Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
Proposed SR 823 Over Blue Run Road (CR 29)
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, Ohio 43017

Ohio Department of Transportation
District 9

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DLZ Job No. 0121-3070.03

January 18, 2007

Prepared by:

DLZ

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TranSystems

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

PROPOSED SR 823 OVER BLUE RUN ROAD (CR 29)

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

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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 Blue Run Road (CR 29) only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing two structures, eastbound and westbound structures, respectively for the proposed SR 823 over Blue Run Road (CR 29). The two structures as planned, are single-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 approach embankments. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

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

2.0 GENERAL PROJECT INFORMATION

It is understood that the plan location of the bridge structure for the proposed SR 823 over Blue Run Road (CR 29) 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 approximately stations 578+55.2 and 579+40.5 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, the height of the embankment at stations 578+55.2 (Rear Abutment) and 579+40.5 (Forward Abutment) will be approximately 47.3 and 31.0 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of SR 823 and the approximate existing grade along the proposed alignment.

It should be noted that a drainage channel is situated near the proposed MSE wall at the rear abutment. The leveling pad should be founded 1 foot below the creek elevation, or at the top of bedrock.

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 two borings (B-13 and B-14) drilled for the final (approved) bridge configuration and four borings (TR-07 through TR-10) drilled for a preliminary bridge configuration. Borings, B-13 and B-14 were drilled on June 30, 2006. Borings (TR-07 through TR-10) were drilled for a previous design configuration, and where drilled between March 11 and 15, 2005. All boring logs are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were planned and staked in the field 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, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Soils at the abutment locations are composed primarily of residual and colluvial soils. These soils are generally thin, covering moderate to steep slopes. Although not encountered in the borings, lacustrine soils in the 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 northwest of the structures roughly above elevation 1020. Similarly, bedrock of the Cuyahoga Formation of Mississippian age can be found in the lower portions of valleys northwest of the structure location below approximately elevation 690. In the area of the structure, the bedrock encountered in the borings was covered by a thin overburden ranging in thickness between 3 feet and 18.5 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 exploration 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 B-13 and B-14 were drilled for the rear and forward abutments, respectively, of the final (approved) structure configuration. The findings of structure borings drilled for previous configurations are also being considered in the evaluation of the subsurface conditions. Borings TR-09 and TR-10 are considered in the evaluation of the rear abutment location. Similarly, borings TR-07 were TR-08 were considered in the evaluation of the forward abutment location.

All borings encountered surficial material consisting of 1 inch to 4 inches of topsoil. Native cohesive soil deposits underlay the topsoil in all borings. The cohesive deposits consisted mainly of medium stiff to hard sandy silt (A-4a), stiff to hard (A-4b), soft to hard silt and clay (A-6a), and soft silty clay (A-6b). The native soil deposits extended to depths ranging between approximately 3.0 and 18.5 feet below the ground surface, where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was confirmed by coring in all borings. The bedrock consisted of soft to hard, slightly weathered to decomposed, slightly to highly fractured sandstone. Medium hard to hard sandstone with interbedded siltstone was encountered in boring B-14 below the sandstone. The amount of rock recovered in each core run varied between 94 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 17 and 83 percent with an average of 60 percent indicating fair quality rock.

Unconfined compressive strength of tested cores ranged between 11,952 psi and 5,840 psi. The tested cores correspond to samples at depths between 10.7 feet and

27.3 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-13	10.7-11.1	151.8	11,952
B-14	26.8-27.3	155.3	5,840

4.2.3 Groundwater Conditions

Seepage was encountered in borings TR-09 and TR-10 at an approximate depth of 1 foot. Water was used during rock coring and masked any seepage zones that might exist in the rock. No measurable water levels were observed in any borings prior to rock coring operations (before adding core water). Measurable water levels were present in all borings upon the completion of coring between approximate depths of 1.3 and 17.4 feet. However, water used during rock coring operations masked any seepage zones that might exist in the rock.

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. Additionally, 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 / MSE wall (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. Given the site conditions, 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.

The drilled shaft design parameters cited above consider axial loading only. If it is necessary to design drilled shafts to resist significant lateral loads, DLZ should be informed of the loading conditions to ensure recommendations for adequate socket lengths are provided.

When using drilled shaft foundations in conjunction with MSE retaining walls, it is necessary to consider the placement of the drilled shafts with respect to the MSE wall and soil reinforcing straps. Drilled shafts should be a sufficient distance from the back of the MSE wall such that the soil reinforcement can be spayed around the shafts with angles of 15 degrees or less. From the center of the drilled shafts to the back of the MSE wall, this dimension is approximately two times the shaft diameter.

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 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.

It is understood through comments from ODOT Office of Structural Engineering (OSE) that it is preferred to have the proposed MSE walls bearing on bedrock. As such, spread footings bearing on the compacted fill at the top of the MSE wall 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.

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 Recommendation

Structural Element	Structure / Boring	Existing Ground Surface Elevation $^{+}$ (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment (east)	Left / TR-10 B-13	768.1 770.0	Pipe Piles	758.1 * 760.0 *	Pile Capacity^{++}
Rear Abutment (east)	Left / TR-10 B-13	768.1 770.0	Drilled Shafts	758.1 * 760.0 *	80 ksf^{+++}
Rear Abutment (east)	Left / TR-10 B-13	768.1 770.0	Spread Footings	MSE Fill**	4 ksf
Rear Abutment (east)	Right / TR-09	772.8	Pipe Piles	760.8 *	Pile Capacity^{++}
Rear Abutment (east)	Right / TR-09	772.8	Drilled Shafts	760.8 *	80 ksf^{+++}
Rear Abutment (east)	Right / TR-09	772.8	Spread Footings	MSE Fill**	4 ksf
Forward Abutment (east)	Left / TR-08	802.4	Pipe Piles	781.4 *	Pile Capacity^{++}
Forward Abutment (east)	Left / TR-08	802.4	Drilled Shafts	781.4 *	80 ksf^{+++}
Forward Abutment (east)	Left / TR-08	802.4	Spread Footings	MSE Fill**	4 ksf
Forward Abutment (east)	Right / TR-07 B-14	814.3 801.7	Pipe Piles	799.3 *	Pile Capacity^{++}
Forward Abutment (east)	Right / TR-07 B-14	814.3 801.7	Drilled Shafts	799.3 *	80 ksf^{+++}
Forward Abutment (east)	Right / TR-07 B-14	814.3 801.7	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.

$^{+}$ Surveyed ground surface elevation at the boring.

$^{++}$ 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.

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.

The MSE walls were analyzed for bearing capacity, and stability (sliding and overturning). All calculations are included in Appendix IV. Other external and internal stability analyses (ie. reinforcing strap design) 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. 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.

The assumed shear strength parameters for the native soils have been selected based upon the results of laboratory testing, hand penetrometer test results, Standard Penetration Test (SPT) results, and conservative engineering judgment.

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 Rock (Rear Abutment)	Bedrock	145	NA	NA	NA	NA
Foundation Soil (Forward Abutment)	Native	125	1,500	0	0	29
Foundation Soil (Rear Abutment)	Native	125	1,000	0	0	29
Foundation Soil (Forward/Rear Abutments)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Wall Evaluations and Recommendations

Rear Abutment MSE Wall

Based on the structure site plan, the maximum height of the MSE wall at the rear abutment (station 578+54.5) is approximately 47.3 feet. The overburden in this area is relatively thin (3.0 to 6.0 feet). The MSE wall bearing capacity calculations indicated that the undrained bearing capacity of the native foundation soil is inadequate. Consequently, it is recommended that the leveling pad be extended to bedrock or existing soils be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation, which should be a minimum of 1 foot below the flow line of the drainage channel. 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 the full height (H+D) times 0.7 must be used for the proposed MSE wall at this location using a compacted granular fill foundation.

It should be noted that variations in the topography will likely be encountered within the proposed footprint of the proposed MSE wall, causing the top of bedrock elevation to vary slightly. 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.

As stated previously, the MSE wall at the rear abutment is in close proximity to a drainage channel, which is running essentially parallel to Blue Run Road (CR 29). The approximate elevation of bedrock under the MSE wall at the rear abutment ranges from 765.1 to 766.8, which is near the bottom of the drainage channel elevation. If scour and erosion near the toe of the MSE wall are a concern, then slope protection should be provided with riprap or other means.

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

Forward Abutment MSE Wall

Based on the structures site plan, the maximum height of the MSE wall at the forward abutment (station 579+40.5) is approximately 31.0 feet. The overburden in this area is relatively thin (6.5 to 18.5 feet). The MSE wall bearing capacity calculations indicated that the undrained bearing capacity of the native foundation soil is inadequate. Consequently, it is recommended that the leveling pad be extended to bedrock or existing soils 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 the full height (H+D) times 0.7 must be used for the proposed MSE wall at this location.

Calculations for the bearing capacity, overturning and sliding are attached for the native soil and compacted granular fill foundations.

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

Table 4-MSE Retaining Wall Parameters and Analyses Results
(Rear Abutment) ***Granular Fill Foundation (Undercut to Bedrock)***
Borings TR-09, TR-10 & B-13

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} = 14,589 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 14,589 psf
Global Stability Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock or Granular Fill) Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock or Granular Fill) Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock or Granular Fill)
Estimated Settlement of MSE volume Total settlement = 0 inches Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 50.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = 35.0 feet [0.7(H+D)]

Table 5-MSE Retaining Wall Parameters and Analyses Results
(Forward Abutment) ***Granular Fill Foundation (Undercut to Bedrock)***
Borings TR-07, TR-08 & B-14

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on Φ' = 30º)
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} = 10,606 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 10,606 psf
Global Stability Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock or Granular Fill) Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock or Granular Fill) Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock or Granular Fill)
Estimated Settlement of MSE volume Total settlement = 0 inches Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 34.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = 24.0 feet [0.7(H+D)]

5.3 Groundwater Considerations

Seepage was encountered only in borings TR-09 and TR-10 at an approximate depth of 1 foot. Water was used during rock coring and masked any seepage zones that might exist in the rock. No measurable water levels were observed in any borings prior to rock coring operations (before adding core water). Measurable water levels were present in all borings upon the completion of coring between approximate depths of 1.3 and 17.4 feet. 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 due to the proximity of a nearby creek. The contractor should be prepared to deal with seepage and water flow that may enter any excavations.

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.

5.4 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. test weld
- 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

