Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
SR 823 Over Swauger Valley-Minford Road
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:

TRANSYSTEMS CORPORATION

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DLZ Job No. 0121-3070.03

September 26, 2006

Prepared by

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

SR 823 OVER SWAUGER VALLEY – MINFORD ROAD

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

For:

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1.0 INTRODUCTION

This report includes the findings of evaluation 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 Swauger Valley – 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 Swauger Valley – Minford Road (CR-31). 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 the proposed SR 823 over Swauger Valley – Minford Road 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 442+17 and 444+06 to contain the abutments and hold back the roadway embankment for the 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 442+17 (Rear Abutment) and 444+06 (Forward Abutment) will be approximately 63.0 and 58.5 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of SR 823 and the approximate existing grade along the Swauger Valley – Minford Road.

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 five final and four preliminary structural borings. Borings B-5 through B-9 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over the Swauger Valley – Minford Road (CR-31). The borings were drilled between June 15 and 16, 2006. Preliminary structural borings (TR-20 through TR-23) were drilled for a previous design configuration. The preliminary borings were drilled between August 3, 2004 and February 24, 2005. A boring plan is presented in Appendix I. Boring logs for borings TR-20 through TR-23, and B-5 through B-9 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

Final Borings B-5 through B-9 and TR-22 are considered most representative of the conditions near the proposed structures. Other preliminary borings are included for informational purposes.

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). It should be noted that as-per-plan coordinates and elevations were used for borings B-5, B-7, B-9, and TR-21 in lieu of as-drilled survey information.

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 Logan

Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the west of the structures roughly above elevation 880. In the area of the structure, the bedrock was covered by a thin soil overburden ranging in thickness between 1.5 and 7.5 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-20, TR-21, B-5, and B-7 were drilled for the west (forward) abutment. Borings TR-22, TR-23, and B-9 were drilled for the east (rear) abutment, while borings B-6 and B-8 were drilled for the piers.

Borings TR-20, TR-22, B-5, B-7, and B-9 encountered surficial material consisting of 1 to 8 inches of topsoil. The topsoil in borings B-5, B-7, and B-9 was underlain by bedrock. Borings TR-20 through TR-23, B-6, and B-8, encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of medium stiff to hard silt and clay (A-6a), stiff to hard sandy silt (A-4a), very stiff silt (A-4b), while the granular soil deposits consisted mainly of loose gravel (A-1-a) and very dense sandy silt (A-4a). The native soil deposits extended to an approximate depth ranging between 1.5 and 7.5 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 of medium hard to hard, slightly to highly weathered, slightly to moderately fractured sandstone. The amount of rock recovered in each core run varied between 81 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 17 and 100 percent with an average of 81 percent indicating good rock.

Unconfined compressive strength of tested cores ranged between 7,966 psi and 13,418 psi. The tested cores correspond to samples at depths between 3.5 feet and 18.5 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1, on the following page.

Table 1-Unconfined Compressive Strength Results

Boring	Depth (ft)	Unconfined Compressive Strength (psi)
B-5	3.5-4.0	8,382
B-6	18.0-18.5	13,418
B-7	6.5-7.0	7,966
B-8	17.0-17.5	10,997
B-9	7.2-7.7	8,153

4.2.3 Groundwater Conditions

Seepage was not encountered in any boring during drilling. 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-6 and B-8 upon the completion of coring between approximate depths of 0.5 and 12.5 feet. Boring TR-21 was drilled in a streambed and hence was completely submerged in water.

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 bearing on rock have been evaluated. On the other hand, the site is well suited for the use of MSE wall 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 pile 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 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. 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. Although the borings did not encounter significant seepage, 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 cavein. 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 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 bedrock or granular fill placed on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be 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 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. 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 below 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 Recommendation

Structural Element	Structure / Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-9	647.5 $^{+}$	Pipe Piles	636.5 *	Pile Capacity^{++}
			Drilled Shafts	636.5 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
	Right / TR-22	636.2	Pipe Piles	625.2 *	Pile Capacity^{++}
			Drilled Shafts	625.2 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
Pier	Left / B-8	638.4	Spread Footings	627.9	80 ksf
			Drilled Shafts	622.9 *	80 ksf^{+++}
	Right / B-6	635.9	Spread Footings	627.4	80 ksf
			Drilled Shafts	622.4 *	80 ksf^{+++}
Forward Abutment	Left / B-7	658.0 $^{+}$	Pipe Piles	647.0 *	Pile Capacity^{++}
			Drilled Shafts	647.0 *	80 ksf^{+++}
			Spread Footings	MSE Fill**	4 ksf
	Right / B-5	644.0 $^{+}$	Pipe Piles	635.5 *	Pile Capacity^{++}
			Drilled Shafts	635.5 *	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.

$^{+}$ Ground surface elevation was estimated from the established topographic mapping in lieu of as-drilled survey information.

$^{++}$ 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)	Compacted Granular Fill	120	0	34	0	34
Foundation Soil (Forward Abutment)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Wall Evaluations and Recommendations

The MSE wall at the rear abutment (station 442+17) is understood to have a maximum height of approximately 63 feet. The overburden in this area is very thin. It is recommended that the leveling pad be extended to bedrock or soil be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation. If founded on bedrock, no embedment into the rock is required. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. 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. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to ODOT BDM Item 204. In addition, because the wall will be founded on or near bedrock, stability should be adequate. For stability, calculations have shown that a minimum reinforcement length of (H+D) times 0.7, or 44.1 feet, must be used for 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 significantly. If soft soils are encountered while excavating for the MSE wall-leveling pad, these soils should be removed and replaced with compacted granular fill. 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.

The MSE wall at the forward abutment (Station 444+14) is understood to have a maximum height of approximately 58.5 feet. The overburden in this area is relatively thin (1.0 to 4.5 feet). It is recommended that the leveling pad be extended to bedrock or soil be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation. If founded on bedrock, no embedment into the rock is required. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. 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. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204. In addition, because the wall will be founded on or near bedrock, stability should be adequate. For stability, calculations have shown that a minimum reinforcement length of (H+D) times 0.7, or 41.0 feet must be used for the proposed MSE wall at this location.

It should be noted that the foundation leveling pad of the MSE wall at the forward abutment is in close proximity to a creek, which is running essentially parallel to Swauger Valley – Minford Road. The approximate elevation of bedrock under the MSE wall at the forward abutment ranges from 642.5 to 654.5 feet, which is near the bottom of the creek at elevation 631. If scour and erosion near the toe of the MSE wall are a concern, then slope protection should be provided with riprap.

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 compacted granular fill foundations. Drawings illustrating the typical soil and rock benches are presented in Appendix IV.

A summary of soil properties, summary of the results of calculations, and MSE retaining wall parameters are presented in Tables 4 and 5 on the following pages.

Table 4-MSE Retaining Wall Parameters and Analyses Results
(Rear Abutment)
Borings TR-23 & B-9

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} = 15,893 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 15,893 psf

Global Stability
Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)

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

Approximate Maximum Height of MSE Wall = 63.0 feet
Approximate Embedment Depth = 0.0 feet (Bedrock)
Minimum Length of Reinforcement for External Stability = 44.1 feet

Table 5-MSE Retaining Wall Parameters and Analyses Results
(Forward Abutment)
Borings TR-20 & B-5

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} = 14,734 psf

Allowable Bearing Capacity – Drained Condition
q_{all} = 14,734 psf

Global Stability
Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock)
Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)

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

Approximate Maximum Height of MSE Wall = 58.5 feet
Approximate Embedment Depth = 0.0 feet (Bedrock)
Minimum Length of Reinforcement for External Stability = 41.0 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 Riedy
Geotechnical Engineer

Wael Alkasawneh, P.E.
Geotechnical Engineer

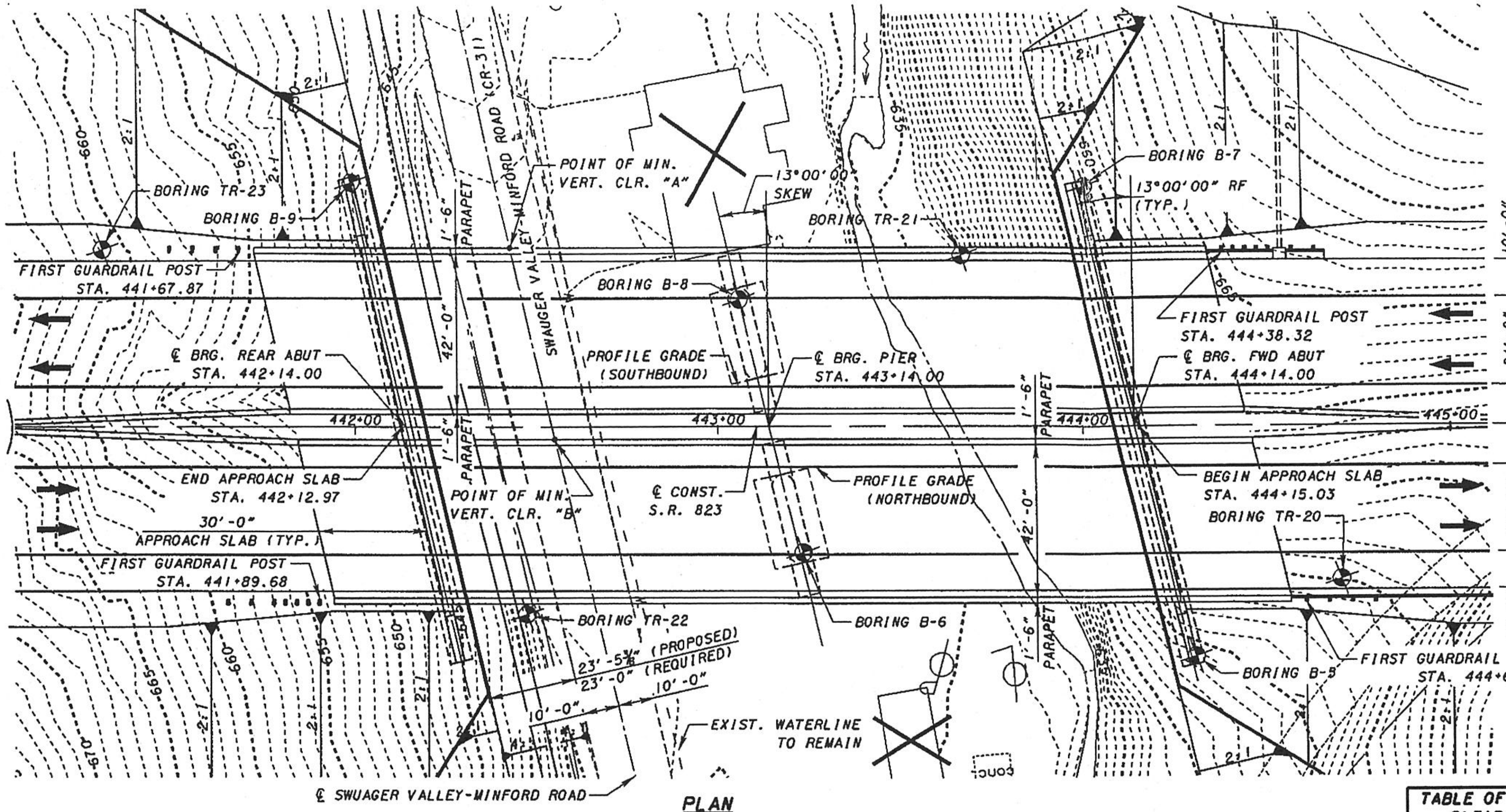
sjr

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\05 Swauger Valley - Minford Road\Final\Joint Structure Report\Swauger Valley Road Structure Report 09-26-06 - WMA_rev.doc

