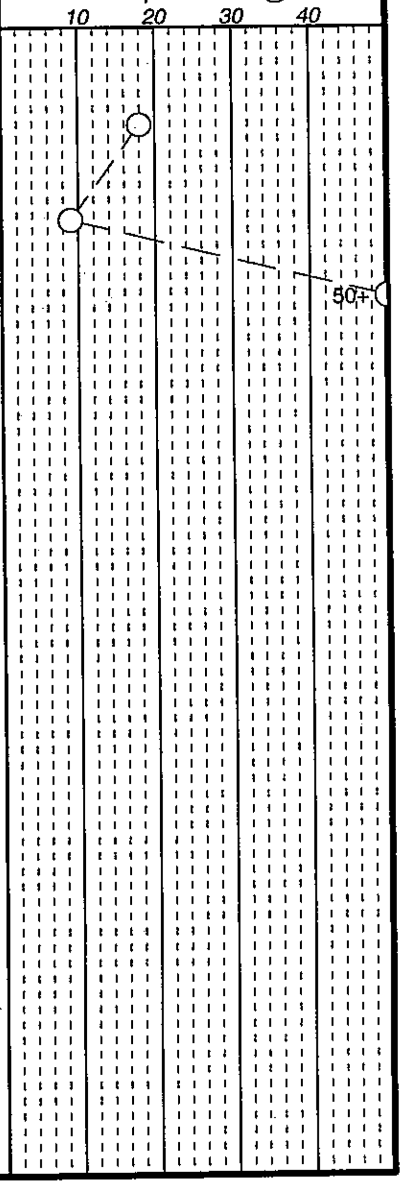
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-17
Location: Sta. 485+26.9, 24.3 ft. RT of SR 823 CL
Date Drilled: 2/23/2005

WATER OBSERVATIONS: Water seepage at: 6.3'-7.0'
Water level at completion: 1.6' (inside hollowstem augers, 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	631.7												
0.4	631.3						Topsoil - 5"						
		6 8 10	18	1			Medium dense brown SILT (A-4b), little fine to coarse sand, trace clay; damp.						
3.0	628.7	6 4 5	18	2			Loose brown GRAVEL WITH SAND AND SILT (A-2-4); damp.						
5.5	626.2						Very dense brown SANDY SILT (A-4a); wet.						
6.3	625.4	3 50/5	11	3A 3B			Severely weathered gray SANDSTONE.						
7.0	624.7	Core 120"	Rec 120"	RQD 83%	R-1		Medium hard brown and gray SANDSTONE; fine grained, moderately weathered, slightly micaceous, slightly fractured. @ 7.3'-7.4', very soft, highly weathered. @ 8.5', irregular fracture. @ 8.7', gray. @ 16.0', 1" soft, weathered zone.						
17.0	614.7	Core 120"	Rec 120"	RQD 97%	R-2		Hard brown and gray SANDSTONE; fine grained, slightly weathered, slightly micaceous, slightly fractured. @ 22.8'-23.0', very soft, highly weathered siltstone seam. @ 23.0'-23.2', siltstone seam.						
27.0	604.7						Bottom of Boring - 27.0'						

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



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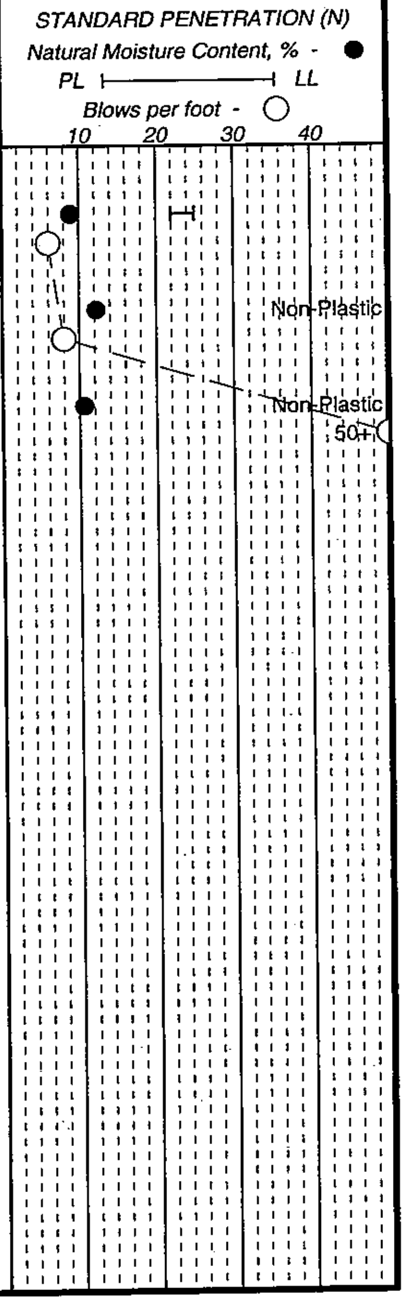
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-18 | Location: Sta. 484+38.6, 39.0 ft. LT of SR 823 CL | Date Drilled: 8/17/04

WATER OBSERVATIONS: Water seepage at: None
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 Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	631.3						Topsoil - 12"						
1.0	630.3	2 3 3	18	1			Medium stiff brown SILT (A-4b), little clay, little fine to coarse sand, little gravel; contains roots; dry to damp.	13	7	--	9	58	13
3.0	628.3	3 4 4	18	2			Loose brown SANDY SILT (A-4a), little clay, trace to little gravel; damp.	0	3	--	40	45	12
		6 7 50/3	12	3				11	20	--	28	31	10
7.3	624.0	Core 84"	Rec 84"	RQD 88%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, slightly to moderately fractured. @ 7.3'-7.6', broken. @ 7.3'-7.8',8.0',8.6'-8.8', brown, rust-stained fractures. @ 7.3'-7.8', vertical fracture.						
		Core 72"	Rec 71"	RQD 94%	R-2								
20.3	611.0						Bottom of Boring - 20.3'						