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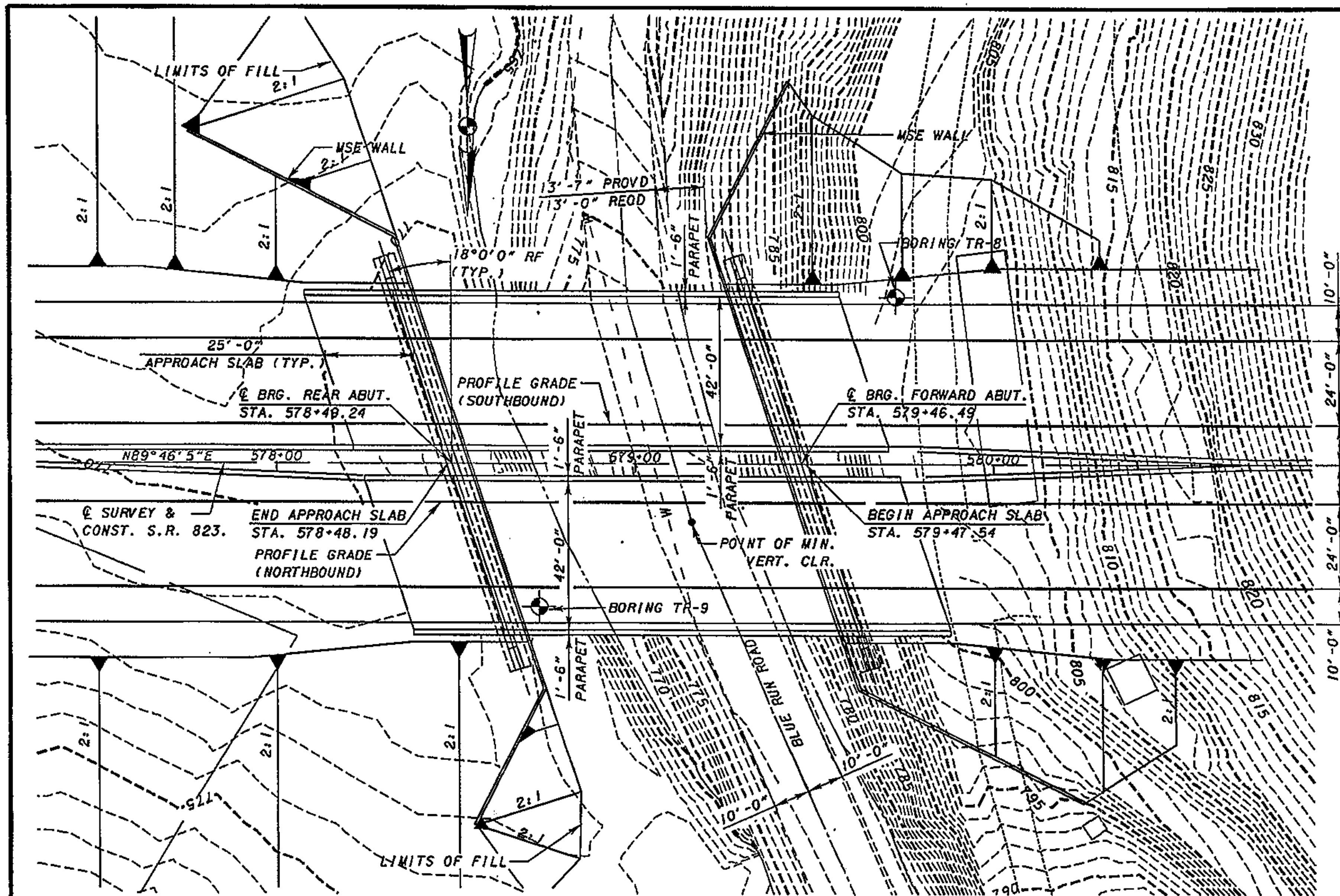
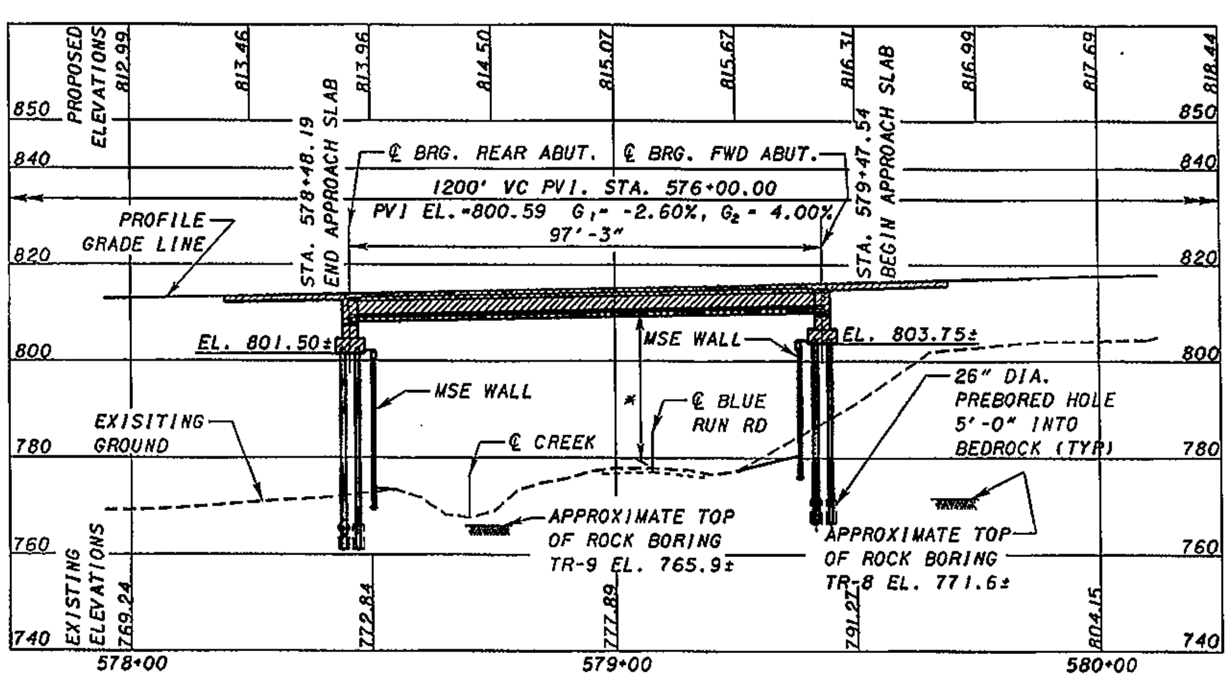
APPENDIX I

Structure Plan and Profile Drawing – 11”x17”

Boring Plan – 11”x17”



PLAN



ELEVATION ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

* MIN. VERT. CLR.
29'-1 9/16" ACTUAL
14'-6" REQUIRED

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	SIDE
REAR ABUT.	x	RT.
REAR ABUT.	x	LT.
FWD. ABUT.	x	RT.
FWD. ABUT.	x	LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.

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

TRAFFIC DATA

(SR 823)
CURRENT YEAR ADT (20XX) = 19,800
CURRENT YEAR ADTT (20XX) = 2,722
DESIGN YEAR ADT (20XX) = 26,000
DESIGN YEAR ADTT (20XX) = 3,640

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHWON 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" DIA. C.I.P. PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE.

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED DURING TS&L SUBMITTAL

PROPOSED STRUCTURE

TYPE: SINGLE SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS.
SPAN: 97'-3" c/c BEARINGS
ROADWAY: 2 - 42'-0" TOE TO TOE OF PARAPETS
LOADING: HS-25 AND ALTERNATE MILITARY LOADING; FWS = 60 PSF
SKEW: 18°00'00"
CROWN: 0.016 FT/FT
ALIGNMENT: TANGENT
WEARING SURFACE: 1" MONOLITHIC SURFACE
APPROACH SLABS: AS-1-81 (25'-0" LONG)
LATITUDE:
LONGITUDE:

DESIGN AGENCY: TranSystems
DESIGNED MSL | CHECKED PJP | DRAWN MTH | REVISED | REVIEWED JRC | DATE 7/18/06 | STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 578+48.19
STA. 579+47.54

PRELIMINARY SITE PLAN- ALTERNATIVE 4
BRIDGE NO. SCI-823-XXXX
S.R. 823 OVER BLUE RUN ROAD (CR-29)

SCI-823-0.00
PID 19415

1 / 3

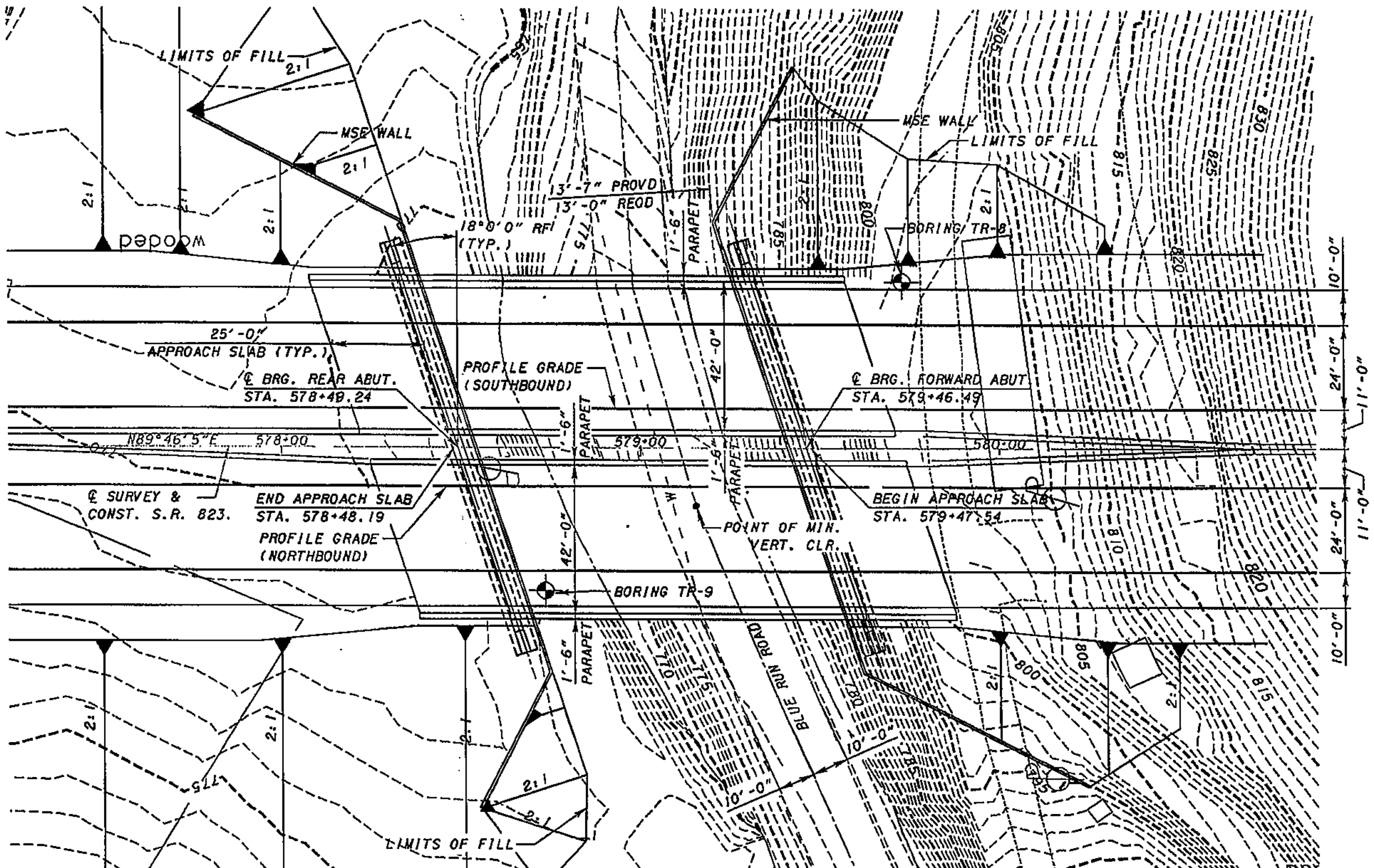
FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	SIDE
REAR ABUT.	x	RT.
REAR ABUT.	x	LT.
FWD. ABUT.	x	RT.
FWD. ABUT.	x	LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.

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

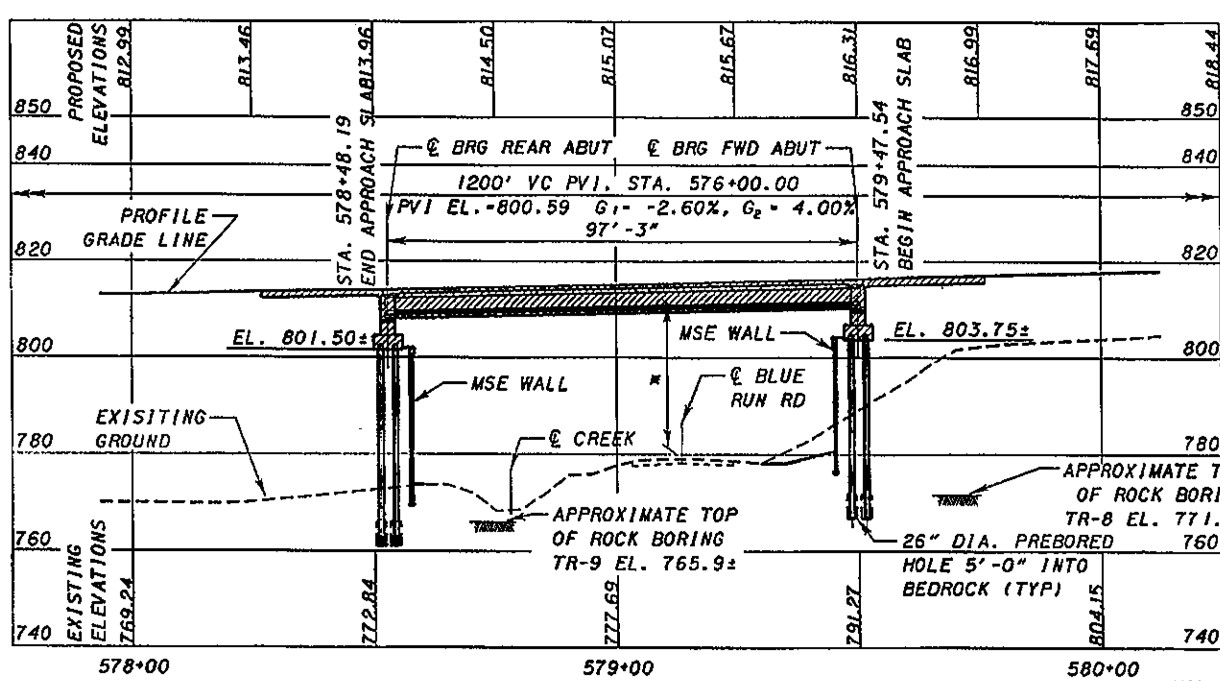
TRAFFIC DATA

(SR 823)
CURRENT YEAR ADT (20XX) - 19,800
CURRENT YEAR ADTT (20XX) - 2,722
DESIGN YEAR ADT (20XX) - 26,000
DESIGN YEAR ADTT (20XX) - 3,640



PLAN

℄ BLUE RUN ROAD



ELEVATION ALONG PROFILE GRADE S.R. 823 RIGHT BRIDGE

* MIN. VERT. CLR.
29'-1 9/16" ACTUAL
14'-6" REQUIRED

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHWON 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" DIA. C.I.P. PILES AND HAVE A MAXIMUM CAPACITY OF 70 TONS PER PILE.