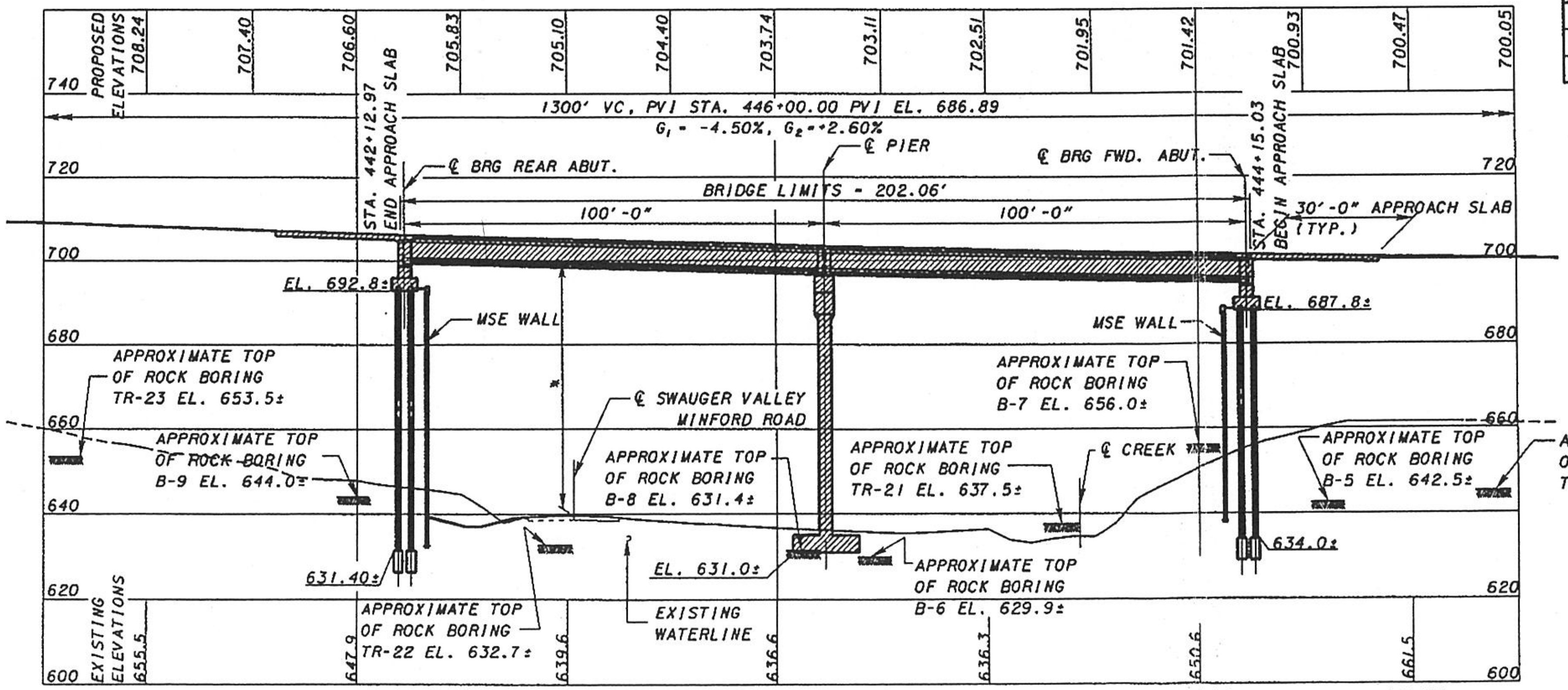
APPENDIX I

Structure Plan and Profile Drawing – 11”x17”

Boring Plan - 11”x17”



PLAN



* SEE TABLE OF VERTICAL CLEARANCES

ELEVATION ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS

LOCATION	STATION	OFFSET
REAR ABUT.	441+89.68	RT.
REAR ABUT.	441+67.87	LT.
FWD. ABUT.	444+60.13	RT.
FWD. ABUT.	444+38.32	LT.

BORING LOCATIONS

BORING No.	STATION	OFFSET
TR-20	444+69.72	42.10' RT.
TR-21	443+66.99	46.44' LT.
TR-22	442+46.91	51.49' RT.
TR-23	441+30.33	48.06' LT.
B-5	444+29.99	63.31' RT.
B-6	443+22.99	34.59' RT.
B-7	444+00.99	65.40' LT.
B-8	443+05.99	34.57' LT.
B-9	441+98.99	66.15' LT.

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

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	56.6±	58.7'±
PREFERRED	15.0'	15.0'

TRAFFIC DATA

(SR 823)

CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,968
DESIGN YEAR ADTT (2030) = 4,368

PROPOSED STRUCTURE

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

SPANS: 100'-0", 100'-0" c/c BEARINGS

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

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

SKEW: 13°00'00" RF

CROWN: 0.016 FT./FT.

ALIGNMENT: TANGENT

WEARING SURFACE: 1" MONOLITHIC SURFACE

APPROACH SLABS: AS-1-81 (30 FT LONG)

LATITUDE: 38°51'0"N

LONGITUDE: 82°52'3"W

HYDRAULIC DATA

DRAINAGE AREA = 13.424 sq.mi. = 8591 acres

Q_{50} = XXX cfs
Q_{100} = XXX cfs
V_{50} = XXX fps
V_{100} = XXX fps
EL 50 = XXX
EL 100 = XXX

OHWM: EL. XXX

AREA BELOW OHWM: XXX ACRES

TEMP. FILL BELOW OHWM: XXX 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 HP 14" DIA. C.I.P. REINFORCED CONCRETE PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE

DESIGN AGENCY
Systems
5747 PERIMETER DRIVE, SUITE 240
DUBLIN, OHIO 43017

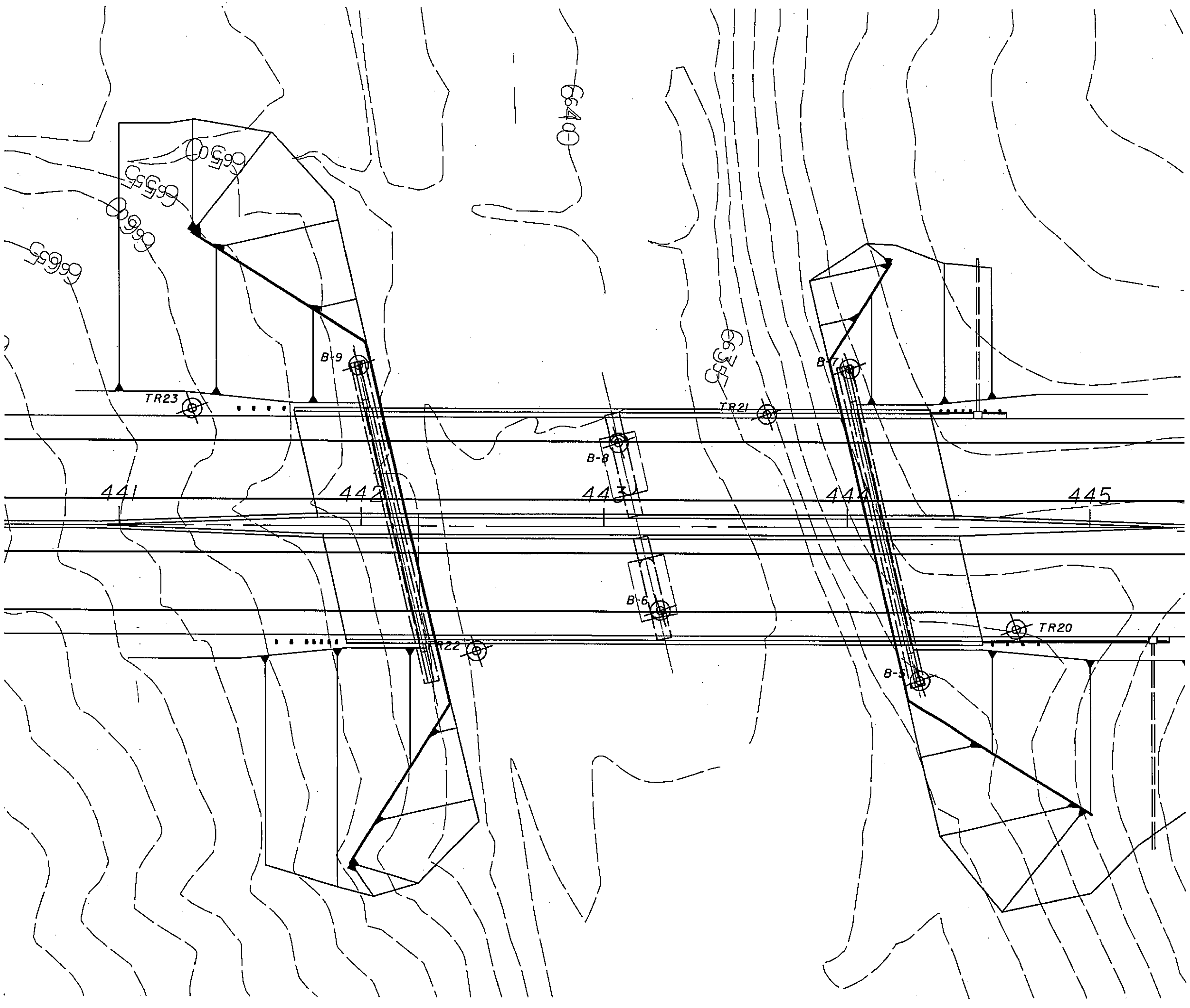
DESIGNED	DRAWN	REVIEWED	DATE
MSL	MTN	JRC	07/31/06
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP			