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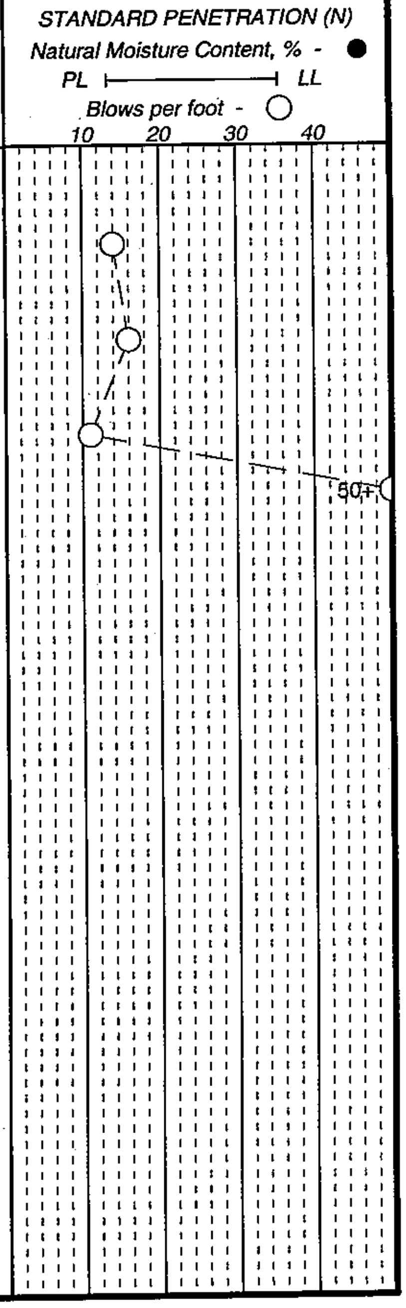
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-19
Location: Sta. 483+69.8, 46.5 ft. RT of SR 823 CL
Date Drilled: 8/16/04 to 8/17/04

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 16.3' (includes drilling water)

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
0	633.0						Topsoil - 12"						
1.0	632.0	3 7 7	18	1			Medium dense brown SANDY SILT (A-4a), trace gravel, trace clay; contains sandstone fragments; damp.						
		4 7 9	18	2									
5													
		4 5 6	18	3									
8.7	624.3	50/2	2	4			Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured. @ 9.2'-9.4', decomposed. @ 8.8'-9.0', brown. @ 13.1'-13.3', vertical fracture.						
10		Core 30"	Rec 30"	RQD 57%	R-1								
15		Core 108"	Rec 108"	RQD 70%	R-2		@ 13.9'-14.0', vertical fracture. @ 15.5', unfractured to slightly fractured. @ 14.7'-15.5', broken zone. @ 15.4'-15.5', clay filled fracture.						
20.2	612.8						Bottom of Boring - 20.2'						
25													
30													



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: 9/14/2006

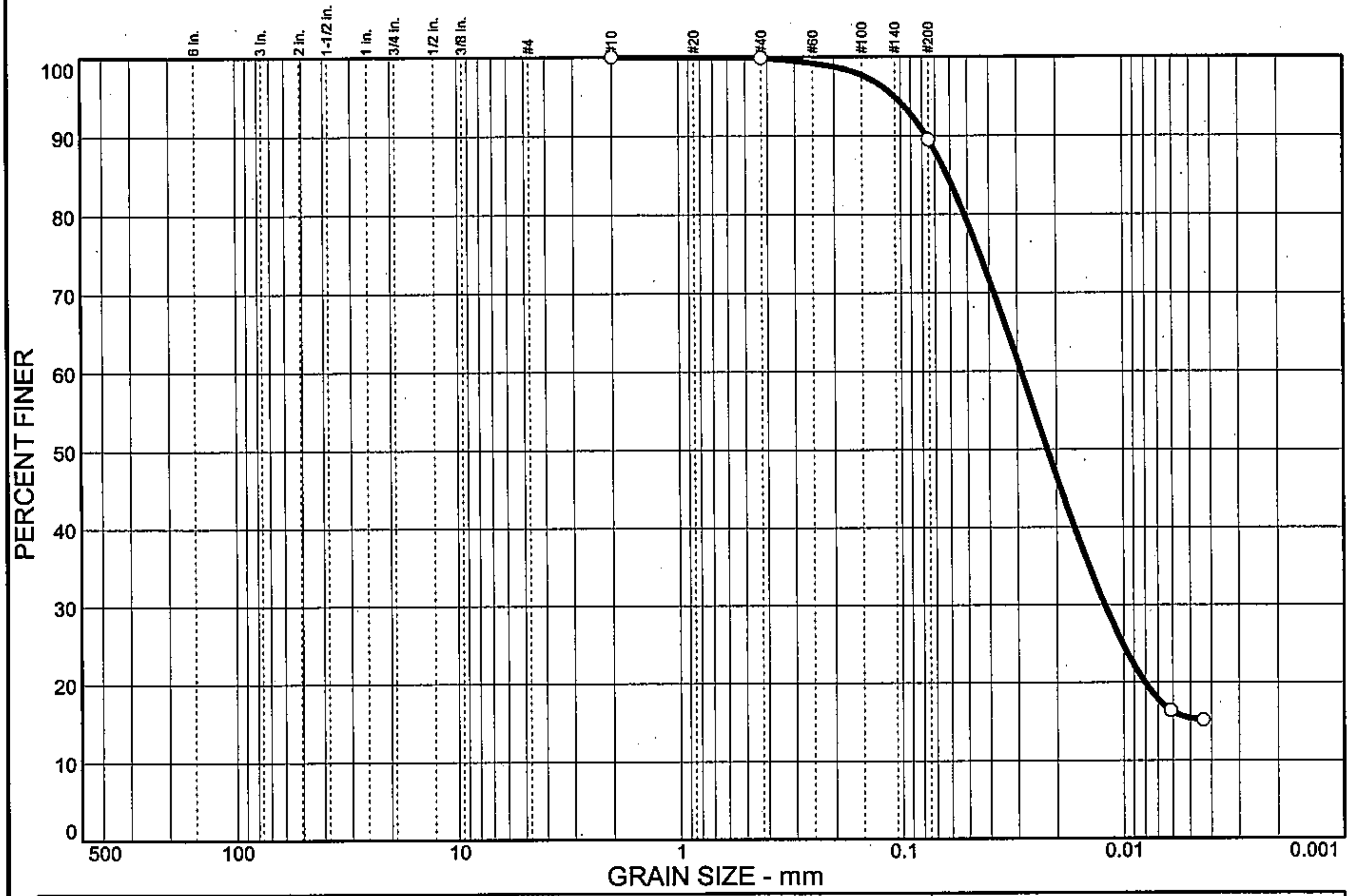
Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-10	2	16.5-17.0	1.861	1.865	1.865	1.864	4.665	4.656	4.652	4.658	2.499	0.0073493	523.59	157.07	28,240	**10,393**
			1.861	1.864	1.866											
B-11	1	13.5-14.0	1.555	1.866	1.869	1.815	4.468	4.468	4.471	4.469	2.462	0.0066869	491.85	162.16	28,630	**10,537**
			1.863	1.868	1.868											
B-12	2	24.5-25.0	1.865	1.855	1.861	1.862	4.497	4.505	4.487	4.496	2.415	0.0070808	513.72	159.95	26,380	**9,709**
			1.865	1.861	1.864											
B-12	1	13.0-13.5	1.867	1.867	1.869	1.867	4.615	4.615	4.612	4.614	2.472	0.0073039	490.11	147.94	32,140	**11,829**
			1.866	1.865	1.866											