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED DURING TS&L SUBMITTAL

PROPOSED STRUCTURE

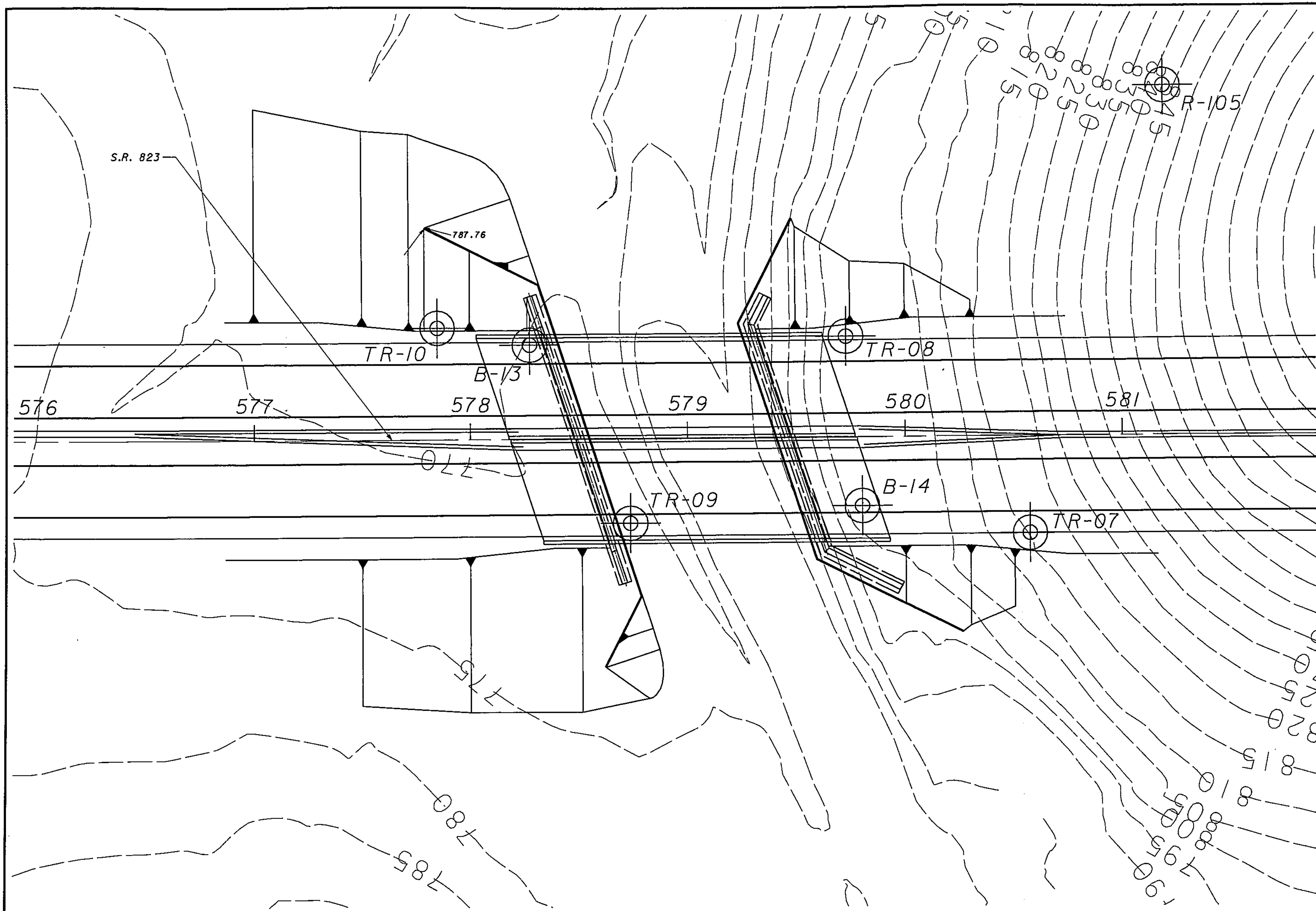
TYPE: SINGLE SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS.
SPAN: 97'-3" c/c BEARINGS
ROADWAY: 2 - 42'-0" TOE TO TOE OF PARAPETS
LOADING: HS-25 AND ALTERNATE MILITARY LOADING; FWS - 60 PSF
SKEW: 18°00'00"
CROWN: 0.016 FT/FT
ALIGNMENT: TANGENT
WEARING SURFACE: 1" MONOLITHIC SURFACE
APPROACH SLABS: AS-1-81 (25'-0" LONG)
LATITUDE:
LONGITUDE:

DESIGN AGENCY
Systems
DESIGNED MSL
CHECKED PJP
DRAWN MTN
REVISED
REVIEWED JRC
DATE 7/18/06
STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 578+48.19
STA. 579+47.54

PRELIMINARY SITE PLAN- ALTERNATIVE 4
BRIDGE NO. SCI-823-XXXX
S.R. 823 OVER BLUE RUN ROAD (CR-29)

SCI-823-0.00
PID 19415



N

0 10 20 40
HORIZONTAL SCALE IN FEET

CALCULATED
CHECKED

BORING PLAN
S.R. 823 OVER BLUE RUN ROAD

SCI-823

APPENDIX II

General Information – Drilling Procedures and Logs of Borings

Legend – Boring Log Terminology

Boring Logs – Six (6) 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:

<u>Term</u>	<u>Relative Moisture or Appearance</u>
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.

<u>Term</u>	<u>Relative Moisture or Appearance</u>
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.**

<u>Term</u>	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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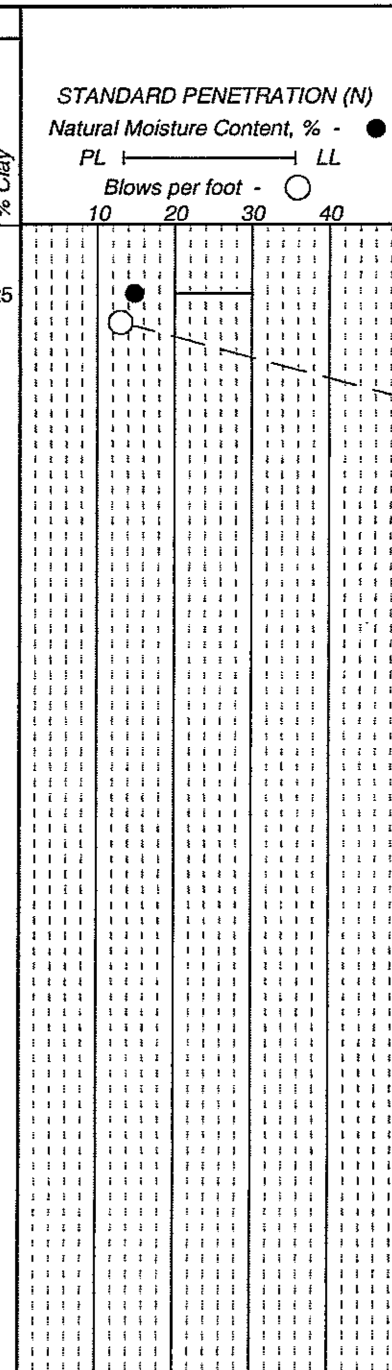
Client: TranSystems, Inc. *Project:* SCI-823-0.00 *Job No.* 0121-3070.03

LOG OF: Boring B-13 *Location:* Sta. 578+27.4, 43.8 ft. LT of SR 823 CL *Date Drilled:* 06/30/06

WATER OBSERVATIONS:
Water seepage at: None observed
Water level at completion: None (Prior to coring rock)
7.0' (Includes core water, measured inside augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	770.0												
0.3	769.7						Topsoil - 4"						
		4 5 8	17	1		3.0	Very stiff brown SILT (A-4b), some clay, trace fine sand; damp.	0	3	--	3	68	25
3.5	766.5	28 50/2	7	2			Severely weathered gray SANDSTONE.						
5.0	765.0						Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, laminated to thinly bedded, moderately fractured.						
10		Core 120"	Rec 114"	RQD 76%	R-1		@ 10.7', qu=11,952 psi.						
15.0	755.0						Bottom of Boring - 15.0'						
20													
25													
30													



FILE: 0121-3070-03 [1/18/2007 3:37 PM]

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

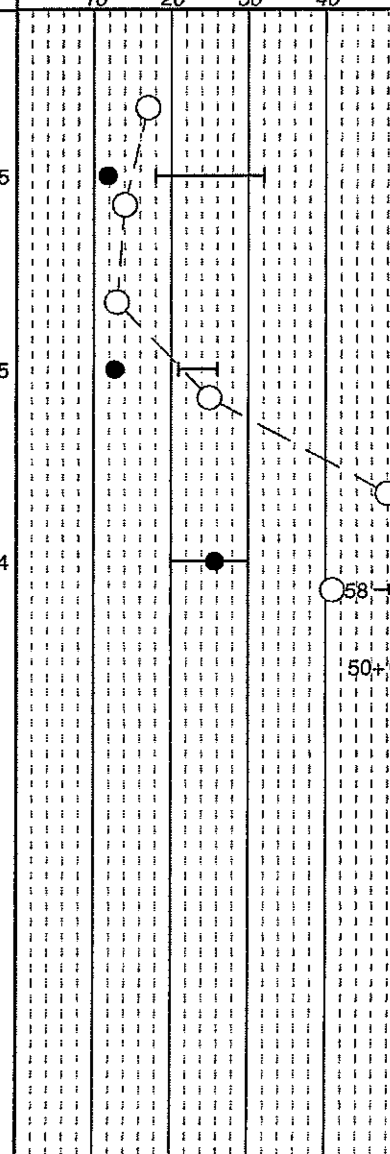
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-14 | *Location:* Sta. 579+80.2, 31.8 ft. RT of SR 823 CL | *Date Drilled:* 06/30/06

WATER OBSERVATIONS:
Water seepage at: None Observed
Water level at completion: None (Prior to Coring Rock)
13.5' (Includes Core Water, Measured Inside Augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.1	801.7 / 801.6						Topsoil - 1"						
		7 9 8	15	1		4.5+	Hard brown SILT AND CLAY (A-6a), little fine to coarse sand, trace coarse sand; damp.						
5		7 8 6	14	2		4.5+		10	8	--	8	49	25
		9 6 7	3	3		4.5+	@ 8.0', Some gravel (Rock Fragments).						
8.5 / 10	793.2	6 8 17	13	4		4.0	Very stiff to hard brown SANDY SILT (A-4a), little gravel, little clay; damp.	16	12	--	11	45	15
		18 21 27	18	5		4.5+							
14.0 / 15	787.7	7 18 40	18	6		4.5+	Hard mottled brown and gray SILT (A-4b), trace fine to coarse sand, some clay; moist.	0	2	--	5	70	24
		21 50/3	8	7		4.5+							
18.5 / 20	783.2	Core 120"	Rec 116"	RQD 61%	R-1		Medium hard to hard brown SANDSTONE; fine to medium grained, moderately weathered, argillaceous, laminated to thinly bedded, moderately fractured. @ 20.0', 21.4', iron stained, high angle fractures.						
21.5 / 25	780.2						Medium hard to hard gray SANDSTONE interbedded with siltstone; very fine to fine grained, moderately weathered, pyritic (holos), arenaceous, thinly bedded, highly to moderatley fractured. @ 26.8' to 27.3', qu=5,840 psi, Er=627,457 psi.						
28.5 / 30	773.2						Bottom of Boring - 28.5'						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○
10 20 30 40



FILE: 0121-3070-03 | 1/18/2007 3:37 PM

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

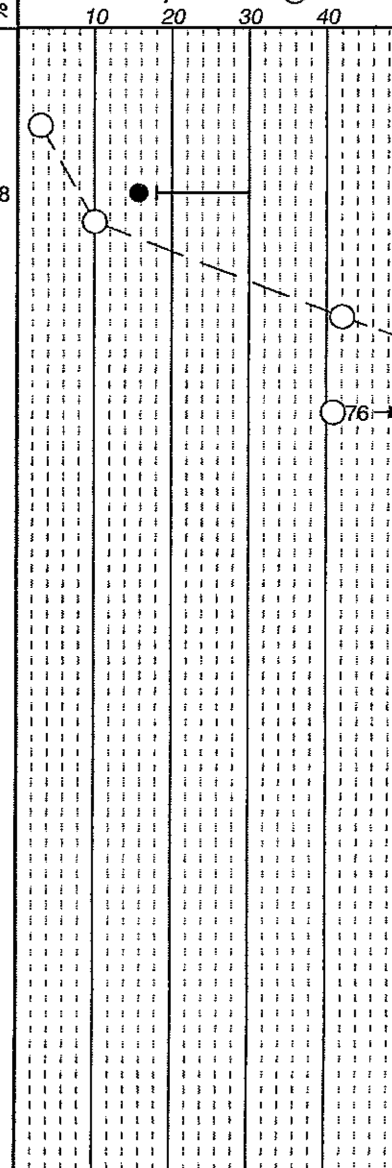
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-7 | Location: Sta. 580+57.2, 44.9 ft. RT of SR 823 CL | Date Drilled: 03/15/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
4.1' (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
0.2	814.3 / 814.1						Topsoil - 2"						
		1 1 2	5	1			Soft dark gray SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; organic; damp.						
3.0	811.3	4 5 5	17	2		2.0	Stiff light brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; (contains relic rock structure); damp.	3	14	--	7	48	28
6.5	807.8	8 20 22	18	3			Severely weathered light brown SANDSTONE.						
		18 30 46	12	4		4.5+							
10.0	804.3	Core 54"	Rec 52"	RQD 17%	R-1		Soft light brown SANDSTONE; highly weathered to decomposed, very fine grained, thinly laminated to thinly bedded, highly fractured, contains several healed fractures.						
13.0	801.3						Soft to medium hard gray SANDSTONE; highly weathered, very fine to fine grained, thinly laminated, argillaceous, highly fractured.						
18.0	796.3	Core 120"	Rec 120"	RQD 60%	R-2		Medium hard gray SANDSTONE; highly to moderately weathered, argillaceous, micacious, thinly laminated to medium bedded, slightly fractured. @ 21.0' to 21.2', Decomposed.						
24.5	789.8						Bottom of Boring - 24.5'						

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



FILE: 0121-3070-03 [1/18/2007 3:49 PM]