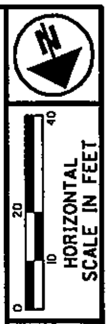
SCIOTO COUNTY
STA. 442+12.97
STA. 444+15.03

SITE PLAN
BRIDGE NO. SCI-823-XXXX
S.R. 823 OVER SWAUGER VALLEY-MINFORD ROAD (CR-31)

SCI-823-0.00
PID 19415

2 / 4





CALCULATED

CHECKED

BORING PLAN
SR 823 OVER SWAUGER VALLEY-MINFORD ROAD

SCI-823

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

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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-5 *Location:* Sta. 444+30.2, 63.3 ft. RT of SR 823 CL *As Per Plan *Date Drilled:* 06/15/06

WATER OBSERVATIONS:
Water seepage at: none
Water level at completion: none (prior to coring)
0.5' (inside hollowstem augers, 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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢⊣ LL Blows per foot - ○ 10 20 30 40
	644.0													
0.1	643.9						Topsoil - 1"							
		18 50/0	6	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone)							50+
1.5	642.5	Core 120"	Rec 114"	RQD 83%	R-1		Medium hard to hard gray SANDSTONE, fine to very fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, moderately fractured, contains few argillaceous laminations. @ 1.5'-7.2', 7.9'-8.3', 10.9', rust staining. @ 3.1'-3.3', high angle fracture. @ 3.5', qu = 8,382 psi.							
11.5	632.5						Bottom of Boring - 11.5'							

5
10
15
20
25
30

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Client: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

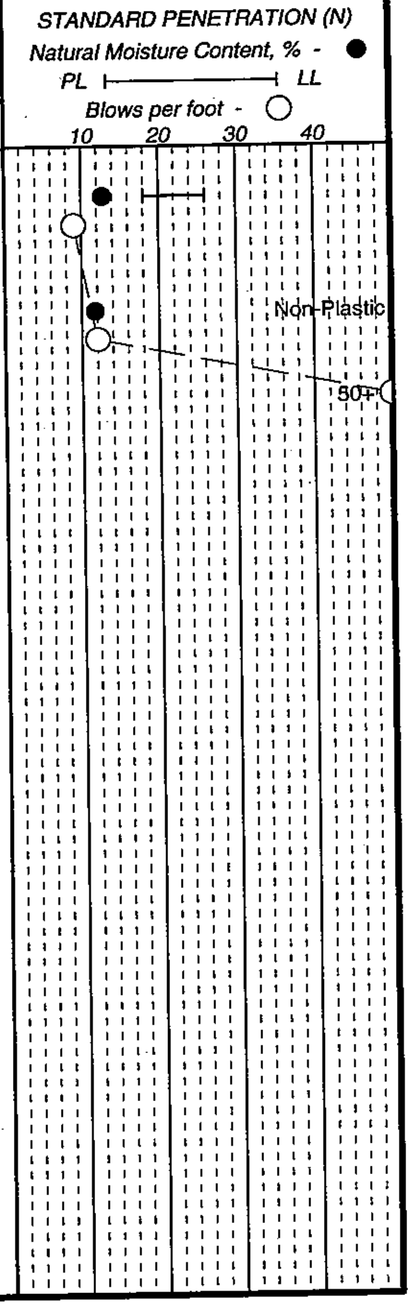
LOG OF: Boring B-6

Location: Sta. 443+23.0, 34.6 ft. RT of SR 823 CL

Date Drilled: 06/15/06 to 06/16/06

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 Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	635.9												
		7 5 4	17	1		4.5+	Hard brown SANDY SILT (A-4a), little clay, trace to little gravel; damp.	16	23	--	15	28	18
5		6 9 3		2		4.5+		10	25	--	14	40	11
6.0	629.9	50/1	1	3			Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, massively bedded, slightly fractured, contains moderate argillaceous laminations. @ 6.5'-7.4', rust staining. @ 6.8', 6.9', 7.1', 7.9', rust stained low angle fractures.						
10		Core 108"	Rec 105"	RQD 81%	R-1								
15		Core 120"	Rec 120"	RQD 89%	R-2		@ 18.0', qu = 12,418 psi.						
20													
25		Core 12"	Rec 10"	RQD 83%	R-3								
26.5	609.4						Bottom of Boring - 26.5'						
30													



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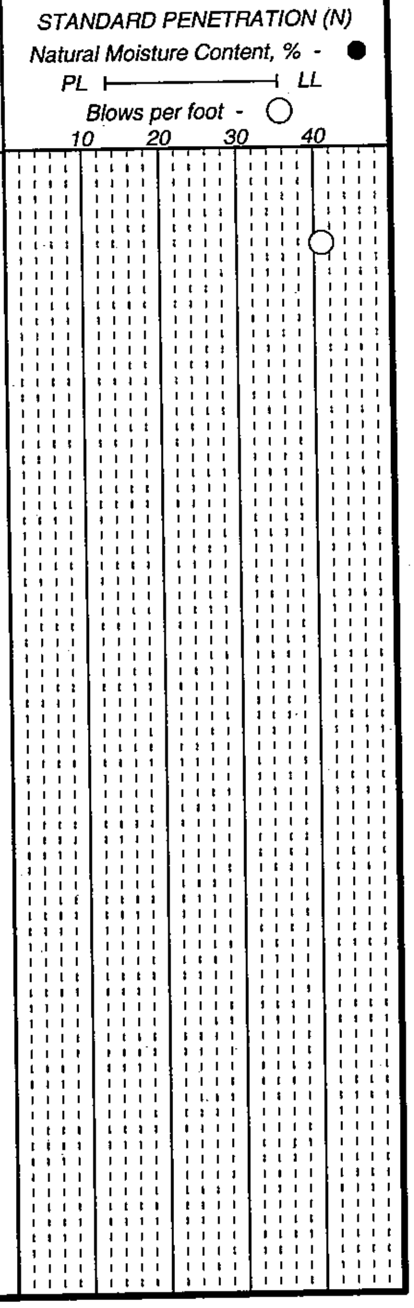
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring B-7
Location: Sta. 444+00.8, 65.4 ft. LT of SR 823 CL *As Per Plan
Date Drilled: 06/15/06

WATER OBSERVATIONS: Water seepage at: none
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
	658.0												
0.2	657.8						Topsoil - 2"						
		8 18 23	13	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone)						
2.5	655.5	Core 120"	Rec 97"	RQD 76%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, fossiliferous (trace fossils), massive bedding, highly fractured. @ 2.5'-9.3', rust staining. @ 2.5'-5.0', broken zone. @ 3.5', lost water circulation. @ 6.5', qu = 7,966 psi. @ 8.7'-8.9', high angle fracture.						
12.5	645.5						Bottom of Boring - 12.5'						



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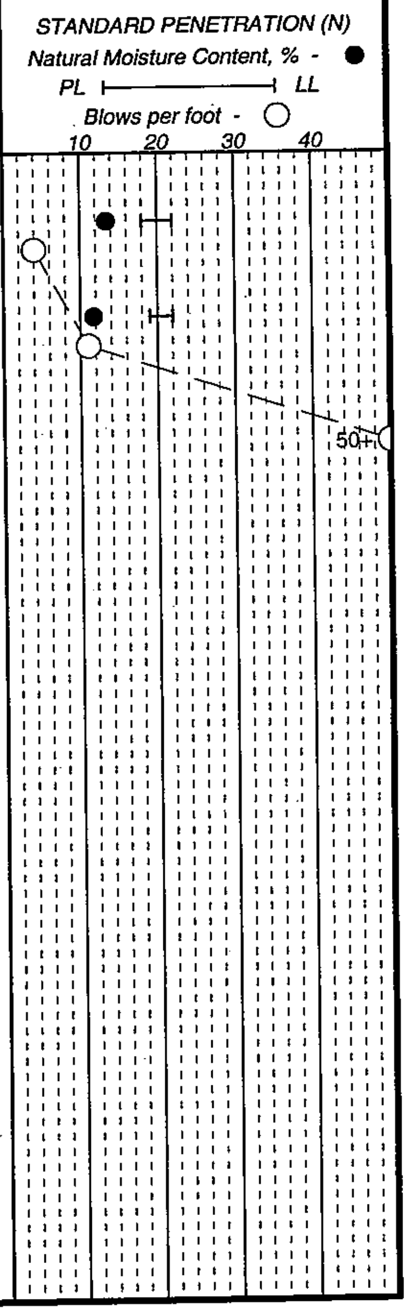
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring B-8
Location: Sta. 443+05.8, 34.6 ft. LT of SR 823 CL
Date Drilled: 06/16/06

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 Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	638.4	3 2 2	11	1		4.0	Very stiff to hard brown SILT (A-4b), little clay, trace fine to coarse sand, trace gravel; damp.	4	12	--	15	52	17
3.0 5	635.4	7 6 5	17	2		4.5+	Hard brown SANDY SILT (A-4a), little clay, little gravel; damp.	12	23	--	17	35	13
		9 15 50/4		3		4.5+							
7.5 10 15	630.9	Core 108"	Rec 87"	RQD 68%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, highly fractured, contains few laminations. @7.5'-8.7', rust staining.						
20 25		Core 120"	Rec 120"	RQD 90%	R-2		@ 17.0', qu = 10,997 psi.						
		Core 12"	Rec 12"	RQD 100%	R-3								
27.5 30	610.9						Bottom of Boring - 27.5'						