DLZ

Engineers * Architects * Scientists

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.1	10.4	74.1	15.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	89.5		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 19 LL= 24 PI= 5

Coefficients

D_{85}= 0.0618 D_{60}= 0.0288 D_{50}= 0.0220
D_{30}= 0.0122 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(3)

Remarks

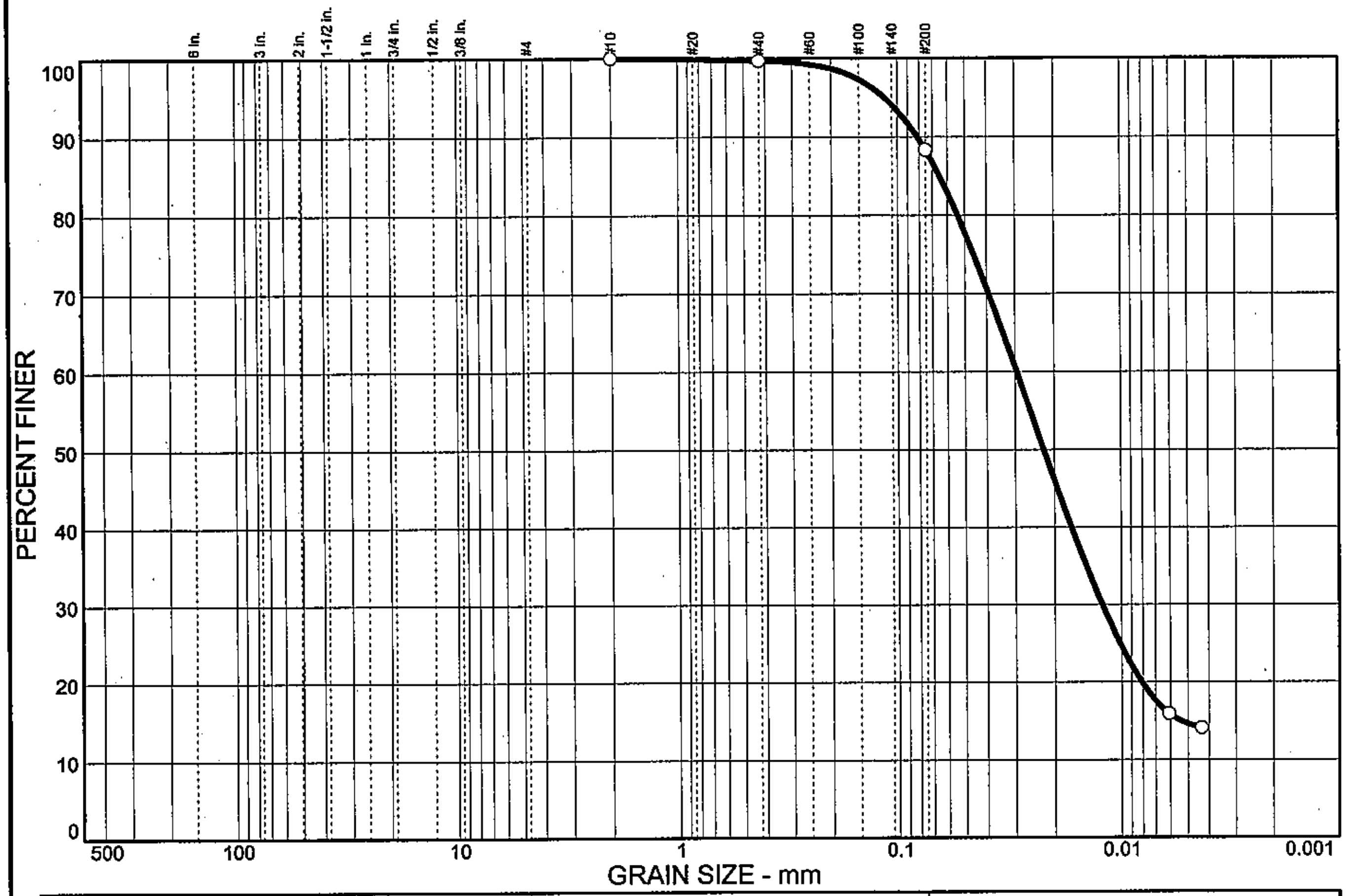
Moisture Content= 15.1%

Sample No.: 1 **Source of Sample:** B-10 **Date:** 7/21/06
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.3	11.4	73.7	14.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	88.3		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 18 LL= 26 PI= 8

Coefficients

D_{85}= 0.0650 D_{60}= 0.0295 D_{50}= 0.0223
D_{30}= 0.0122 D_{15}= 0.0054 D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