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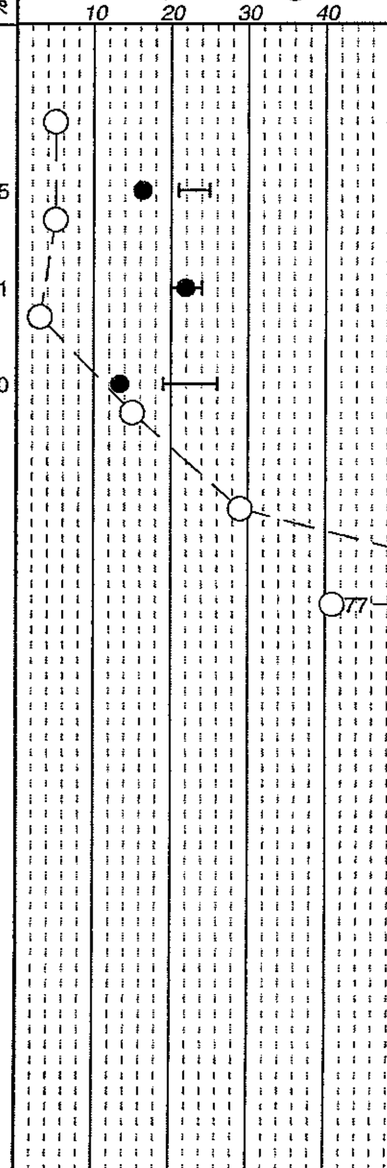
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-8 | Location: Sta. 579+73.0, 46.7 ft. LT of SR 823 CL | Date Drilled: 03/11/05 to 03/14/05

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	802.4												
0.1	802.3						Topsoil - 1"						
		2 3 2	14	1		2.0	Stiff dark gray SANDY SILT (A-4a), little clay, some gravel, contains organic material; damp.						
5		2 2 3	11	2		1.0		22	10	--	7	46	15
6.5	795.9	1 1 2	9	3		1.5	Stiff to very stiff brown SILT (A-4b), little to some clay, little to some fine to coarse sand, trace gravel; moist.	1	11	--	7	60	21
10		3 7 8	10	4		2.0		5	13	--	12	50	20
		4 10 19	18	5		3.5							
13.5	788.9	13 31 46	18	6		4.5	Severely weathered light brown SANDSTONE.						
15													
16.0	786.4						Soft light brown SANDSTONE; highly weathered to decomposed, highly fractured.						
19.3	783.1	Core 114"	Rec 107"	RQD 46%	R-1		Soft to medium hard gray SANDSTONE; very fine grained, highly weathered, micaceous, argillaceous, thinly laminated to thinly bedded, highly fractured, contains ferric bands.						
20													
25							@ 27.7' to 27.9', Decomposed Zone.						
30		Core	Rec	RQD	R-2								

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



FILE: 0121-3070-03 [1/18/2007 3:49 PM]

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

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

LOG OF: Boring TR-8 | Location: Sta. 579+73.0, 46.7 ft. LT of SR 823 CL | Date Drilled: 03/11/05 to 03/14/05

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	772.4													
30.5	771.9	120"	120"	81%			Soft to medium hard gray SANDSTONE; very fine grained, highly weathered, micaceous, argillaceous, thinly laminated to thinly bedded, highly fractured, contains ferric bands.							
34.5	767.9						Bottom of Boring - 34.5'							

STANDARD PENETRATION (N): Natural Moisture Content, % - ●; PL ├──┤ LL; Blows per foot - ○; scale 10, 20, 30, 40

Depth scale: 35, 40, 45, 50, 55, 60

FILE: 0121-3070-03 [1/18/2007 3:49 PM]

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

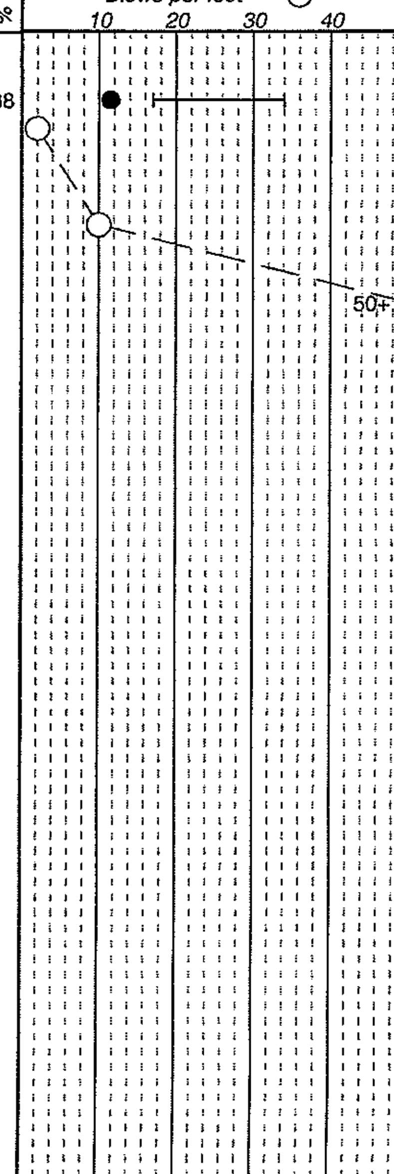
LOG OF: Boring TR-9 | Location: Sta. 578+73.6, 39.1 ft. RT of SR 823 CL | Date Drilled: 03/15/05

WATER OBSERVATIONS: Water seepage at: 1.0'
Water level at completion: None (prior to coring)
3.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	772.8												
0.3	772.5						Topsoil - 3"						
		WOH WOH 2	7	1		0.25	Soft dark brown SILTY CLAY (A-6b), trace to little fine to coarse sand; contains shale fragments; damp to moist.	0	5	--	5	52	38
3.0	769.8	4 5 5	18	2		2.5	Very stiff brown SANDY SILT (A-4a), trace clay, trace gravel; damp.						
6.0	766.8	19 50/3	8	3		4.5+	Severely weathered light brown SANDSTONE; argillaceous.						
7.0	765.8	Core 120"	Rec 120"	RQD 83%	R-1		Medium hard gray SANDSTONE; slightly weathered, micaceous, argillaceous, massive bedding, slightly fractured. @ 7.0' to 7.3', Broken.						
17.0	755.8						Bottom of Boring - 17.0'						

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

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



FILE: 0121-3070-03 [1/18/2007 3:51 PM]

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

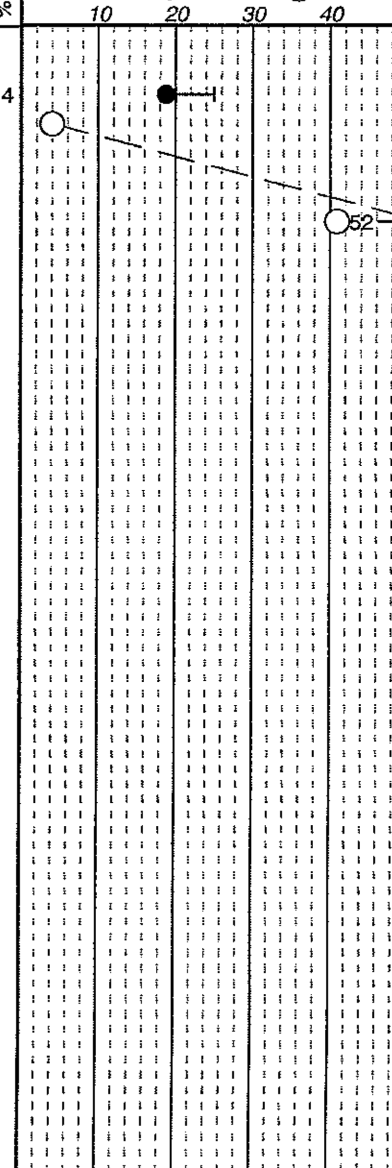
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-10 | *Location:* Sta. 577+84.9, 51.8 ft. LT of SR 823 CL | *Date Drilled:* 03/15/05

WATER OBSERVATIONS: Water seepage at: 1.0'
Water level at completion: None (prior to coring)
1.3' (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
0	768.1												
0.3	767.8						Topsoil - 3"						
		1 2 2	8	1		1.0	Medium stiff dark brown SANDY SILT (A-4a), some gravel, little clay; damp to moist.	29	12	--	7	38	14
3.0	765.1	12 11 41	18	2			Severely weathered light grayish brown SANDSTONE.						
5.0	763.1						Medium hard light brown SANDSTONE; highly weathered, thickly bedded, broken, contains high angle healed fractures.						
7.1	761.0	Core 54"	Rec 54"	RQD 33%	R-1		Medium hard to hard gray SANDSTONE; slightly to moderately weathered, micaceous, argillaceous, massive, moderately to slightly fractured.						
		Core 120"	Rec 120"	RQD 87%	R-2								
19.5	748.6						Bottom of Boring - 19.5'						

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

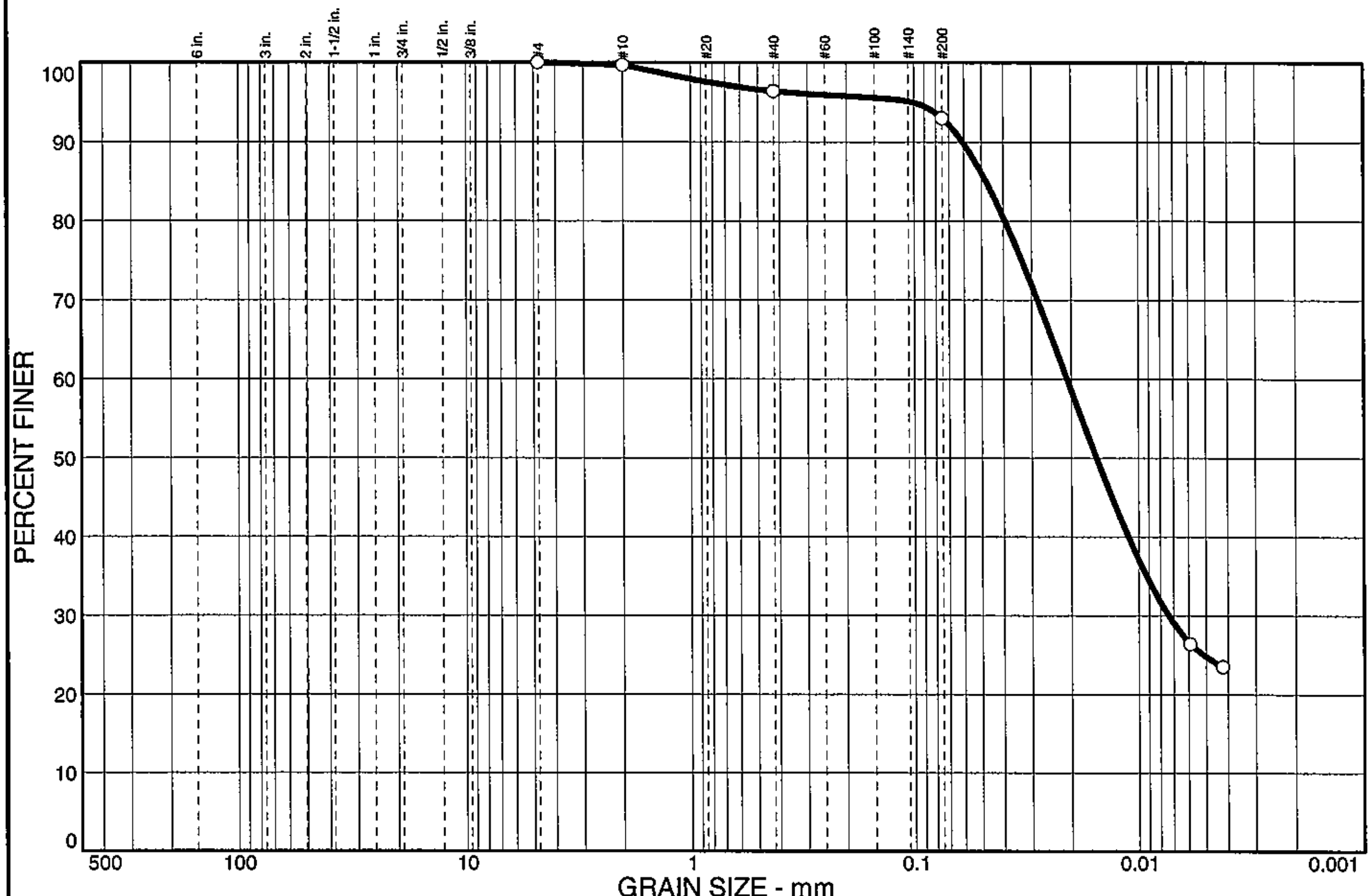


Depth scale: 10, 15, 20, 25, 30

FILE: 0121-3070-03 [1/18/2007 3:51 PM]

APPENDIX III
Laboratory Test Results

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.3	3.3	3.4	68.3	24.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.7		
#40	96.4		
#200	93.0		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 30 PI= 10

Coefficients

D_{85}= 0.0479 D_{60}= 0.0209 D_{50}= 0.0155
D_{30}= 0.0074 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(9)