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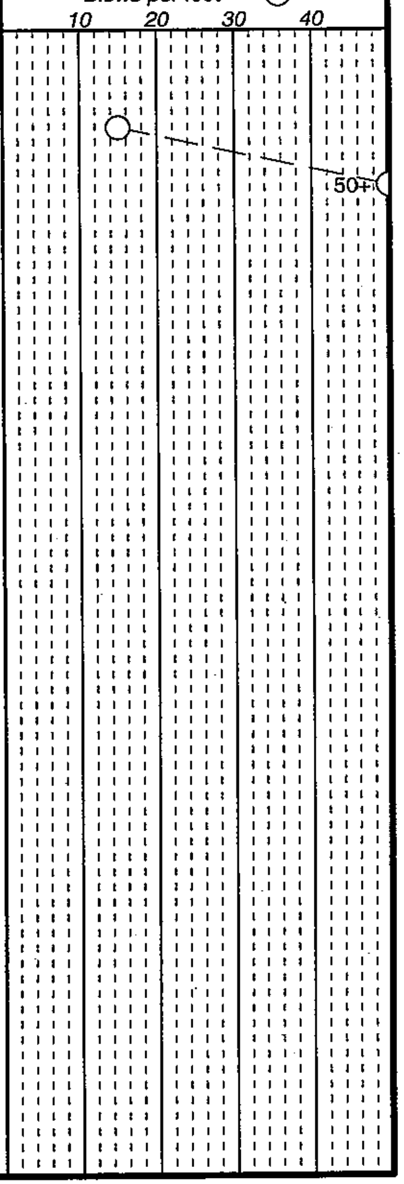
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring B-9
Location: Sta. 441+98.6, 66.2 ft. LT of SR 823 CL *As Per Plan
Date Drilled: 06/15/06

WATER OBSERVATIONS:
Water seepage at: none
Water level at completion: none (prior to coring)
1.0' (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	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.1	647.5 647.4						Topsoil - 1"						
		7 7 8	4	1			Medium dense gray SANDY SILT (A-4a), damp. (Decomposed Sandstone) @ 4.0', auger refusal.						
4.0	643.5	50/0	0	2									
5 10		Core 120"	Rec 117"	RQD 66%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, micaceous, massive bedding, highly fractured. @ 4.0'-11.4', rust staining. @ 7.2', qu = 8,153 psi. @ 8.7'-8.8', 8.9'-9.0', Decomposed argillaceous zones.						
14.0	633.5						Bottom of Boring - 14.0'						
15 20 25 30													

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



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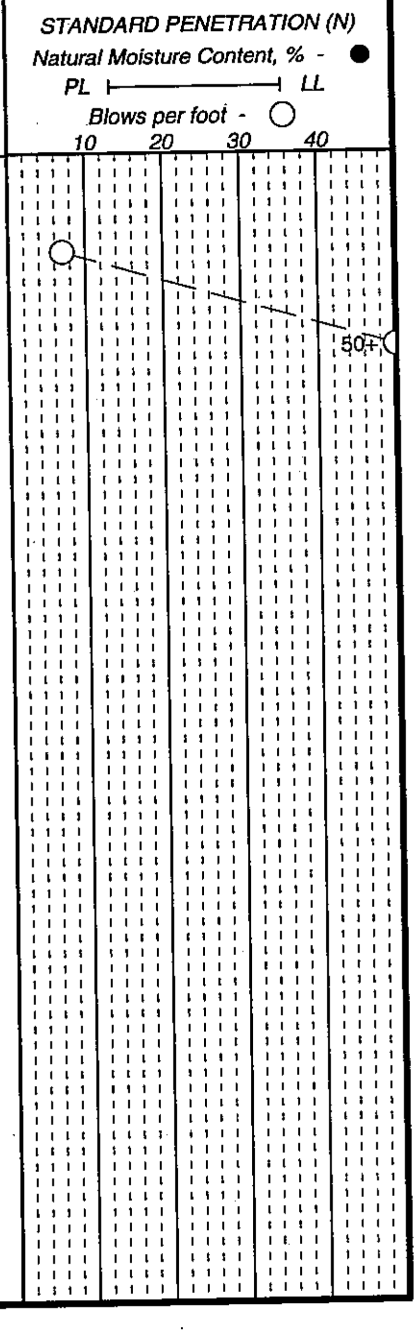
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-20
Location: Sta. 444+69.7, 42.1 ft. RT of SR 823 CL
Date Drilled: 8/4/04

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: 6.3' (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
	650.0												
0.2	649.8						Topsoil - 2"						
		3 3 4	18	1		0.5	Medium stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, little gravel; contains sandstone fragments; moist.						
		1 3 50/3	15	2									
5.0	645.0	Core 96"	Rec 91"	RQD 88%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, massively bedded, slightly fractured. @ 5.0'-5.3', broken. @ 9.3'-9.5', broken zone, possible core loss.						
		Core 84"	Rec 84"	RQD 86%	R-2		@ 13.9' to 14.5', high angle fracture with reddish brown discoloration.						
20.0	630.0						Bottom of Boring - 20.0'						



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

LOG OF: Boring TR-21

Location: Sta. 443+67.0, 46.5 ft. LT of SR 823 CL *As Per Plan
Date Drilled: 8/3/04

WATER OBSERVATIONS:
Water seepage at: 0.0'
Water level at completion: 0.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No.		Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION						STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
				Drive	Press / Core			% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	
0	639.0						Gray GRAVEL (A-1-a); wet. (Auger sample - boring drilled in stream bed)							
1.5	637.5	Core 114"	Rec 114"	RQD 70%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, argillaceous, massively bedded, slightly fractured. @ 1.5'-3.9', brown, highly weathered, highly fractured to broken. @ 3.3'-3.4', clay filled fracture.							
5														
10														
		Core 108"	Rec 108"	RQD 93%	R-2									
15														
20.0	619.0						Bottom of Boring - 20.0'							
25														
30														

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

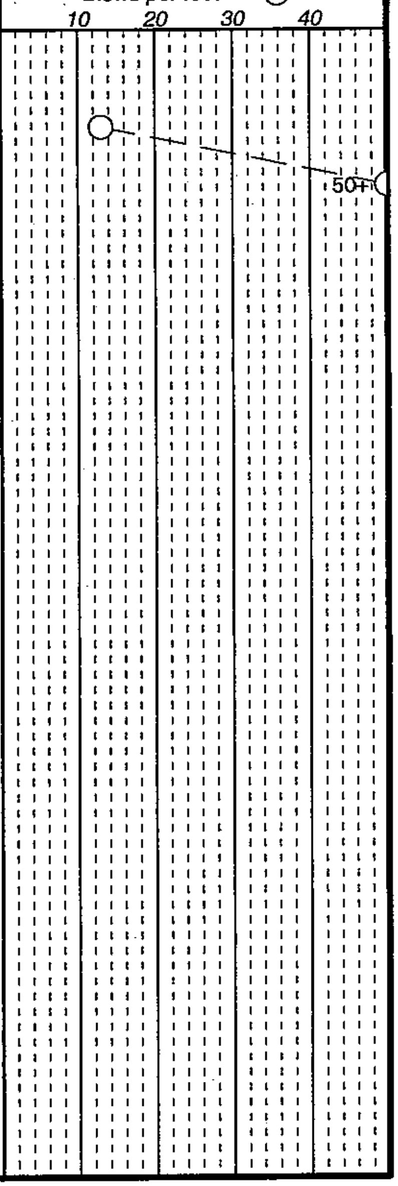
LOG OF: Boring TR-22
Location: Sta. 442+46.9, 51.5 ft. RT of SR 823 CL
Date Drilled: 2/24/2005

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: 4.5' (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	636.2						Topsoil - 8"						
0.7	635.5	3 6 7	18	1		1.25	Stiff brown SANDY SILT (A-4a), trace gravel; organic; moist.						
2.8	633.4												
3.5	632.7	26 50/4	10	2A 2B			Very dense brown SANDY SILT (A-4a), trace gravel; organic; moist.						
4.0	632.2	Core 120"	Rec 120"	RQD 84%	R-1		Severely weathered brown SANDSTONE. Soft brown SANDSTONE; fine grained, moderately weathered, slightly micaceous, moderately fractured. @ 5.2'-5.7',7.1'-7.3',8.7'-8.9' very soft, highly weathered. @ 6.1', gray, medium hard. @ 10.9'-11.0', iron stained horizontal fractures. @ 12.0'-12.8', siltstone.						
14.0	622.2	Core 120"	Rec 120"	RQD 96%	R-2		Hard gray SANDSTONE; fine grained, slightly weathered, slightly micaceous, slightly fractured. @ 14.7'-15.3', very soft gray and brown SILTSTONE, highly weathered. @ 19.3'-19.4', irregular vertical fracture. @ 19.6', 1/2" clay filled fracture. @ 23.2'-23.5', siltstone.						
24.0	612.2						Bottom of Boring - 24.0'						

Depth scale: 0, 5, 10, 15, 20, 25, 30

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



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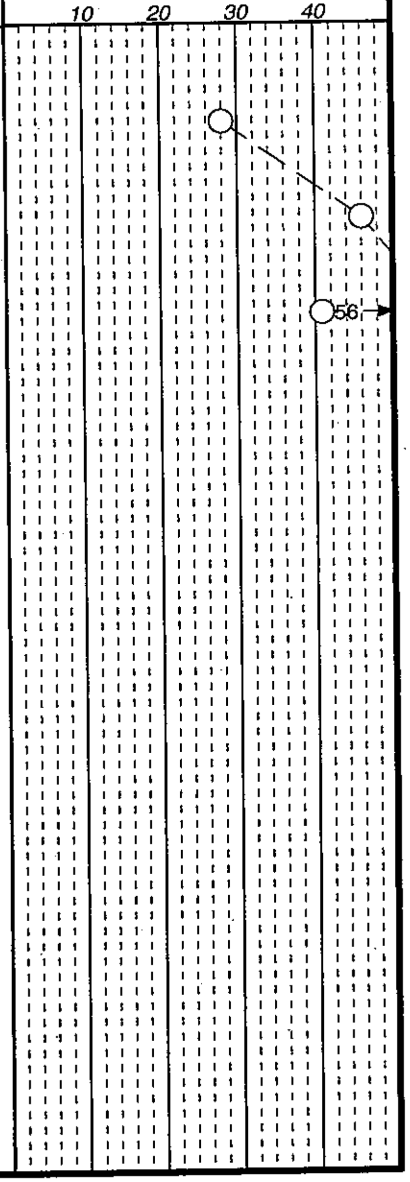
Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-23
Location: Sta. 441+30.3, 48.1 ft. LT of SR 823 CL
Date Drilled: 8/9/04