Remarks

Moisture Content= 38.0%

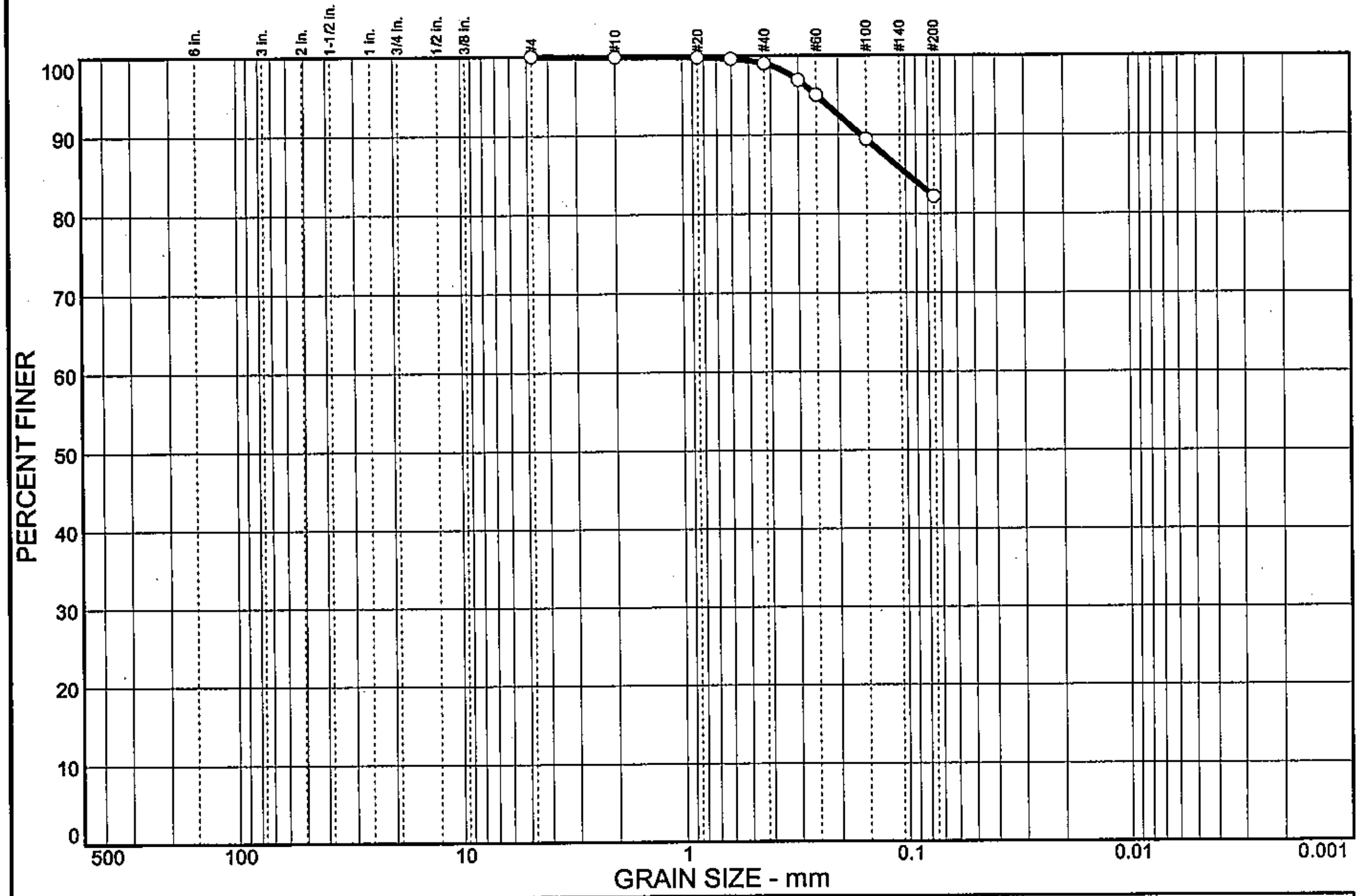
Sample No.: 3 **Source of Sample:** B-10 **Date:** 7/21/06
Location: **Elev./Depth:** 6.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.1	0.9	16.8	82.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.8		
#30	99.6		
#40	99.0		
#50	96.9		
#60	95.0		
#100	89.4		
#200	82.2		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.0986 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

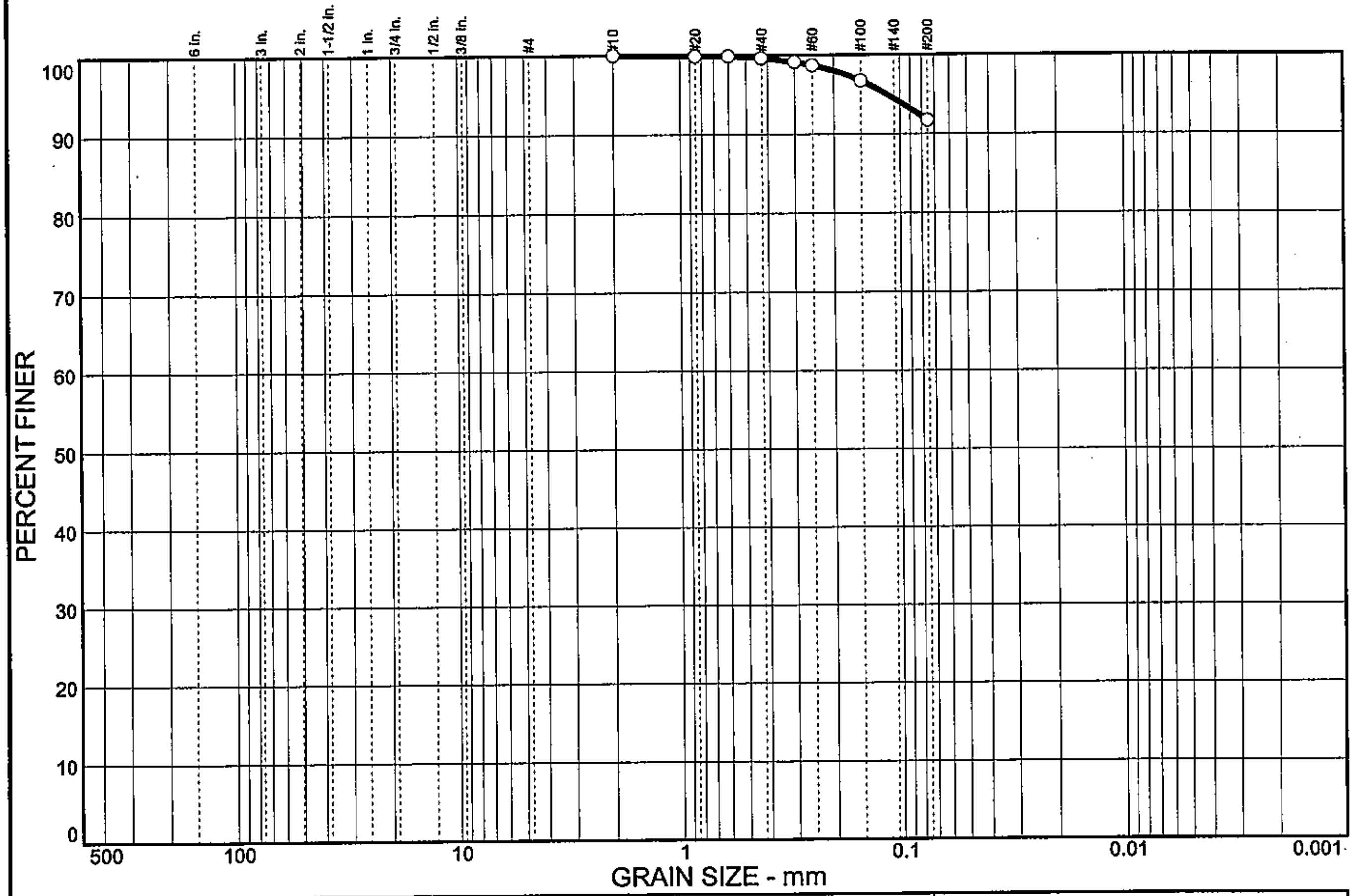
Moisture Content = 13.8%

Sample No.: 1 **Source of Sample:** B-11 **Date:** 07/12/06
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.4	7.9	91.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#30	99.9		
#40	99.6		
#50	99.1		
#60	98.7		
#100	96.7		
#200	91.7		