Remarks

Moisture Content= 14.8%

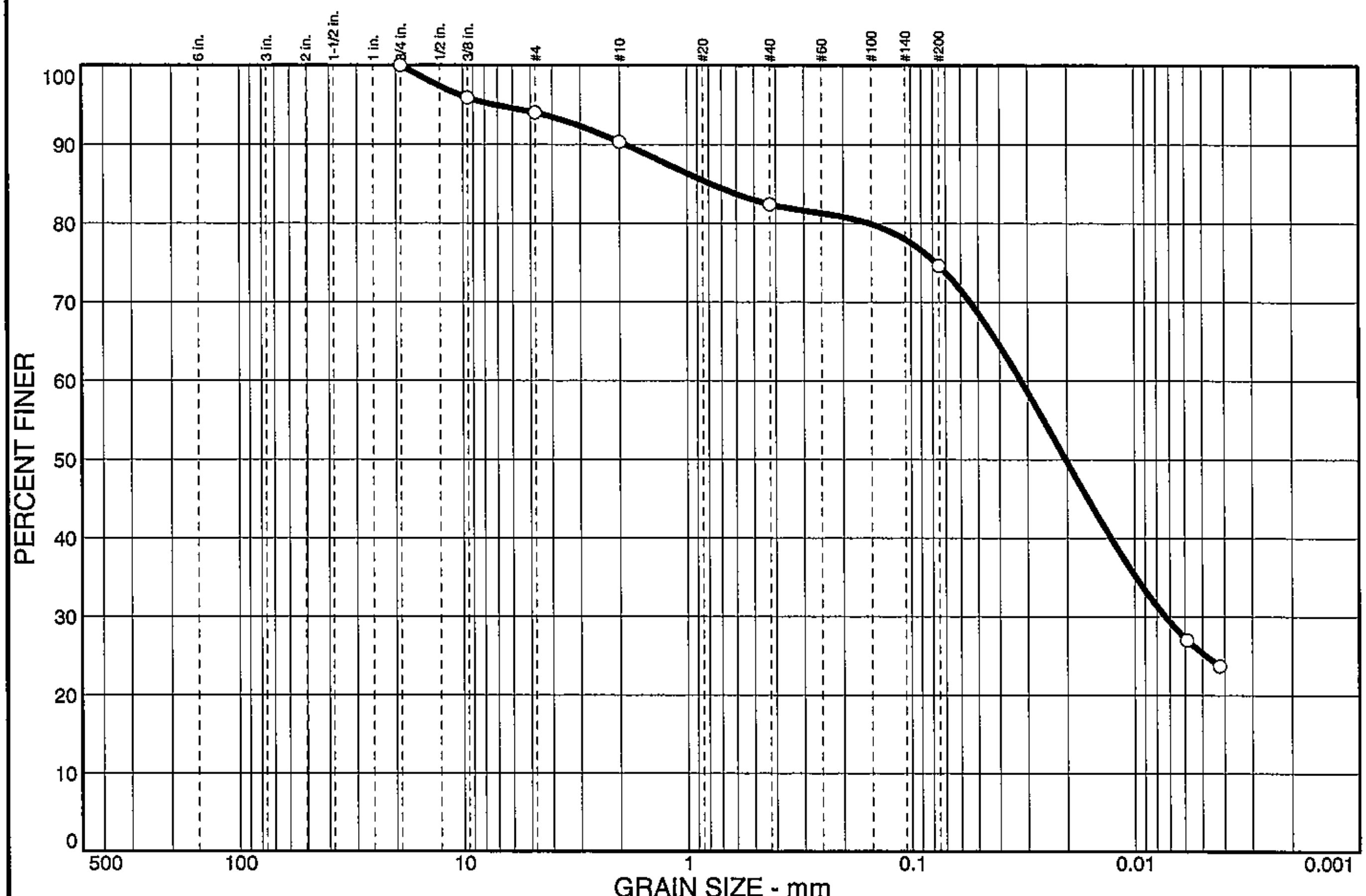
Sample No.: 1 **Source of Sample:** B-13 **Date:** 7/21/06
Location: **Elev./Depth:** 1.0

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	6.0	3.7	7.9	7.8	49.3	25.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	95.9		
#4	94.0		
#10	90.3		
#40	82.4		
#200	74.6		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 18 LL= 32 PI= 14

Coefficients

D_{85}= 0.786 D_{60}= 0.0323 D_{50}= 0.0204
D_{30}= 0.0073 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

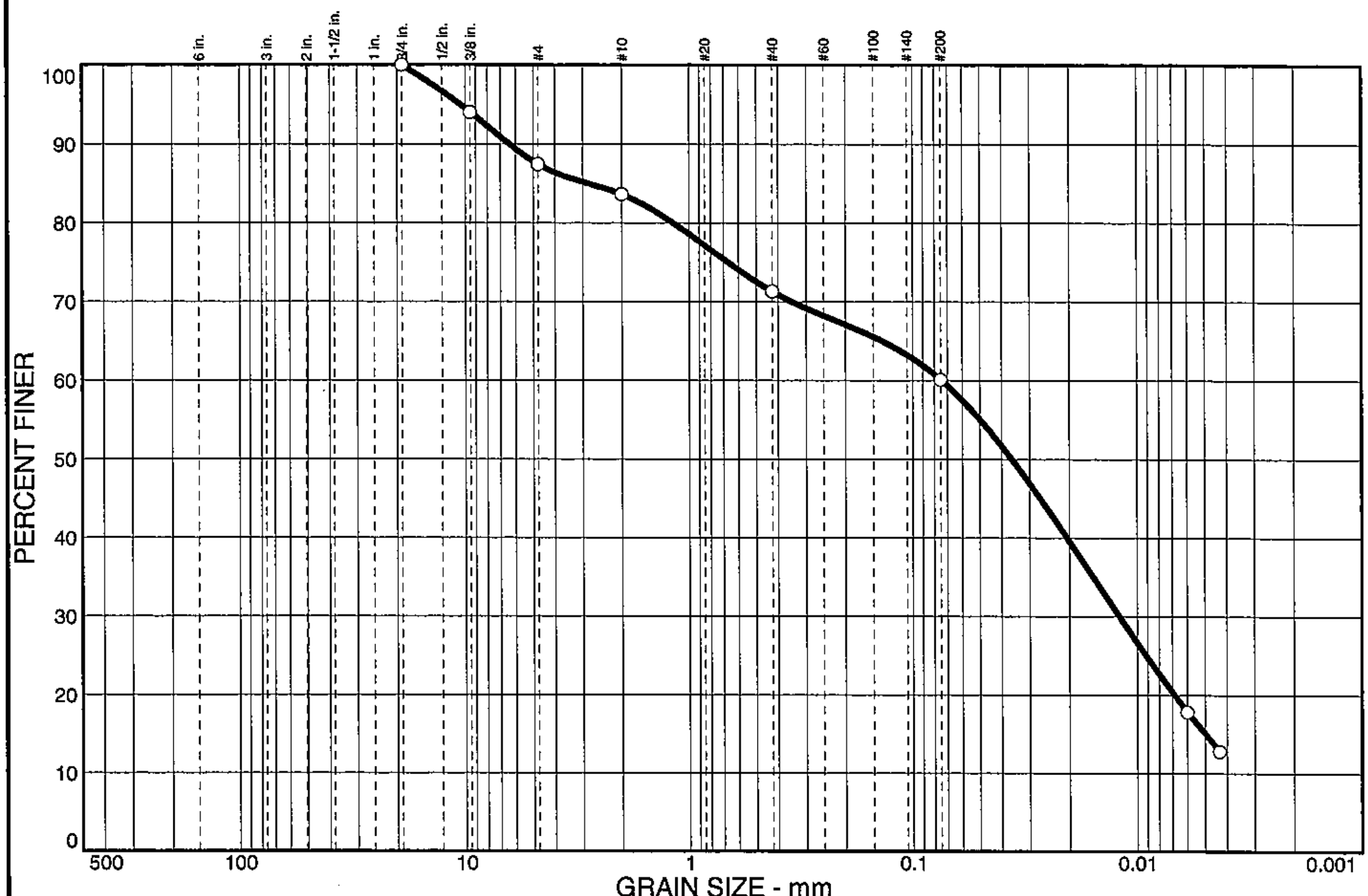
Moisture Content= 11.8%

Sample No.: 2 **Source of Sample:** B-14 **Date:** 7/21/06
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	12.6	3.8	12.3	11.2	45.1	15.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	94.0		
#4	87.4		
#10	83.6		
#40	71.3		
#200	60.1		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 21 LL= 26 PI= 5

Coefficients

D_{85}= 2.84 D_{60}= 0.0743 D_{50}= 0.0362
D_{30}= 0.0120 D_{15}= 0.0050 D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(1)

Remarks

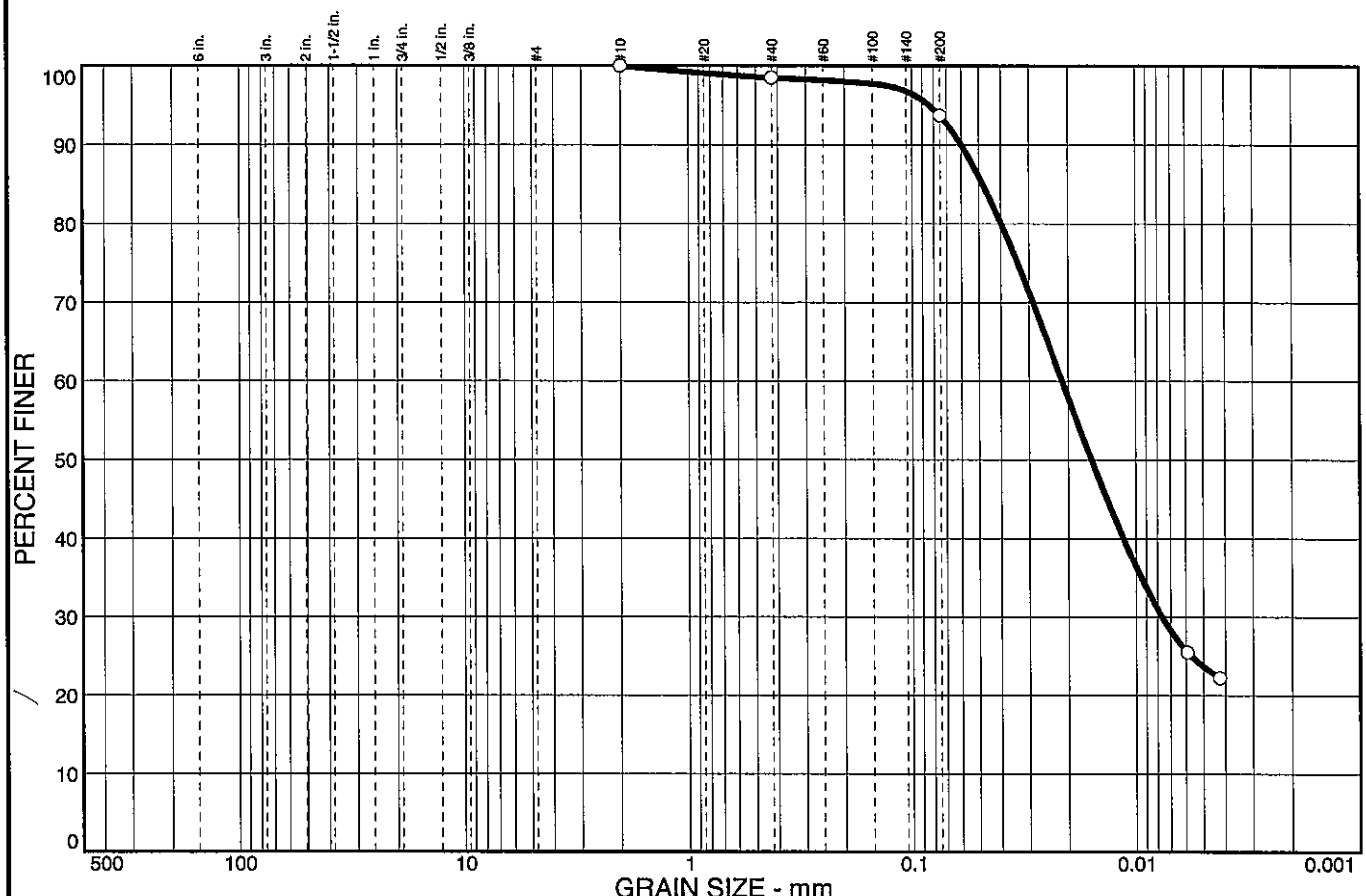
Moisture Content= 12.7%

Sample No.: 4 **Source of Sample:** B-14 **Date:** 7/21/06
Location: **Elev./Depth:** 8.5

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	0.0	0.0	1.5	4.8	70.1	23.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	98.5		
#200	93.7		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 30 PI= 10

Coefficients

D_{85}= 0.0483 D_{60}= 0.0213 D_{50}= 0.0158
D_{30}= 0.0077 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(9)