WATER OBSERVATIONS: Water seepage at: none
Water level at completion: 2.0' (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	GRADATION % Aggregate	GRADATION % C. Sand	GRADATION % M. Sand	GRADATION % F. Sand	GRADATION % Silt	GRADATION % Clay
0 0.3	661.0 660.7						Topsoil - 4"						
		6 13 15	17	1		4.5+	Hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; contains sandstone fragments; damp.						
5		11 26 20	17	2		4.5							
		11 16 40	16	3		4.5+							
7.5	653.5	Core 30"	Rec 26"	RQD 17%	R-1		Soft brown SANDSTONE; fine grained, decomposed.						
10.0 15	651.0	Core 120"	Rec 120"	RQD 84%	R-2		Hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, slightly fractured. @ 12.3',13.5', weathered fractures. @ 12.9' to 13.6', brown.						
20.0 25 30	641.0						Bottom of Boring - 20.0'						

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



APPENDIX III

Laboratory Test Results

DLZ

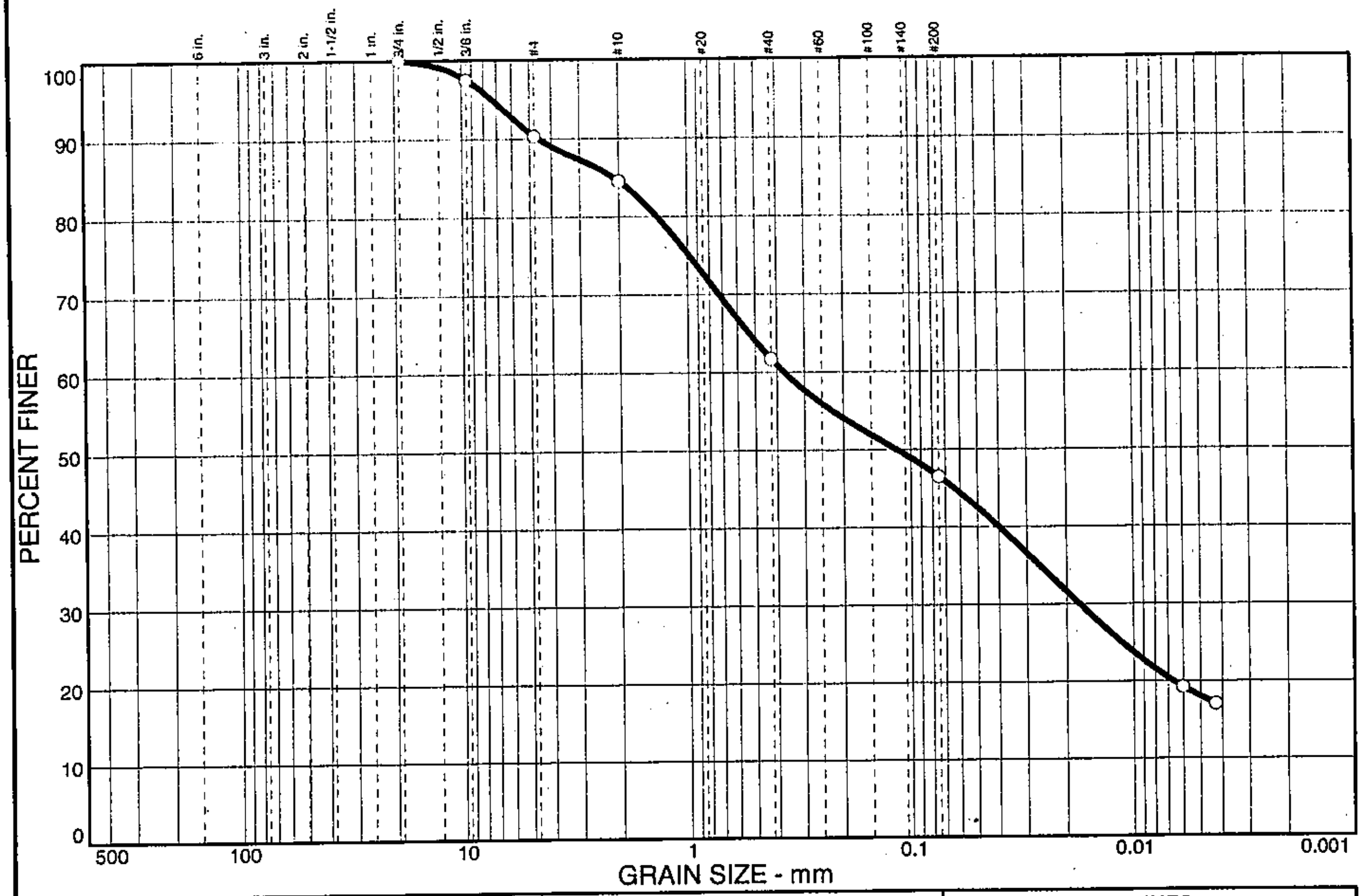
SUBJECT SCI-823 Portsmouth Bypass
Structures and MSE Walls
Unconfined Strength - Rock Results

JOB NUMBER 0121-3070.03
SHEET NO.
COMP. BY SJR
CHECKED BY

Unconfined Compression Test Results - Rock

Boring	Depth (ft.)	Avg Dia. (in.)	Avg L (in.)	L/D	Weight (g)	X-Section Area	Volume (ft^3)	Unit Weight (pcf)	Load (lb-f)	Calculated Stress (psi)	Rock Type
B-5	3.5-4.0	1.972	4.854	2.461	544.65	3.06	0.008575	140.0	25,550	**8,382**	Sandstone
B-6	18.0-18.5	1.986	4.753	2.393	606.38	3.10	0.008517	157.0	40,900	**13,418**	Sandstone
B-7	6.5-7.0	1.968	4.903	2.491	549.42	3.05	0.008627	140.4	24,280	**7,966**	Sandstone
B-8	17.0-17.5	1.986	4.811	2.422	603.89	3.10	0.008621	154.4	33,520	**10,997**	Sandstone
B-9	7.2-7.7	1.969	4.763	2.419	518.96	3.05	0.008389	136.4	24,850	**8,153**	Sandstone

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	9.7	5.8	22.8	15.2	28.5	18.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	97.5		
#4	90.3		
#10	84.5		
#40	61.7		
#200	46.5		

* (no specification provided)

Soil Description

Clayey sand

Atterberg Limits

PL= 18
LL= 26
PI= 8

Coefficients

D_{85}= 2.12
D_{60}= 0.372
D_{50}= 0.116
D_{30}= 0.0176
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS= SC
AASHTO= A-4(1)

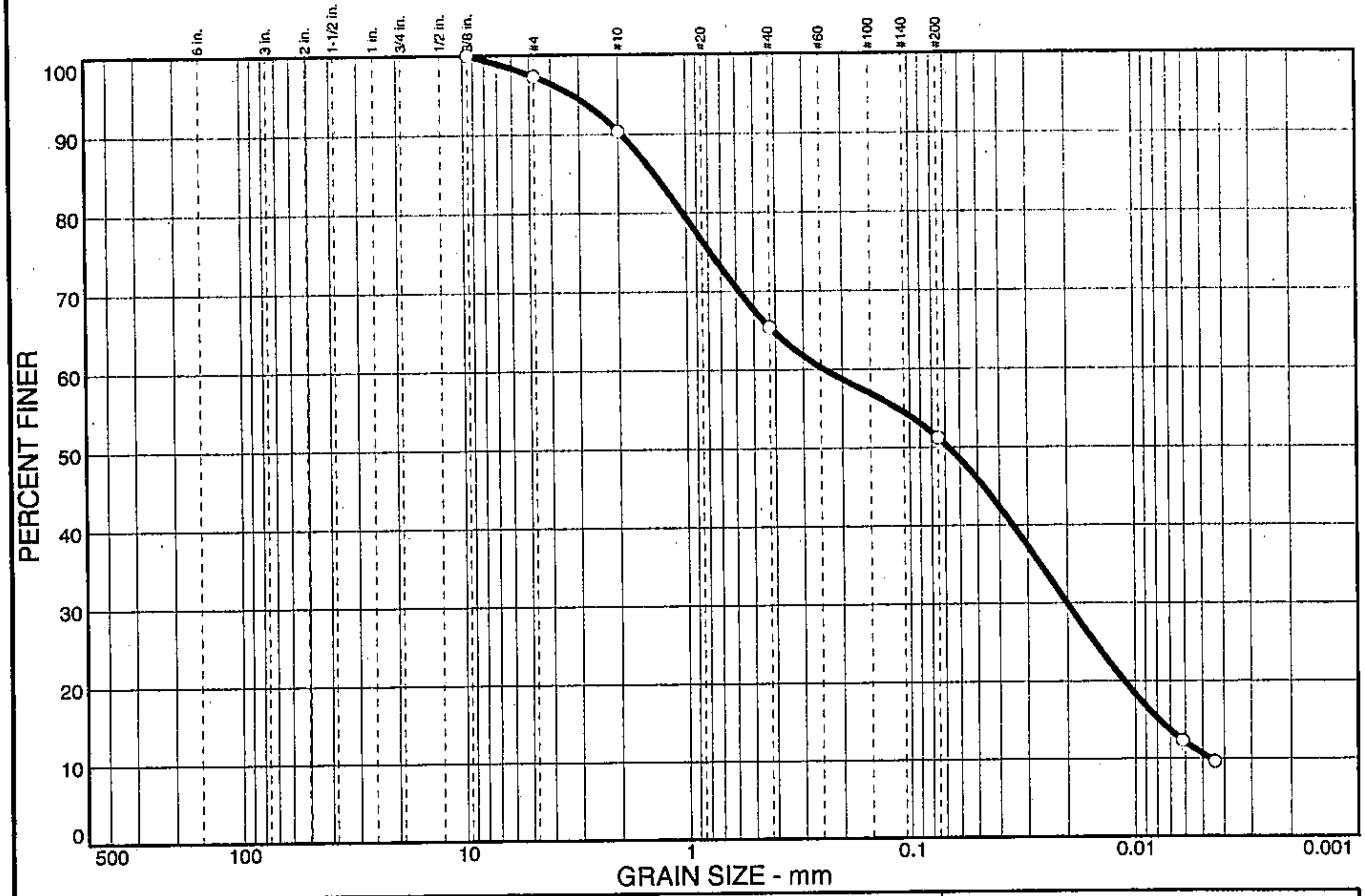
Remarks

Moisture Content= 12.7%

Sample No.: 1
Location:
Source of Sample: B-6
Date: 7/14/06
Elev./Depth: 0.5

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	2.7	7.0	24.9	14.2	40.6	10.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	97.3		
#10	90.3		
#40	65.4		
#200	51.2		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 1.39 D_{60}= 0.241 D_{50}= 0.0677
D_{30}= 0.0201 D_{15}= 0.0079 D_{10}= 0.0046
C_u= 51.92 C_c= 0.36