* (no specification provided)

Soil Description

Silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

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

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content = 17.0%

Sample No.: 1 **Source of Sample:** B-12 **Date:** 07/12/06
Location: **Elev./Depth:** 1.0

DLZ

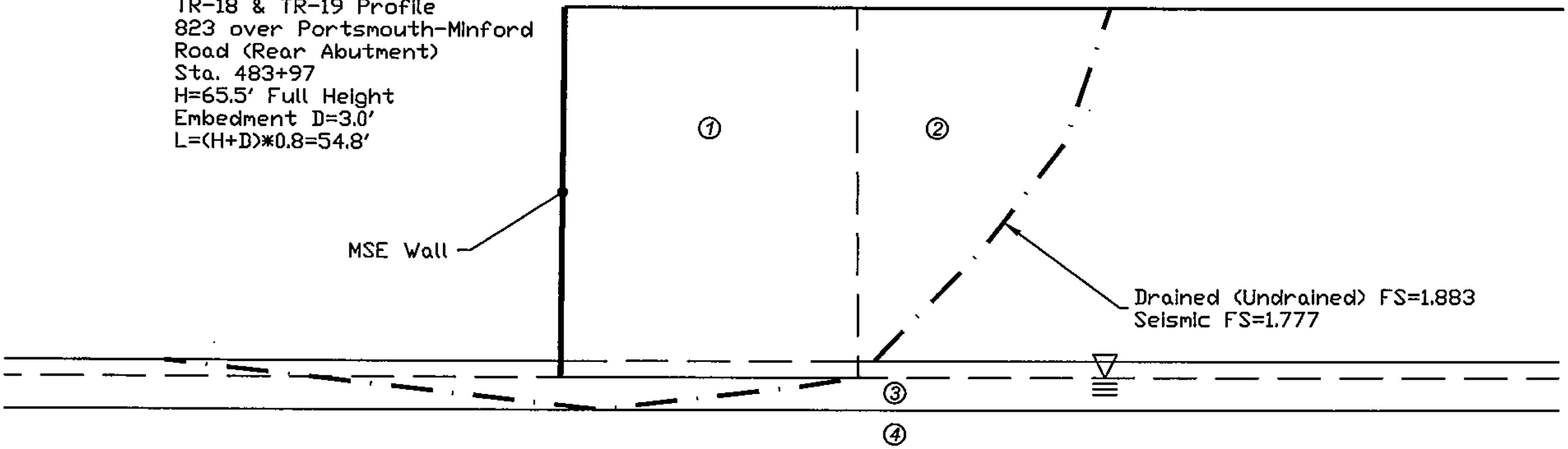
Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

APPENDIX IV

MSE Wall Stability Analysis Results
MSE Wall Bearing Capacity and Stability Calculations
Drilled Shaft – End Bearing and Side Resistance Calculations

Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Medium Dense	Silt/Sandy Silt	0	29	0	29	120
Material 4	Hard	Sandstone	10000	45	10000	45	145

MSE Stability Analysis
TR-18 & TR-19 Profile
823 over Portsmouth-Minford
Road (Rear Abutment)
Sta. 483+97
H=65.5' Full Height
Embedment D=3.0'
L=(H+D)*0.8=54.8'



Drained (Undrained) FS=1.883
Seismic FS=1.777

823 OVER PORTSMOUTH - MINFORD ROAD
REAR ABUTMENT

MSE STABILITY ANALYSIS

SCI-823-0. 00

PROJECT NO. 0121-3070. 03	CALC: SJR	DATE 9/14/06

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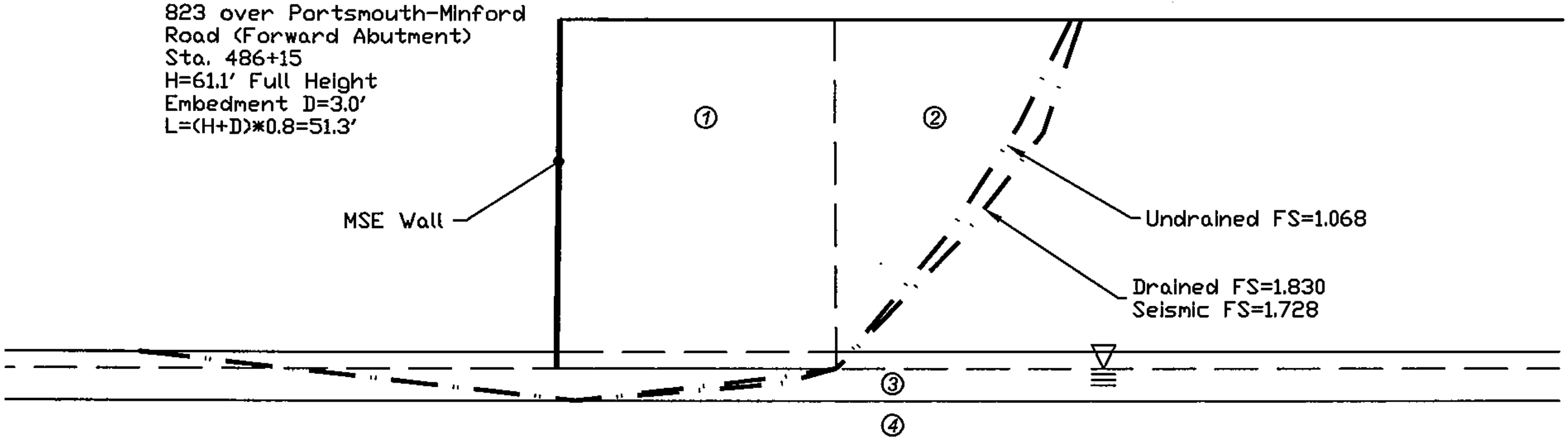
			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	ϕ (deg)	C' (psf)	ϕ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Medium Dense	Silt/Sandy Silt	1000	0	0	29	120
Material 4	Hard	Sandstone	10000	45	10000	45	145