Remarks

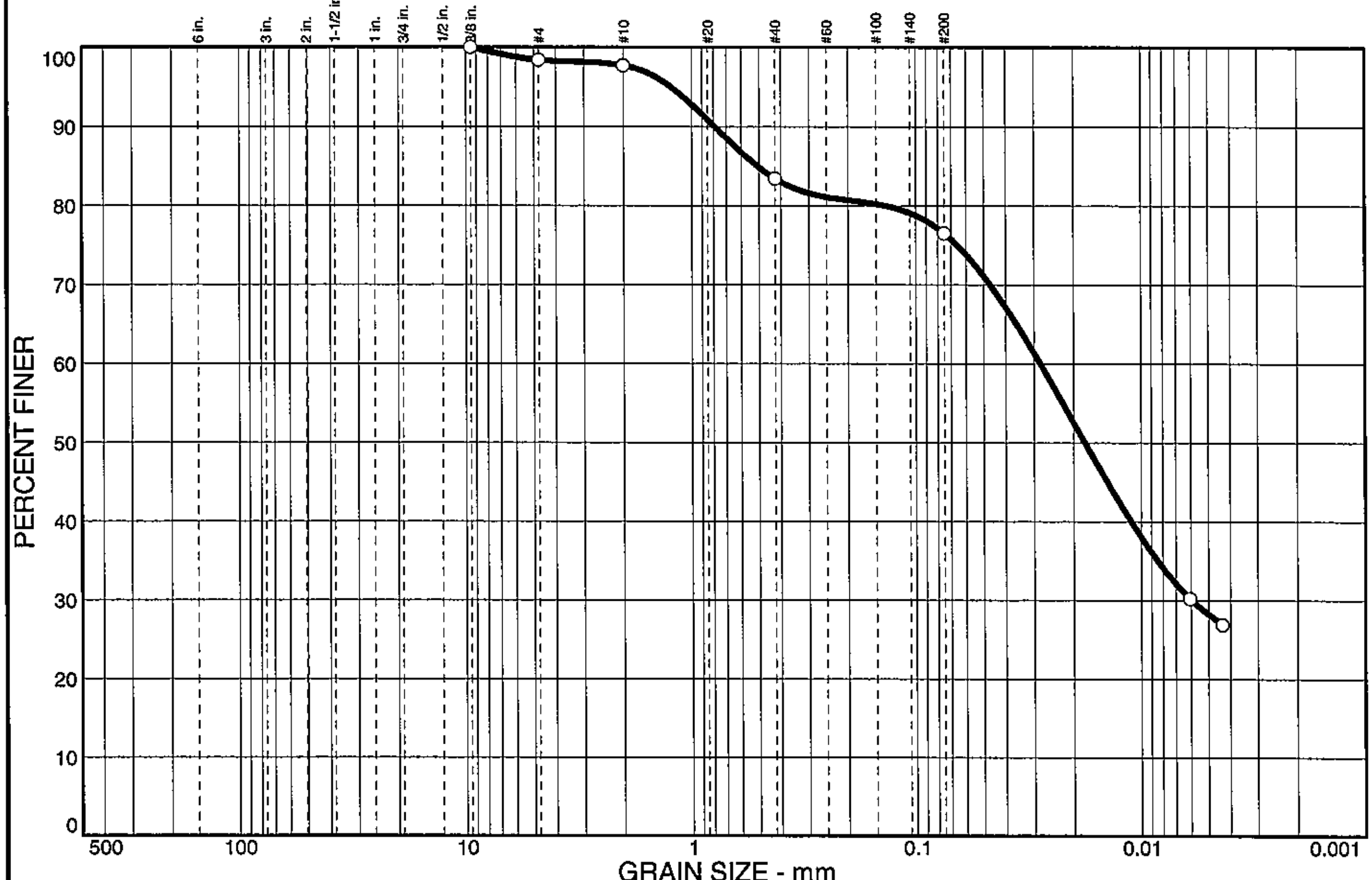
Moisture Content= 25.7%

Sample No.: 6 **Source of Sample:** B-14 **Date:** 7/21/06
Location: **Elev./Depth:** 13.5

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	1.6	0.7	14.3	6.9	48.3	28.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.4		
#10	97.7		
#40	83.4		
#200	76.5		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 18 LL= 30 PI= 12

Coefficients

D_{85}= 0.510 D_{60}= 0.0282 D_{50}= 0.0179
D_{30}= 0.0060 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(7)

Remarks

Moisture Content= 15.7%

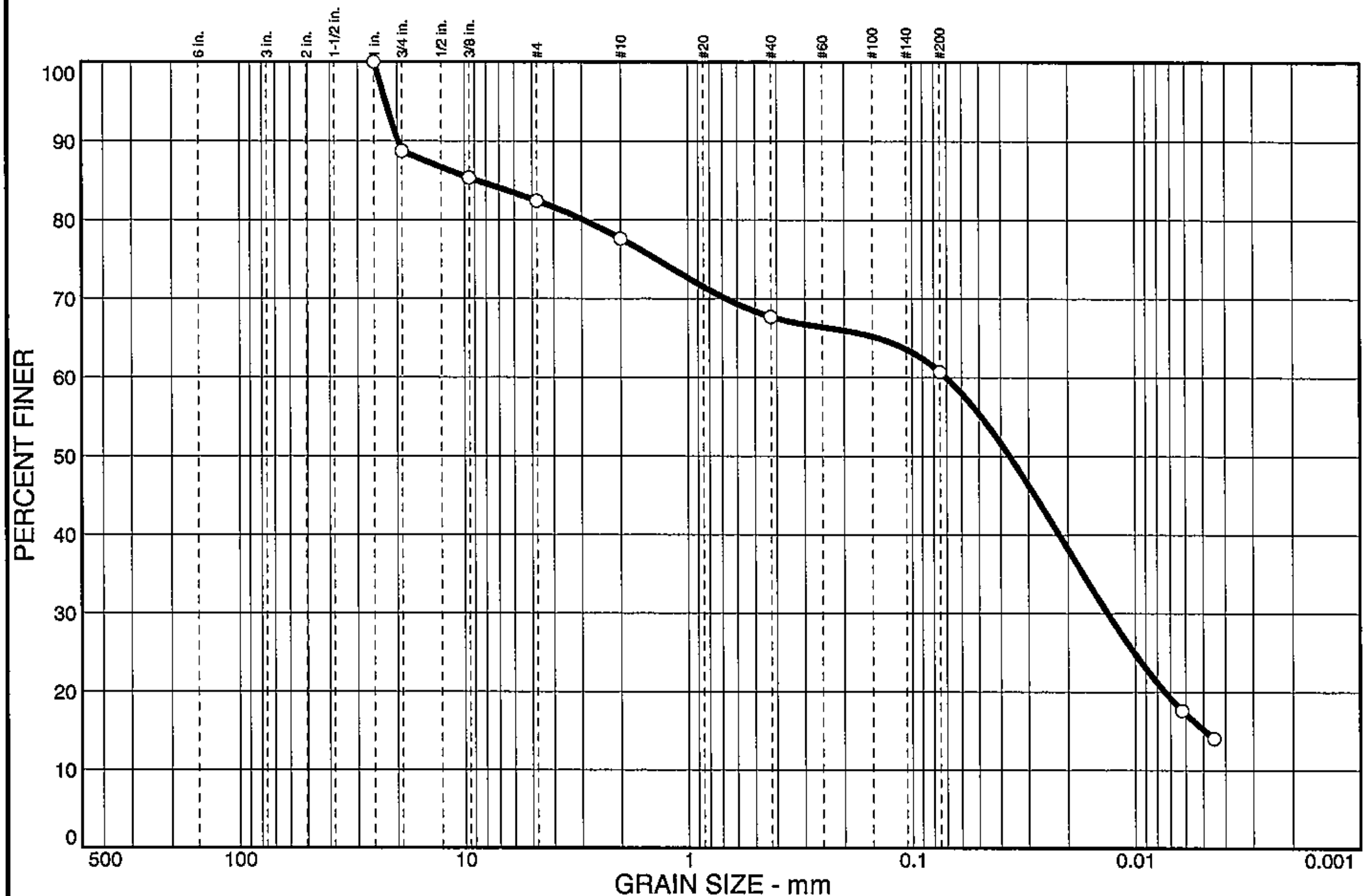
Sample No.: 2 **Source of Sample:** TR-7 **Date:** 4/6/05
Location: **Elev./Depth:** 3.5

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	11.3	6.3	4.8	9.9	7.0	45.6	15.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00 in.	100.0		
0.75 in.	88.7		
0.375 in.	85.3		
#4	82.4		
#10	77.6		
#40	67.7		
#200	60.7		

* (no specification provided)

Soil Description

Sandy silty clay with gravel

Atterberg Limits

PL= 21 LL= 25 PI= 4

Coefficients

D_{85}= 8.86 D_{60}= 0.0704 D_{50}= 0.0366
D_{30}= 0.0133 D_{15}= 0.0049 D_{10}=
C_u= C_c=

Classification

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

Remarks

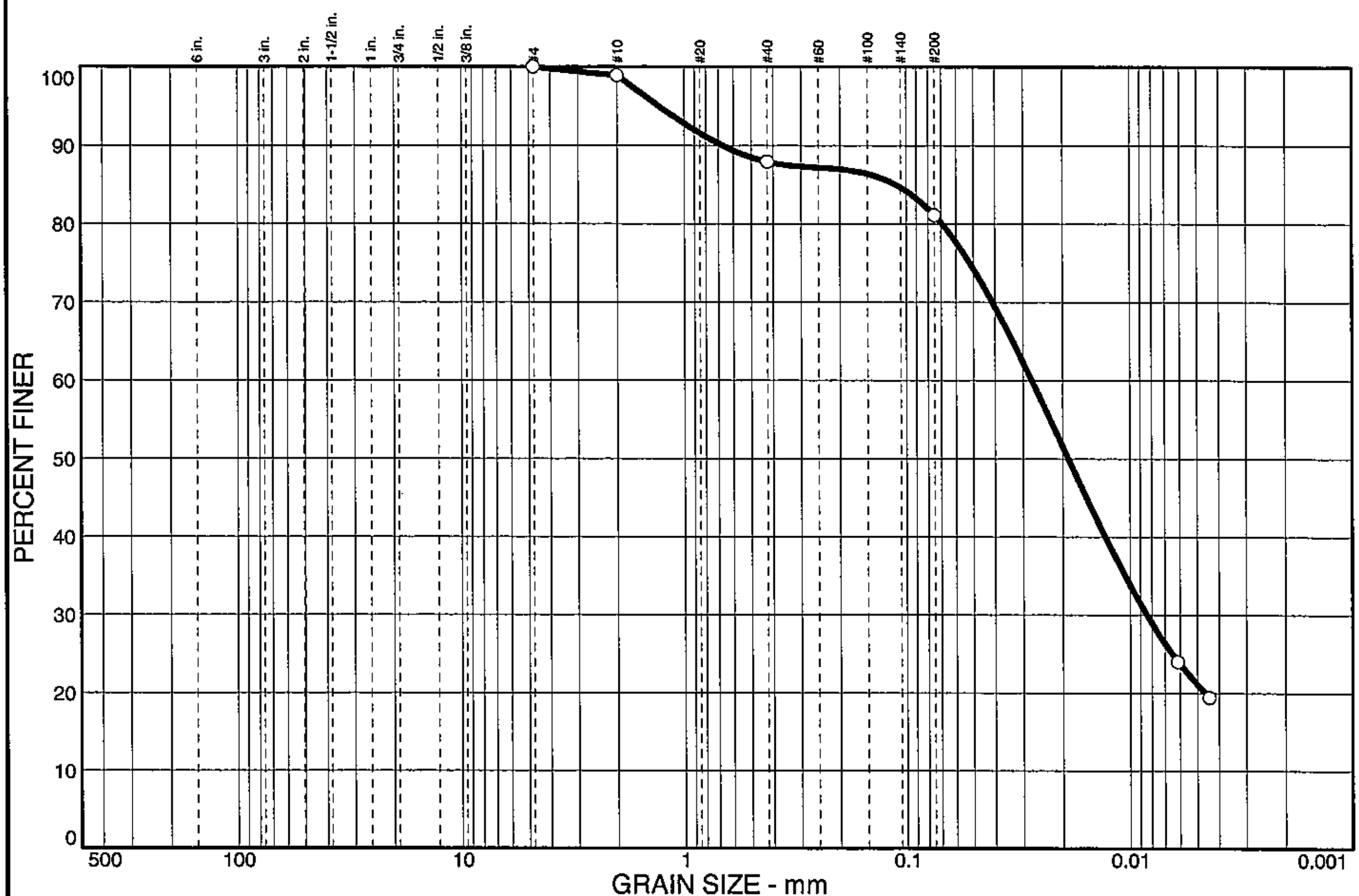
Moisture Content= 16.3%

Sample No.: 2 **Source of Sample:** TR-8 **Date:** 4/7/05
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	0.0	1.1	11.0	6.8	60.1	21.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.9		
#40	87.9		
#200	81.1		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 20 LL= 24 PI= 4

Coefficients

D_{85}= 0.112 D_{60}= 0.0275 D_{50}= 0.0189
D_{30}= 0.0085 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(2)