Classification

USCS= ML AASHTO= A-4(0)

Remarks

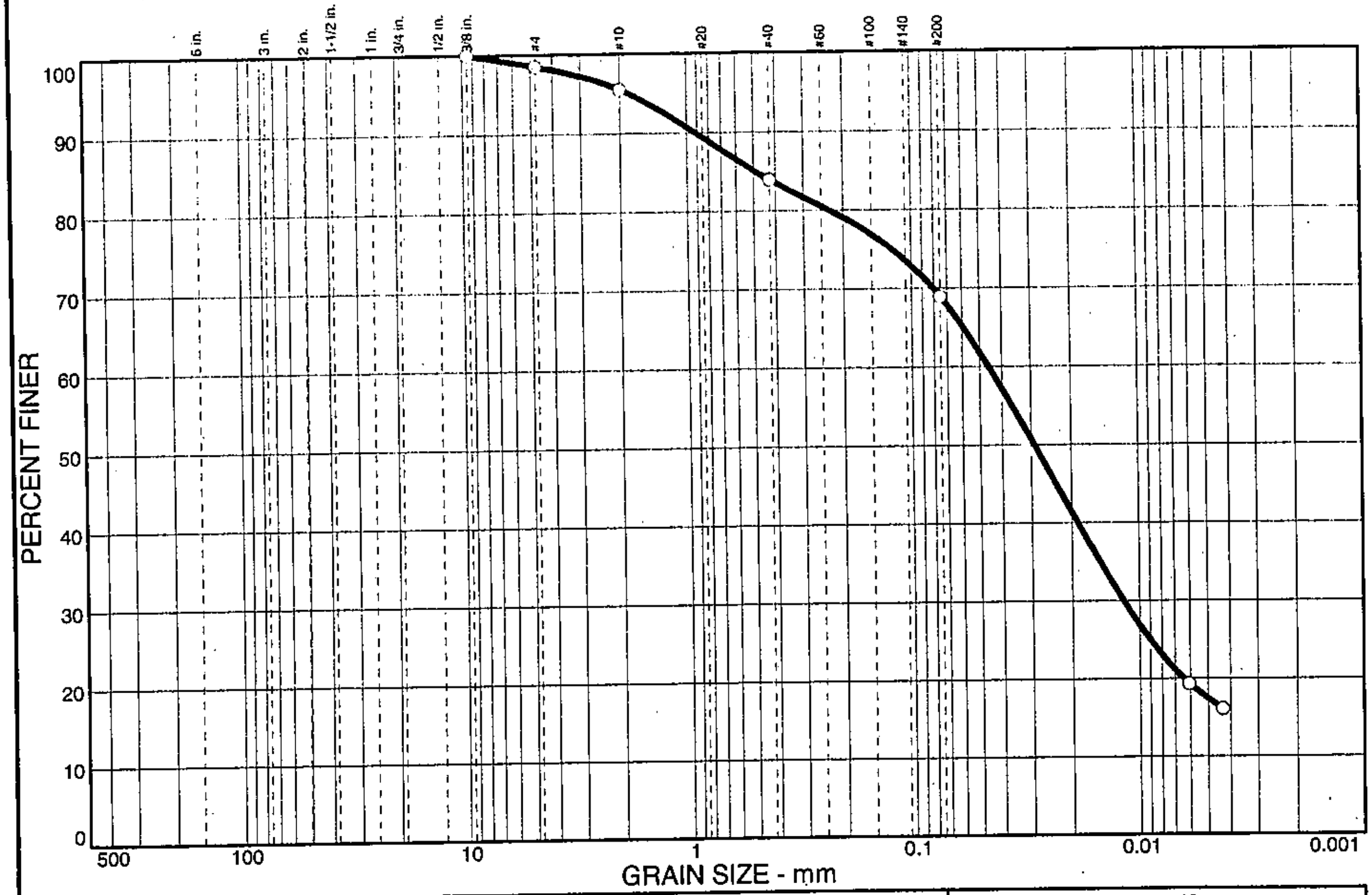
Moisture Content= 11.7%

Sample No.: 2 **Source of Sample:** B-6 **Date:** 7/14/06
Location: **Elev./Depth:** 3.5

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	1.5	3.0	11.6	15.0	51.5	17.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.5		
#10	95.5		
#40	83.9		
#200	68.9		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 18 LL= 22 PI= 4

Coefficients

D_{85}= 0.494 D_{60}= 0.0460 D_{50}= 0.0292
D_{30}= 0.0118 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(0)

Remarks

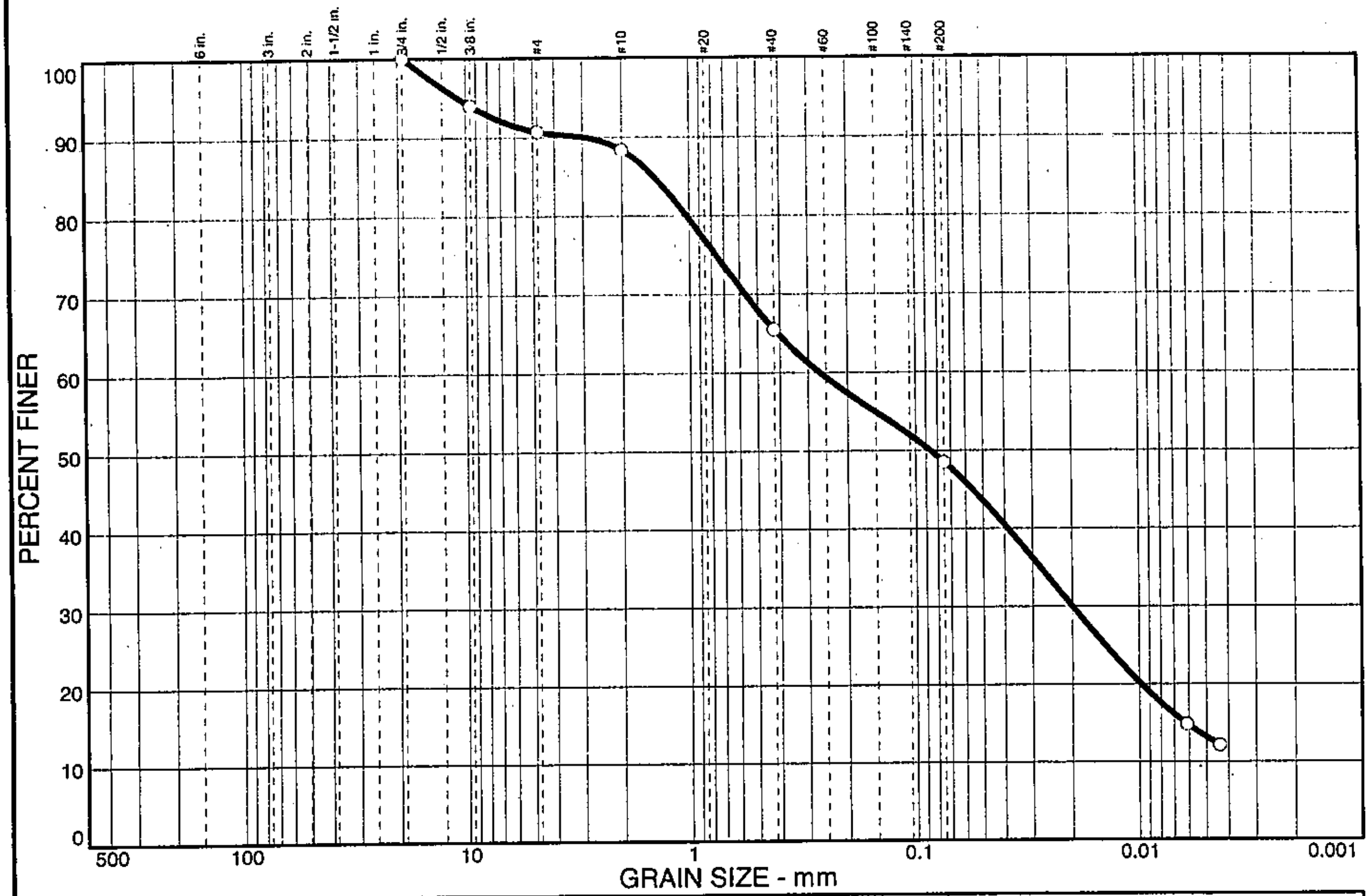
Moisture Content= 13.4%

Sample No.: 1 **Source of Sample:** B-8 **Date:** 7/14/06
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	9.3	2.4	22.8	17.0	35.4	13.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	94.0		
#4	90.7		
#10	88.3		
#40	65.5		
#200	48.5		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= 19 LL= 22 PI= 3

Coefficients

D_{85}= 1.45 D_{60}= 0.267 D_{50}= 0.0866
D_{30}= 0.0200 D_{15}= 0.0063 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content= 11.7%

Sample No.: 2 **Source of Sample:** B-8 **Date:** 7/14/06
Location: **Elev./Depth:** 3.5