MSE Stability Analysis
TR-16 & B-10 Profile
823 over Portsmouth-Minford
Road (Forward Abutment)
Sta. 486+15
H=61.1' Full Height
Embedment D=3.0'
L=(H+D)*0.8=51.3'



823 OVER PORTSMOUTH - MINFORD ROAD
FORWARD ABUTMENT

MSE STABILITY ANALYSIS

SCI-823-0. 00

PROJECT NO. 0121-3070. 03 | CALC: SJR | DATE 9/14/06

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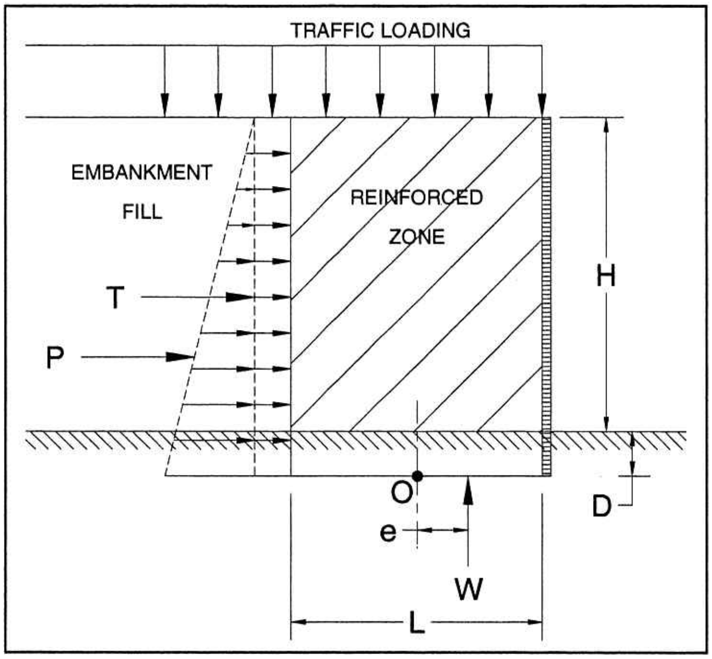
DLZ

SUBJECT

Client	TranSystems	JOB NUMBER	0121-3070.03		
Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	1	OF	6
Item	Bearing Capacity - MSE Wall	COMP. BY	SJR	DATE	9/13/06
SR 823 over Portsmouth Minford Road		CHECKED BY	DAA	DATE	9/14/06
Rear Abutment					

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L-2e}$$

σ_v = 10,339 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT}=cN_c+\sigma'_D N_q+\frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 27,816 psf

$$q_{ALL}=\frac{q_{ULT}}{FS}$$

q_{ALL} = 11,126 psf

Factor of Safety = **2.69** **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT}=c'N_c+\sigma'_D N_q+\frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 27,816 psf

$$q_{ALL}=\frac{q_{ULT}}{FS}$$

q_{ALL} = 11,126 psf

Factor of Safety = **2.69** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	29	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	54.8	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	68.5	ft	
H	=	65.5	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	22.833	ft	Moment arm
Γ Wt	=	34.25	ft	Moment arm
B'	=	44.84	ft	
γ'	=	57.6	pcf	
W_t		13,152	lb/ft of wall	Weight from traffic
W_{mse}	=	450,456	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	27.86	N_c	27.86
N_q	16.44	N_q	16.44
N_γ	19.34	N_γ	19.34

Eccentricity of Resultant Force

e = 4.98 ft

Kern

e < L/6 = 9.13 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	2 OF 6
Item	MSE Wall Stability	COMP. BY	SJR DATE 09/13/06
SR 823 over Portsmouth - Minford Road		CHECKED BY	DAA DATE 9/14/06
Rear Abutment			

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=65.5'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

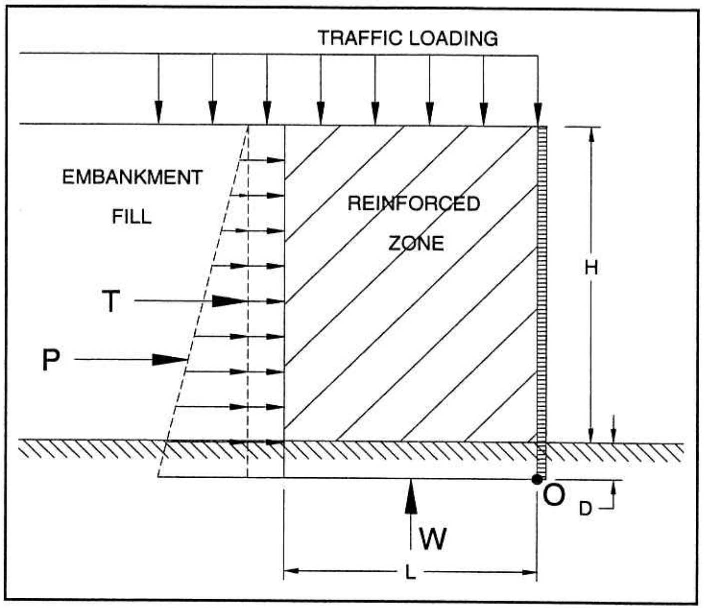
Wall Properties

$H+D$ = 68.5 feet
γ_{mse} = 120 pcf
L = 54.8 feet
L factor = 0.80
ϕ = 30 deg

Foundational Soil Properties

c = 0 psf Cohesion
ϕ' = 29 deg Friction angle
ω_T = 240 psf Traffic loading
Length factor-range (0.7 - 1.0)
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



Thrust: $$P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 98,332 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 157,660 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$

Calculated FS = **1.60**

Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

- Summation of Moments about point "O" (base of wall).
- Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 12,342,494 lb-ft

$$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 2,307,179 lb-ft

$$\Sigma M_{overturnig} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturnin\ g}}$$