Remarks

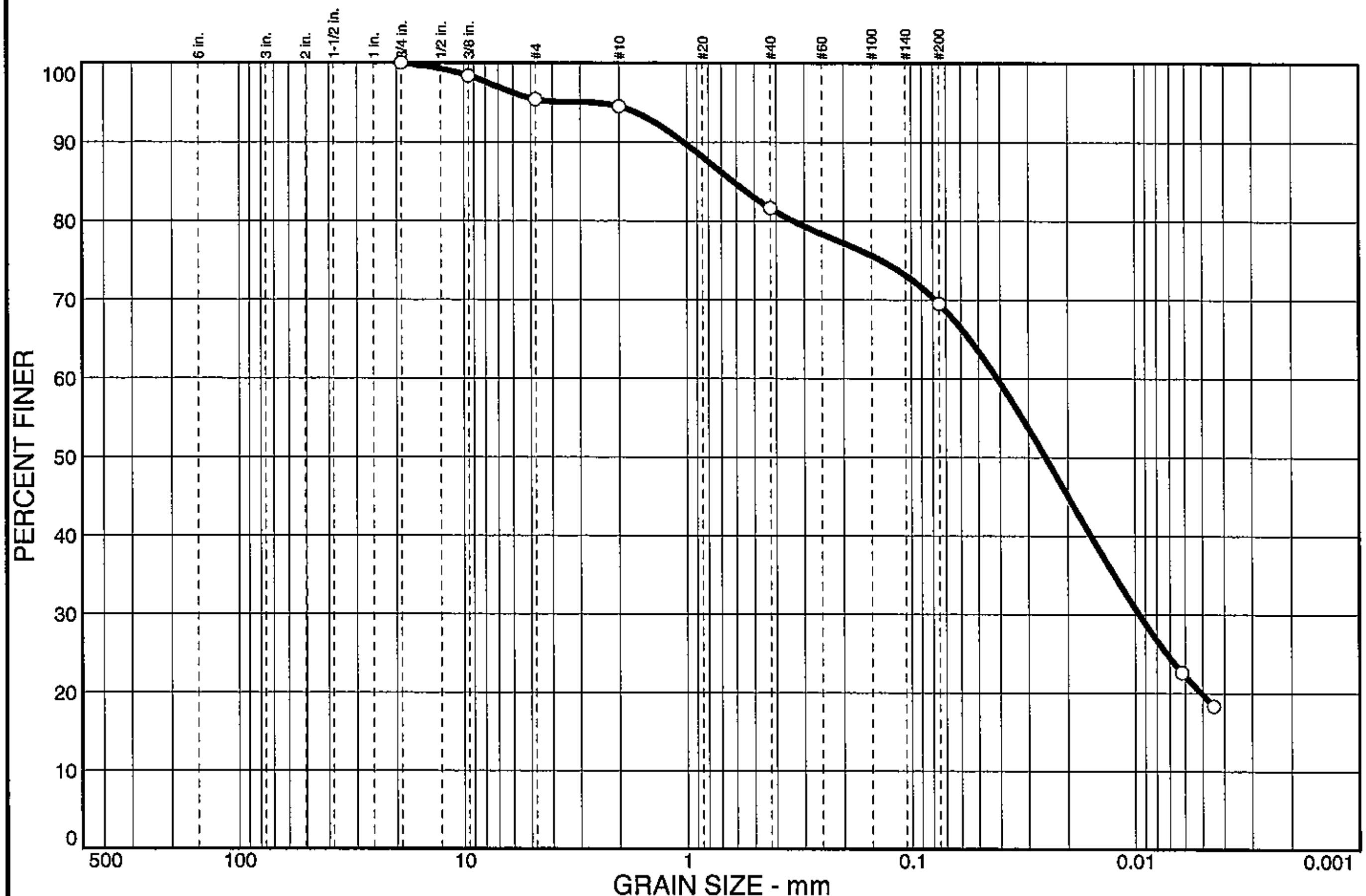
Moisture Content= 21.9%

Sample No.: 3 **Source of Sample:** TR-8 **Date:** 4/7/05
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	4.6	0.9	12.9	12.1	49.7	19.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.4		
#4	95.4		
#10	94.5		
#40	81.6		
#200	69.5		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 19 LL= 26 PI= 7

Coefficients

D_{85}= 0.623 D_{60}= 0.0414 D_{50}= 0.0252
D_{30}= 0.0096 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

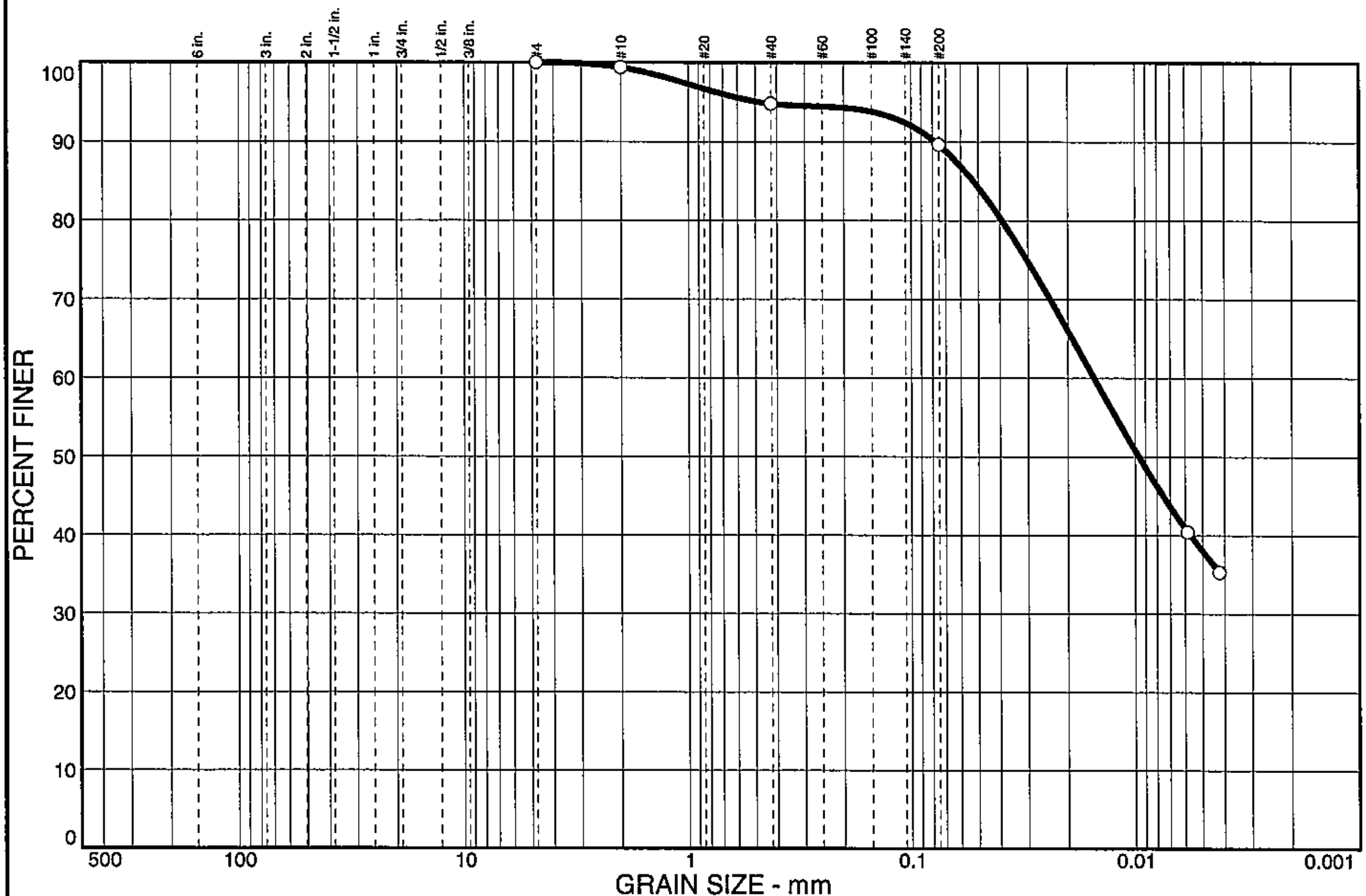
Moisture Content= 13.4%

Sample No.: 4 **Source of Sample:** TR-8 **Date:** 4/7/05
Location: **Elev./Depth:** 8.5

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	0.0	0.6	4.6	5.2	51.7	37.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.4		
#40	94.8		
#200	89.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 17 LL= 34 PI= 17

Coefficients

D_{85}= 0.0528 D_{60}= 0.0153 D_{50}= 0.0097
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(15)

Remarks

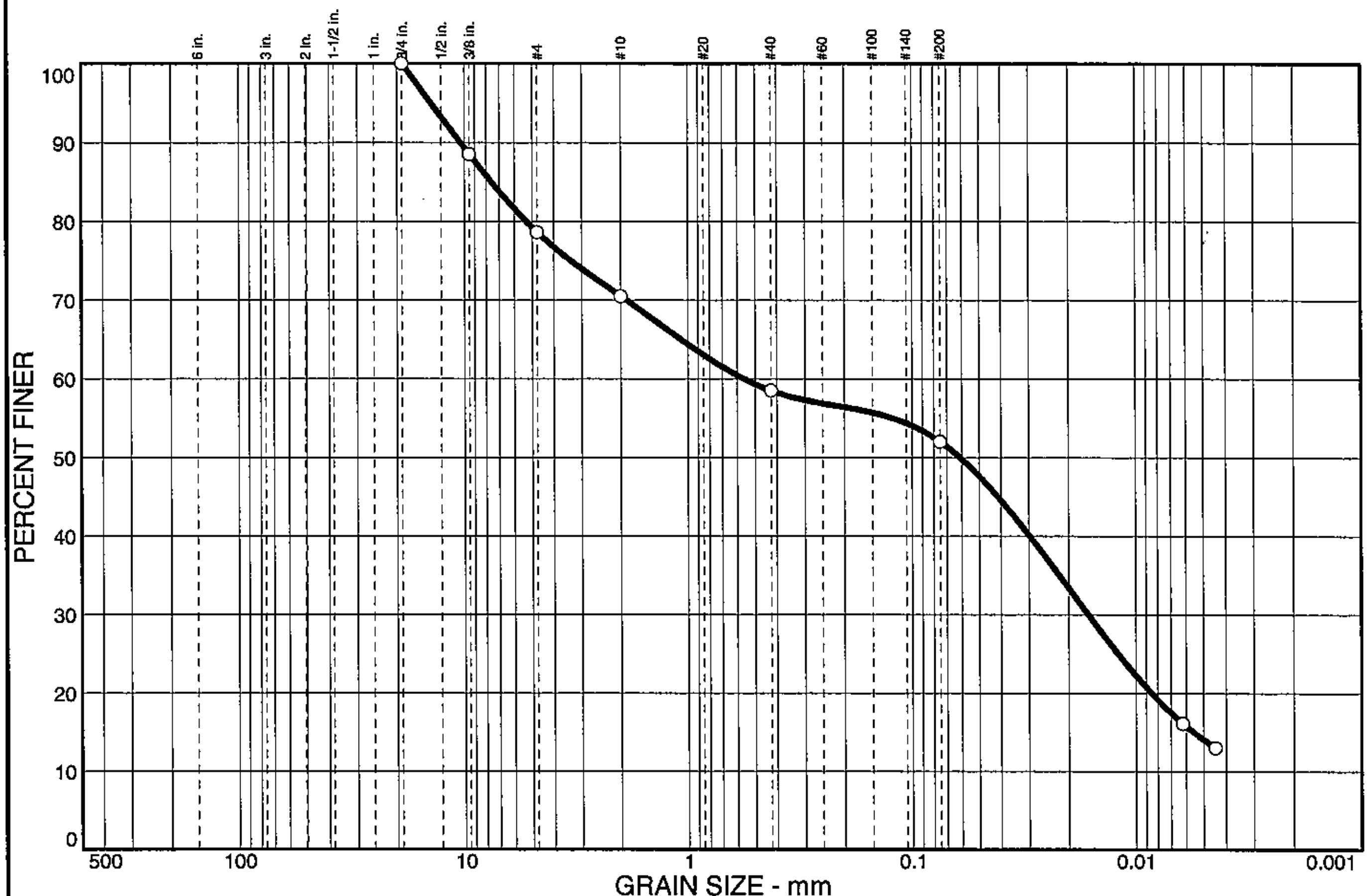
Moisture Content= 11.5%

Sample No.: 1 **Source of Sample:** TR-9 **Date:** 4/7/05
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	21.4	8.1	12.0	6.5	38.0	14.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	88.5		
#4	78.6		
#10	70.5		
#40	58.5		
#200	52.0		

* (no specification provided)

Soil Description

Sandy silty clay with gravel

Atterberg Limits

PL= 20 LL= 25 PI= 5

Coefficients

D_{85}= 7.61 D_{60}= 0.561 D_{50}= 0.0613
D_{30}= 0.0164 D_{15}= 0.0056 D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 18.8%

Sample No.: 1 **Source of Sample:** TR-10 **Date:** 4/7/05
Location: **Elev./Depth:** 1.0

DLZ

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

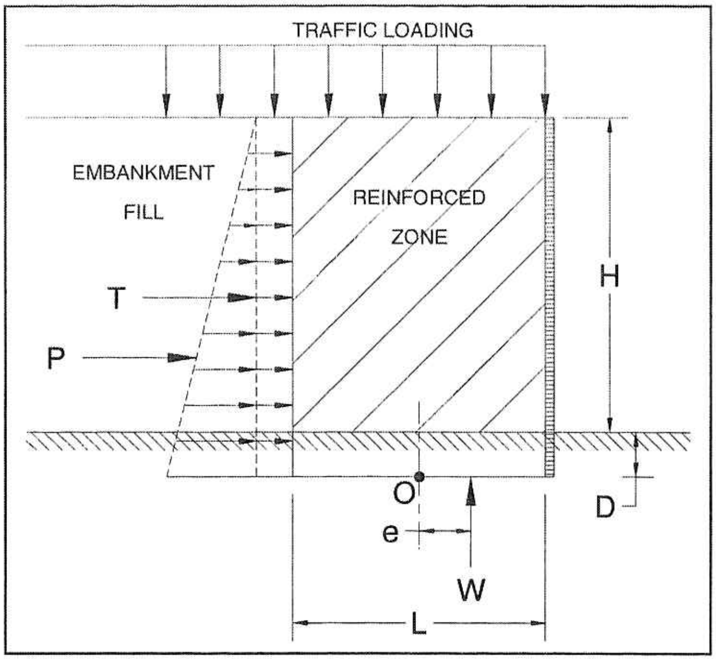
Project No: 0121-3070.03 **Figure**

APPENDIX IV

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

BEARING CAPACITY OF A MSE WALL

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



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
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	47.3	ft	Height of wall
H+D	=	50	ft	
L factor	=	1		Length factor-range (0.7 - 1.0)
L=B	=	50	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	16.667	ft	Moment arm
Γ Wt	=	25	ft	Moment arm
B'	=	44.08	ft	
γ'	=	57.6	pcf	
W_t		12,000	lb/ft of wall	Weight from traffic
W_{mse}	=	300,000	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 7,078 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 = **0.75** **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} = 27,393 psf