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 Friction Calculations

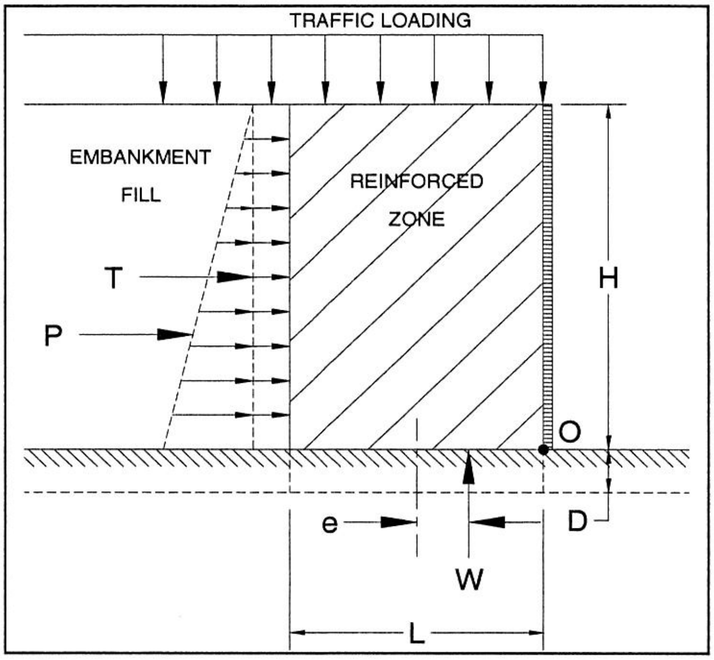
DLZ

SUBJECT
Client TranSystems
Project SCI 823-0.00
Item Bearing Capacity (Rear Abutment)
05 - 823 over Swauger Valley-Minford Rd TR-20
Bedrock / Granular Fill Foundation

JOB NUMBER 0121-3070.03
SHEET NO. 1 OF 4
COMP. BY SJR DATE 9/25/06
CHECKED BY WMA DATE 9/25/06

BEARING CAPACITY OF A MSE WALL *(non-coped)*

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



Soil Properties

γ_{MSE}	=	120	pcf	unit weight	EMB/MSE
ϕ'_{MSE}	=	30	deg.	friction ang.	embankment
γ_{FDN}	=	120	pcf	unit weight	foundation soil
c_{FDN}	=	0	psf	cohesion	undrained
ϕ_{FDN}	=	34	deg.	friction ang.	undrained
c'_{FDN}	=	0	psf	cohesion	drained
ϕ'_{FDN}	=	34	deg.	friction ang.	drained

Loads and Parameters

ωt	=	240	psf	traffic loading
L=B	=	44.1	ft	length of mse block
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	0	ft	embedment depth
Dw	=	0	ft	groundwater depth
H+D	=	63	ft	
H	=	63	ft	height of wall
Ka	=	0.33		
Γ Pa	=	21	ft	moment arm
Γ Wt	=	31.5	ft	moment arm
B'	=	33.60	ft	
γ'	=	57.6	pcf	
W_t	=	10,584	lb/ft of wall	
W_{mse}	=	333,396	lb/ft of wall	

Effective Bearing Pressure

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

σ_v = 10,238 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 39,733 psf

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

q_{ALL} = 15,893 psf

Factor of Safety = **3.88** 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} = 39,733 psf

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

q_{ALL} = 15,893 psf

Factor of Safety = **3.88** OK

Bearing Capacity Factors for Equations

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 = 5.25 ft

Kern

e < L/6 = 7.35 ft

DLZ

SUBJECT

Client TranSystems ODOT D-9

Project SCI 823-0.00 Portsmouth Bypass

Item MSE Wall Stability (Rear Abutment)

05 - 823 over Swauger Valley - Minford Rd

Bedrock / Gran Fill

JOB NUMBER 0121-3070.03

SHEET NO. 2 OF 4

COMP. BY SJR DATE 09/25/06

CHECKED BY WMA DATE 9/25/06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=63'

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	=	63	feet
γ_{mse}	=	120	pcf
L	=	44.1	feet
L factor	=	0.70	

Range (0.7-1.0)

Foundational Soil Properties

c	=	0	psf	cohesion
ϕ'	=	34	deg	friction angle
ω_T	=	240	psf	traffic loading

Embankment Soil Properties

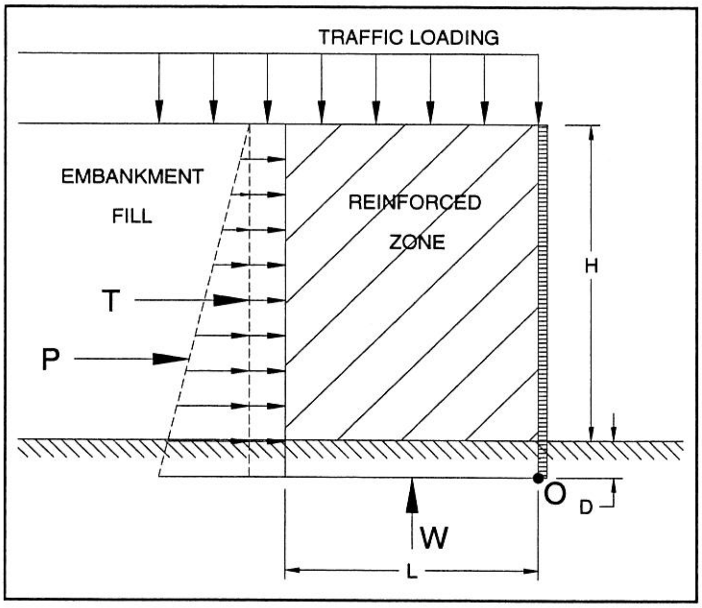
c	=	0	psf	cohesion
ϕ'	=	30	deg	friction angle

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 = 83,576 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 = 150,028 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	Required
FS = **1.80**	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

$\Sigma M_{resisting}$ = 7,351,382 lb-ft

$\Sigma M_{overturning}$ = 1,807,483 lb-ft

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

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

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

Calculated	Required
FS = **4.07**	FS = 2.00

Resistance Against Overturning is **OK**

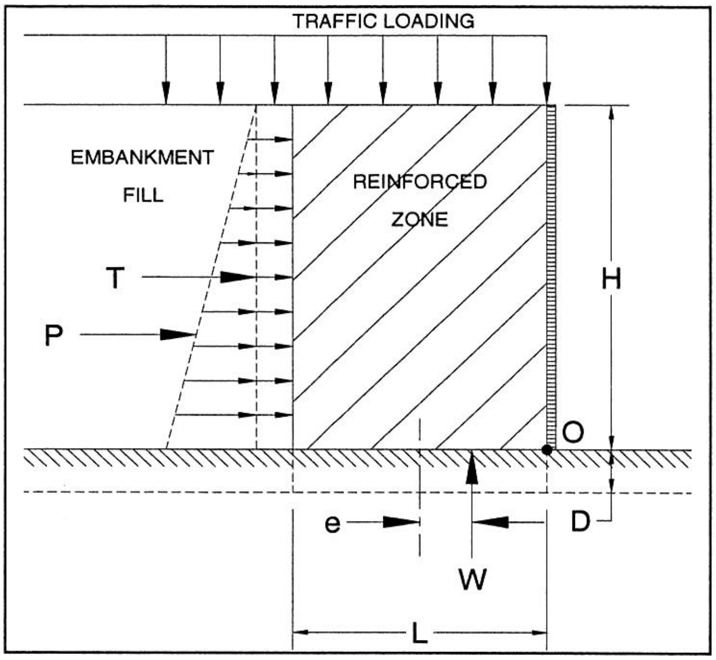
DLZ

SUBJECT			
Client	TranSystems	JOB NUMBER	0121-3070.03
Project	SCI 823-0.00	SHEET NO.	3 OF 4
Item	Bearing Capacity (Forward Abutment)	COMP. BY	SJR DATE 9/25/06
05 - 823 over Swauger Valley-Minford Rd TR-20		CHECKED BY	WMA DATE 9/25/06

Bedrock / Granular Fill Foundation

BEARING CAPACITY OF A MSE WALL *(non-coped)*

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



Effective Bearing Pressure

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

σ_v = 9,544 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 36,836 psf

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

q_{ALL} = 14,734 psf

Factor of Safety = **3.86** 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} = 36,836 psf

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

q_{ALL} = 14,734 psf

Factor of Safety = **3.86** OK

Soil Properties

γ_{MSE}	=	120	pcf	unit weight	EMB/MSE
ϕ'_{MSE}	=	30	deg.	friction ang.	embankment
γ_{FDN}	=	120	pcf	unit weight	foundation soil
c_{FDN}	=	0	psf	cohesion	undrained
ϕ_{FDN}	=	34	deg.	friction ang.	undrained
c'_{FDN}	=	0	psf	cohesion	drained
ϕ'_{FDN}	=	34	deg.	friction ang.	drained

Loads and Parameters

ωt	=	240	psf	traffic loading
L=B	=	40.95	ft	length of mse block
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	0	ft	embedment depth
Dw	=	0	ft	groundwater depth
H+D	=	58.5	ft	
H	=	58.5	ft	height of wall
Ka	=	0.33		
Γ Pa	=	19.5	ft	moment arm
Γ Wt	=	29.25	ft	moment arm
B'	=	31.15	ft	
γ'	=	57.6	pcf	
W_t	=	9,828	lb/ft of wall	
W_{mse}	=	287,469	lb/ft of wall	

Bearing Capacity Factors for Equations

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 = 4.90 ft

Kern

e < L/6 = 6.83 ft

DLZ

SUBJECT	Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03		
	Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	4	OF	4
	Item	MSE Wall Stability (Forward Abutment)	COMP. BY	SJR	DATE	09/25/06
	05 - 823 over Swauger Valley - Minford Rd		CHECKED BY	WMA	DATE	9/25/06
	Bedrock / Gran Fill					

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=58.5'
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 = 58.5 feet
γ_{mse} = 120 pcf
L = 40.95 feet
L factor = 0.70
Range (0.7-1.0)

Foundational Soil Properties

c = 0 psf cohesion
ϕ' = 34 deg friction angle
ω_T = 240 psf traffic loading

Embankment Soil Properties

c = 0 psf cohesion
ϕ' = 30 deg friction angle

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 = 72,394 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}