Calculated FS = **5.35**

Required FS = 2.00

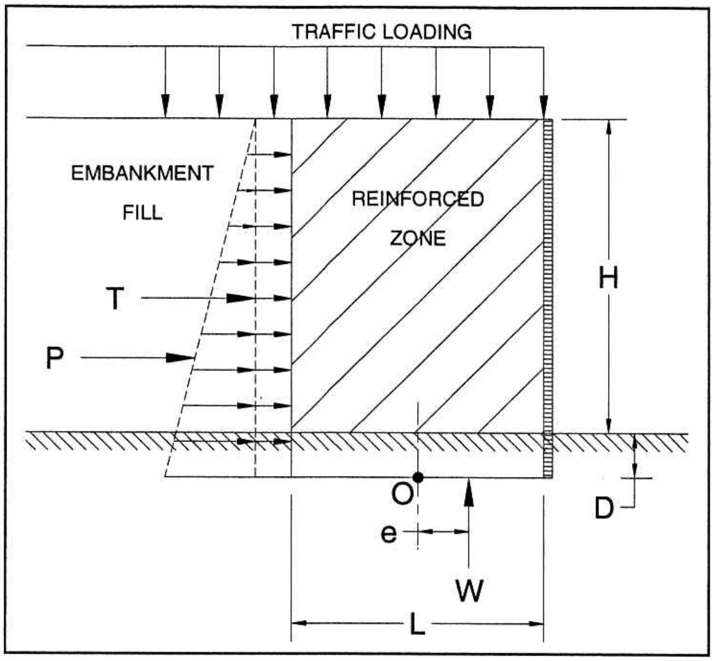
Resistance Against Overturning is **OK**

DLZ

SUBJECT				
Client	TranSystems	JOB NUMBER	0121-3070.03	
Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	3	OF 6
Item	Bearing Capacity - MSE Wall	COMP. BY	SJR	DATE 9/13/06
SR 823 over Portsmouth Minford Road		CHECKED BY	DAA	DATE 9/14/06
Forward Abutment				

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 9,698 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 5,313 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 2,125 psf

Factor of Safety = **0.55** **No Good**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 26,201 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 10,480 psf

Factor of Safety = **2.70** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1000	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	51.28	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	64.1	ft	
H	=	61.1	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	21.367	ft	Moment arm
Γ Wt	=	32.05	ft	Moment arm
B'	=	41.94	ft	
γ'	=	57.6	pcf	
W_t		12,307	lb/ft of wall	Weight from traffic
W_{mse}	=	394,446	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 4.67 ft

Kern

e < L/6 = 8.55 ft

DLZ

SUBJECT

Client TranSystems ODOT D-9
Project SCI 823-0.00 Portsmouth Bypass
Item MSE Wall Stability
SR 823 over Portsmouth - Minford Road
Forward Abutment

JOB NUMBER 0121-3070.03
SHEET NO. 4 OF 6
COMP. BY SJR DATE 09/13/06
CHECKED BY DAA DATE 9/14/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=65.5'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

Wall Properties

$H+D$ = 64.1 feet
γ_{mse} = 120 pcf
L = 51.28 feet
L factor = 0.80 Length factor-range (0.7 - 1.0)
ϕ = 30 deg Friction Angle of Embankment Fill

Foundational Soil Properties

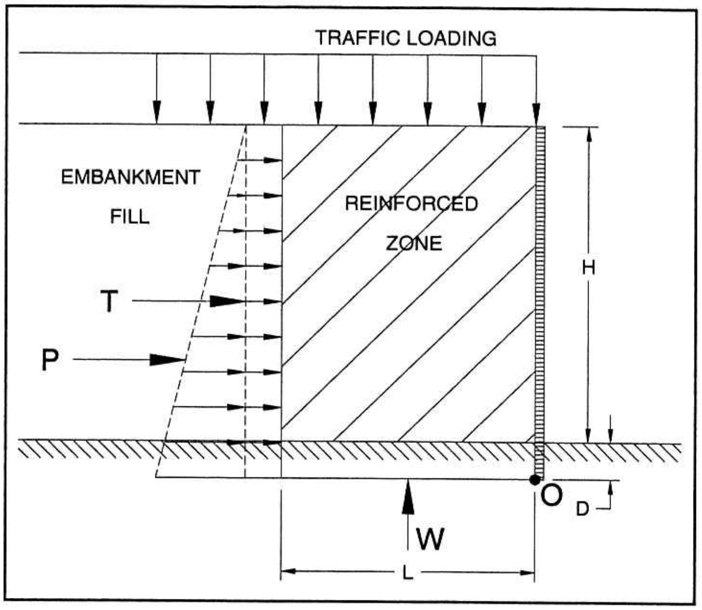
c = 1000 psf Cohesion
ϕ' = 29 deg Friction angle
ω_T = 240 psf Traffic loading

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 86,431 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 138,056 lbs per foot of wall

Use Undrained Value

$P_r = L(c)$ (Undrained)

P_r = 51,280 lbs per foot of wall

USE THIS VALUE

$FS = \frac{P_r}{P_a}$

Calculated FS = **0.59**

Required FS = 1.50

Resistance Against Sliding is **No Good**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).

* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 10,113,589 lb-ft

$$\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 1,900,982 lb-ft

$$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturnin\ g}}$$

Calculated FS = **5.32**

Required FS = 2.00

Resistance Against Overturning is **OK**

DLZ

SUBJECT

Client TranSystems

Project SCI 823-0.00 Portsmouth Bypass

Item Bearing Capacity - MSE Wall

SR 823 over Portsmouth Minford Road

Forward Abutment Granular Fill

JOB NUMBER 0121-3070.03

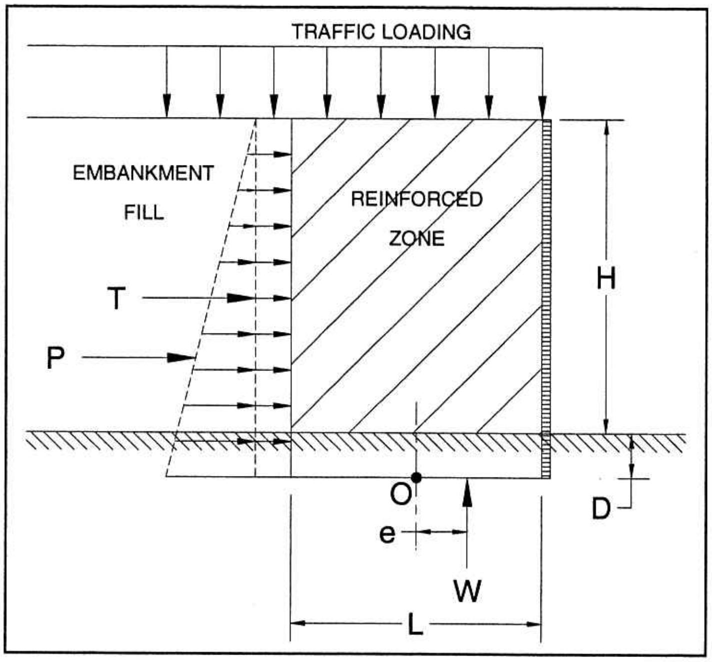
SHEET NO. 5 OF 6

COMP. BY SJR DATE 9/13/06

CHECKED BY DAA DATE 9/14/06

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 9,698 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 54,682 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 21,873 psf

Factor of Safety = **5.64** **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 54,682 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 21,873 psf

Factor of Safety = **5.64** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	51.28	ft	Length of MSE reinforcement
L factor	=	0.8		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	64.1	ft	
H	=	61.1	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	21.367	ft	Moment arm
Γ Wt	=	32.05	ft	Moment arm
B'	=	41.94	ft	
γ'	=	57.6	pcf	
W_t		12,307	lb/ft of wall	Weight from traffic
W_{mse}	=	394,446	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 4.67 ft

Kern

e < L/6 = 8.55 ft

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SUBJECT
Client TranSystems ODOT D-9
Project SCI 823-0.00 Portsmouth Bypass
Item MSE Wall Stability
SR 823 over Portsmouth - Minford Road
Forward Abutment Granular Fill

JOB NUMBER 0121-3070.03
SHEET NO. 6 OF 6
COMP. BY SJR DATE 09/13/06
CHECKED BY DAA DATE 9/14/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=65.5'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

Wall Properties

$H+D$ = 64.1 feet
γ_{mse} = 120 pcf
L = 51.28 feet
L factor = 0.80
ϕ = 30 deg

Foundational Soil Properties

c = 0 psf Cohesion
ϕ' = 34 deg Friction angle
ω_T = 240 psf Traffic loading
Length factor-range (0.7 - 1.0)
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 86,431 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 {AASHTO, Bridge Design Manual, 303.4.1.1}

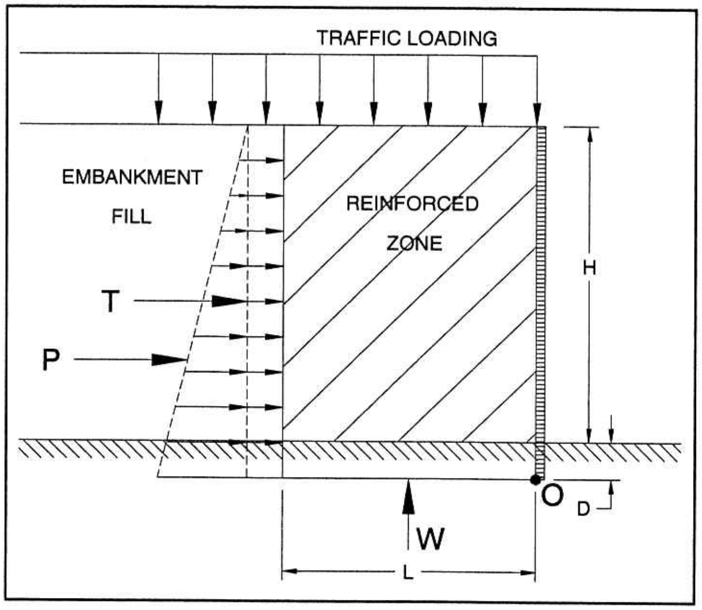
P_r = 177,501 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value



$FS = \frac{P_r}{P_a}$ Calculated FS = **2.05** Required FS = 1.50 Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 10,113,589 lb-ft

$$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 1,900,982 lb-ft

$$\Sigma M_{overturnig} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overtumin\ g}}$$

Calculated FS = **5.32** Required FS = 2.00 Resistance Against Overturning is **OK**

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PLANNERS • SURVEYORS

CLIENT TranSystems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft - End Bearing
Portsmouth Minford Road

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 2
COMP. BY SJR DATE 9-14-06
CHECKED BY DAA DATE 9-14-06

* From lab testing rock core samples (lower bound) $q_u = 8{,}000$ psi

FHWA-IF-99-025 $\quad Eq = 11.6 \quad q_{max}(MPa) = 4.83[q_u(MPa)]^{0.51}$

End Bearing

For RQD between 70 — 100 and $q_u > 0.5$ MPa (5.2 tsf)

$q_u = 8000$ psi $= 55.16$ MPa

$[Eq^{n} = 11.6] \quad q_{max} = 4.83[q_u(MPa)]^{0.51}$

$q_{max} = 4.83[55.16 \text{ MPa}]^{0.51} = 37.34$ MPa

$q_{max} = 37.34$ MPa $= 5416$ psi $= 780$ ksf

$$q_{allow} = \frac{q_{max}}{F.S.} = \frac{780 \text{ ksf}}{3.0} = 260 \text{ ksf}$$

For Competent Sandstone; Typically 2-3 feet below soil-Rock interface.

$\boxed{\text{* Use } q_{allow} = 80 \text{ ksf}}$

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CLIENT TranSystems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft Side Resistance
Portsmouth - Minford Road
PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 2
COMP. BY SJR DATE 9-14-06
CHECKED BY DAA DATE 9-14-06

* From lab testing rock core samples $q_u = 8000$ psi $f'_c = 4500$ psi

FHWA-IF-99-025 $Eq^{\underline{n}}$ 11.24 $f_{max} = 0.65\ p_a [q_u/p_a]^{0.5} \leq 0.65\ p_a [f'_c/p_a]^{0.5}$

Side friction - Smooth Rock Socket

$$f_{max} = 0.65\ p_a [q_u/p_a]^{0.5} \leq 0.65\ p_a [f'_c/p_a]^{0.5}$$

$$f_{max} = 0.65\,(14.70\text{ psi})\left[\frac{8000\text{ psi}}{14.70\text{ psi}}\right]^{0.5} \leq 0.65\,(14.70\text{ psi})\left[\frac{4500\text{ psi}}{14.70\text{ psi}}\right]^{0.5}$$

$$f_{max} = 222.9\text{ psi} > 167.2\text{ psi}$$ Use $f_{max} = 167$ psi

$$f_{allow} = \frac{167\text{ psi}}{3.0} = 55\text{ psi} = 7{,}920\text{ psf}$$

$$\boxed{{}^{*}\text{Use } f_{allow} = 7{,}500\text{ psf}}$$