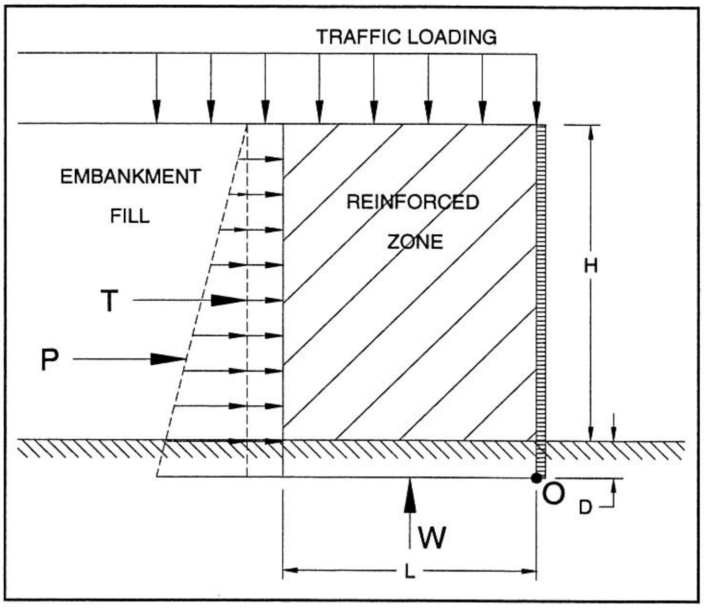
P_r = 129,361 lbs per foot of wall

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 = **1.79**

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

$\Sigma M_{resisting}$ = 5,885,928 lb-ft

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

$\Sigma M_{overturning}$ = 1,456,852 lb-ft

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

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

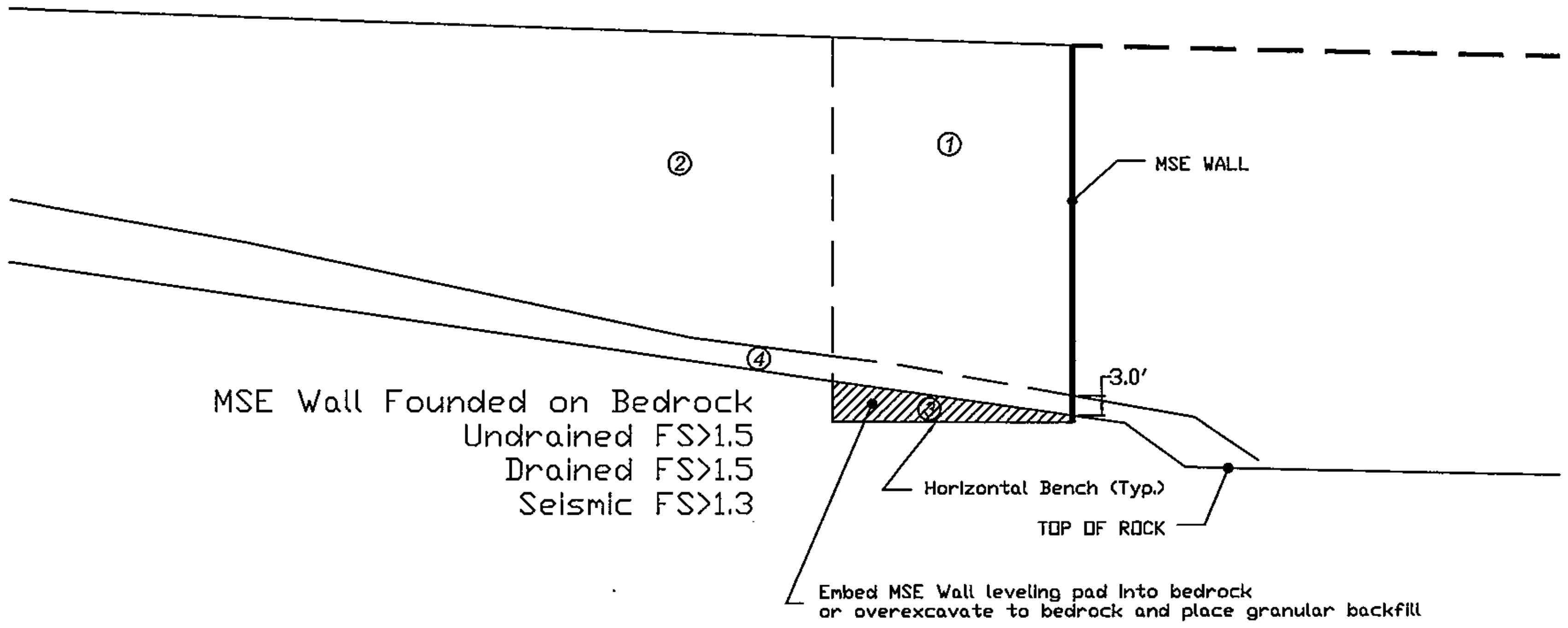
Calculated FS = **4.04**

Required FS = 2.00

Resistance Against Overturning is **OK**

			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	Compacted	Gran. Fill	0	34	0	34	120
Material 4	Hard	Silt and Clay	4500	0	0	29	120

MSE Wall STA: 442+17 (Rear Abutment)
Swauger Valley-Minford Road
Based on TR-23
H=63' (FROM PROPOSED GRADE TO PROPOSED
SR 823 GRADE)
Embedment=0.0' (BEDROCK)
Length=0.7(H+D)=44.1'



MSE Wall Founded on Bedrock
Undrained FS>1.5
Drained FS>1.5
Seismic FS>1.3

823 OVER SWAUGER VALLEY - MINFORD ROAD
REAR ABUTMENT

MSE STABILITY

SCI-823-0. 00

PROJECT NO. 0121-3070. 03 | CALC: SJR | DATE 9/25/06

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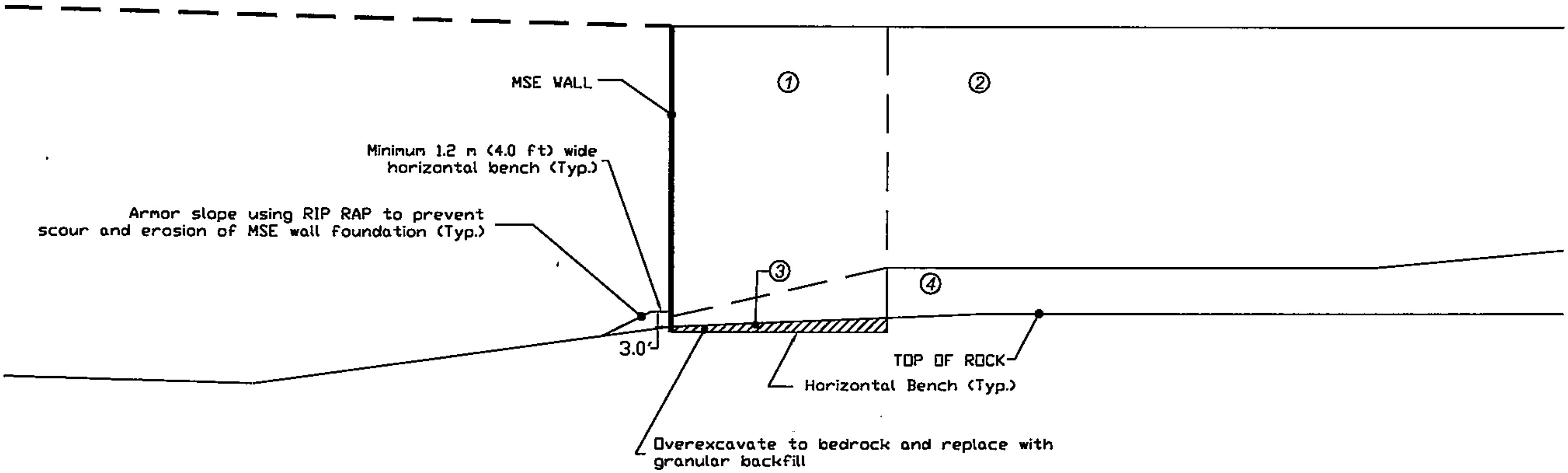
MSE Wall STA: 444+14 (Forward Abutment)
Swauger Valley-Minford Road
Based on B-5, TR-20
H=58.5' (Proposed Grade at Base to
Proposed Grade SR 823)
Embedment=0' (Bedrock)
Length=0.7(H+D)=41.0'

Material	Consistency	Soil Type	Undrained		Drained		
			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	Compacted	Gran. Fill	0	34	0	34	120
Material 4	Soft	Clay	500	0	0	29	120



MSE Wall Founded on Bedrock
Undrained FS>1.5
Drained FS>1.5
Seismic FS>1.3

823 OVER SWAUGER VALLEY - MINFORD ROAD
FORWARD ABUTMENT

MSE STABILITY

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 9/25/06

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CLIENT Tran Systems Corp

PROJECT SCI-823 Portsmouth Bypass

SUBJECT Drilled Shaft - End Bearing
Swanger Valley - Minford Rd.

PROJECT NO. 0121-3070.03

SHEET NO. 1 OF 2

COMP. BY SJR DATE 9-12-06

CHECKED BY WMA DATE 9/12/06

*From lab testing rock core samples $q_u = 8000$ psi

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

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] $q_{max} = 4.83[q_u(MPa)]^{0.51}$

$q_{max} = 4.83[55.16 MPa]^{0.51} = 35.87$ MPa

$q_{max} = 35.87$ MPa $= 5202$ psi $= 749$ ksf

$q_{allow} = \frac{q_{max}}{FS} = \frac{749 ksf}{3.0} = 250$ ksf

* Rock is stronger than concrete, Use $q_u = f'_c = 4500$ psi

$q_{max} = 4038$ psi $= 581$ ksf

$q_{allow} = \frac{581 ksf}{3.0} = 193.7$ ksf

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

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Tran Systems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft - Side Resistance
Swauger Valley - Minford Road

PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 2
COMP. BY SJR DATE 9-12-06
CHECKED BY WMA DATE 9/12/0

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

FHWA-IF-99-025 Eq $\hat{=}$ 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

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

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

$$f_{max} = 222.9\,psi \leq 167.2\,psi$$

Use $f_{max} = 167$ psi

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

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