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

q_{ALL} = 10,957 psf

Factor of Safety = **3.87** **OK**

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 = 2.96 ft

Kern

e < L/6 = 8.33 ft

DLZ	SUBJECT			
	Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
	Project	Blue Run Road	SHEET NO.	2 OF 9
	Item	MSE Wall Stability (Rear Abutment)/Sta. 578+55.2	COMP. BY WMA	DATE 01/09/07
	Borings TR-09, TR-10, and B-13		CHECKED BY SMK	DATE 1-18-07
	Natural Soil Foundation		Assumes Pile Supported Abutments	

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=47.3'
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	=	50	feet
γ_{mse}	=	120	pcf
L	=	50	feet
L factor	=	1.00	
ϕ	=	30	deg

Foundational Soil Properties

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

Length factor-range (0.7 - 1.0)

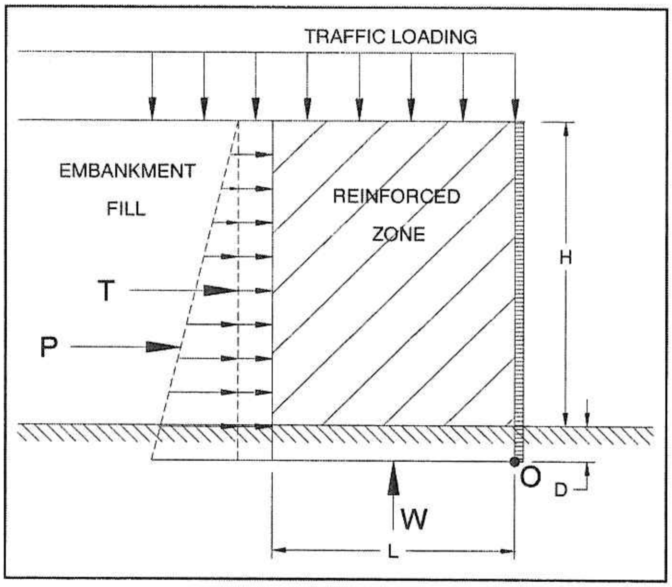
Friction Angle of Embankment Fill

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 = 53,460 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 {ODOT, Bridge Design Manual, 303.4.1.1}

P_r = 105,000 lbs per foot of wall

Use Undrained Value

$P_r = L(c)$ (Undrained)

P_r = 50,000 lbs per foot of wall

USE THIS VALUE

$FS = \frac{P_r}{P_a}$ Calculated: **FS = 0.94** 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

$\Sigma M_{resisting}$ = 7,500,000 lb-ft $\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 924,000 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated: **FS = 8.12** Required: FS = 2.00

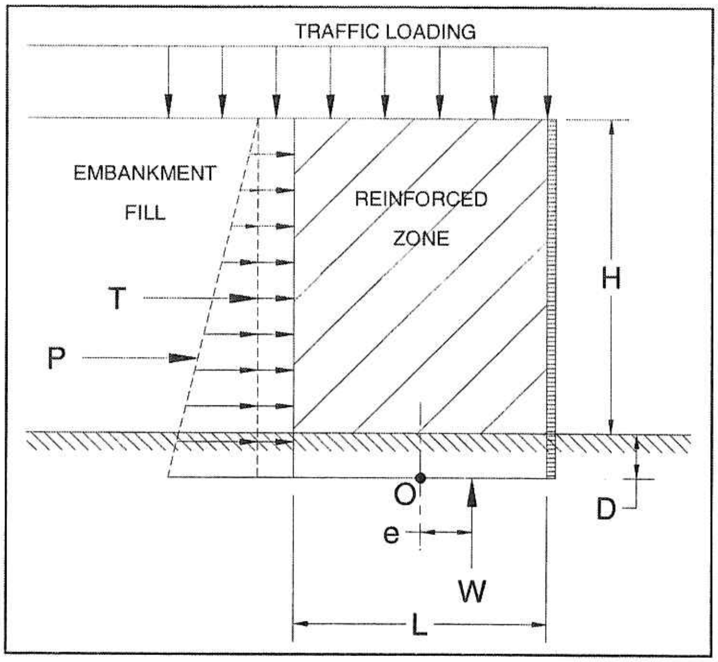
Resistance Against Overturning is **OK**

DLZ	SUBJECT	Client	TranSystems	JOB NUMBER	0121-3070.03		
		Project	Blue Run Road	SHEET NO.	3	OF	9
		Item	Bearing Capacity (Rear Abutment)/Sta. 578+55.2	COMP. BY	WMA	DATE	1/9/07
		Borings TR-09, TR-10, and B-13		CHECKED BY	SAK	DATE	1-18-07

Compacted Granular Fill Foundation — Assumes Pile Supported Abutments

BEARING CAPACITY OF A MSE WALL

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



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
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	47.3	ft	Height of wall
H+D	=	50	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	35	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	16.667	ft	Moment arm
Γ Wt	=	25	ft	Moment arm
B'	=	26.54	ft	
γ'	=	57.6	pcf	
W_t		8,400	lb/ft of wall	Weight from traffic
W_{mse}	=	210,000	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 8,229 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 36,472 psf

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

q_{ALL} = 14,589 psf

Factor of Safety = **4.43** **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} = 36,472 psf

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

q_{ALL} = 14,589 psf

Factor of Safety = **4.43** **OK**

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 = 4.23 ft

Kern

e < L/6 = 5.83 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	Blue Run Road	SHEET NO.	4 OF 9
Item	MSE Wall Stability (Rear Abutment)/Sta. 578+55.2	COMP. BY	WMA DATE 01/09/07
Borings TR-09, TR-10, and B-13		CHECKED BY	SJK DATE 1-18-07
Compacted Granular Fill Foundation	Assumes Pile Supported Abutments		

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=47.3'
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 = 50 feet
γ_{mse} = 120 pcf
L = 35 feet
L factor = 0.70
ϕ = 30 deg

Foundational Soil Properties

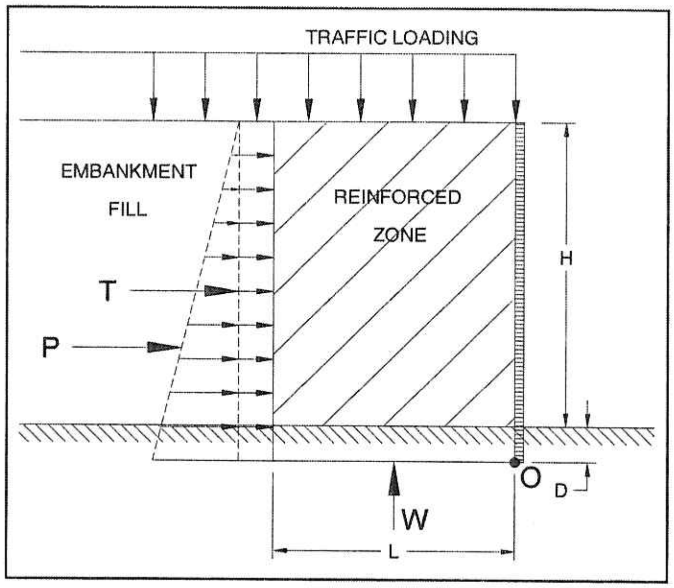
c = 0 psf Cohesion
ϕ' = 34 deg Friction angle
ω_T = 240 psf Traffic loading
Length factor-range (0.7 - 1.0)
Friction Angle of Embankment Fill

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 = 53,460 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 {ODOT, Bridge Design Manual, 303.4.1.1}

P_r = 94,500 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 = **1.77** 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

$\Sigma M_{resisting}$ = 3,675,000 lb-ft

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

$\Sigma M_{overturning}$ = 924,000 lb-ft

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

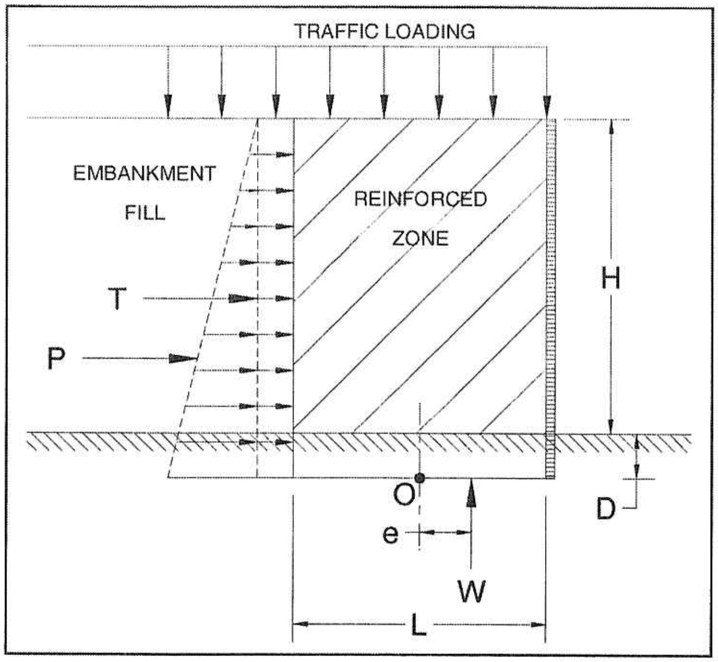
$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

Calculated: FS = **3.98** Required: FS = 2.00

Resistance Against Overturning is **OK**

BEARING CAPACITY OF A MSE WALL

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



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	=	1500	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
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	31	ft	Height of wall
H+D	=	34	ft	
L factor	=	1		Length factor-range (0.7 - 1.0)
L=B	=	34	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	11.333	ft	Moment arm
Γ Wt	=	17	ft	Moment arm
B'	=	29.84	ft	
γ'	=	57.6	pcf	
W_t		8,160	lb/ft of wall	Weight from traffic
W_{mse}	=	138,720	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,922 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 7,883 psf

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

q_{ALL} = 3,153 psf

Factor of Safety = **1.60** **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} = 19,461 psf

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

q_{ALL} = 7,784 psf

Factor of Safety = **3.95** **OK**

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 = 2.08 ft

Kern

e < L/6 = 5.67 ft

DLZ

SUBJECT			
Client	TranSystems	JOB NUMBER	0121-3070.03
Project	Blue Run Road	SHEET NO.	6 OF 9
Item	MSE Wall Stability (Forward Abutment)	COMP. BY	WMA DATE 01/09/07
Sta. 579+40.5 /Borings TR-07,TR-08 and B-14		CHECKED BY	SJK DATE 1-18-07
Natural Soil Foundation	Assumes Pile Supported Abutments		

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=31'
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	=	34	feet
γ_{mse}	=	120	pcf
L	=	34	feet
L factor	=	1.00	
ϕ	=	30	deg

Foundational Soil Properties

c	=	1500	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 = 25,582 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 {ODOT, Bridge Design Manual, 303.4.1.1}

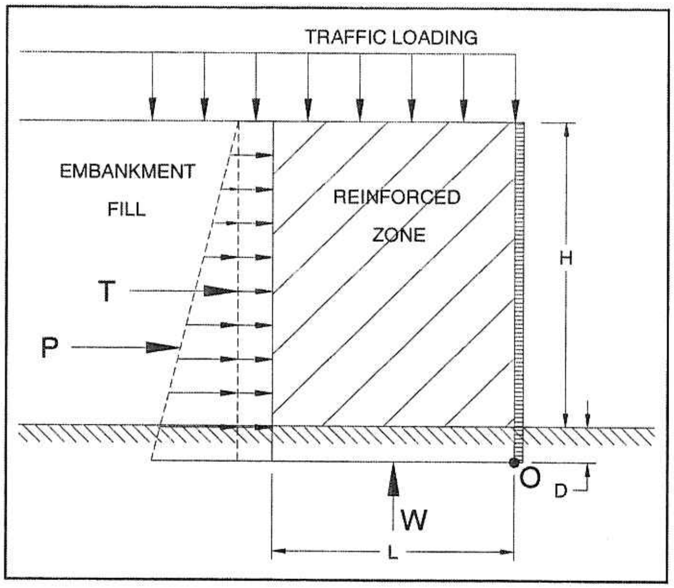
P_r = 48,552 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 51,000 lbs per foot of wall

Use Drained Value



$FS = \frac{P_r}{P_a}$ Calculated $\boxed{FS = 1.90}$ 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

$\Sigma M_{resisting}$ = 2,358,240 lb-ft

$\Sigma M_{overturning}$ = 305,184 lb-ft

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

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

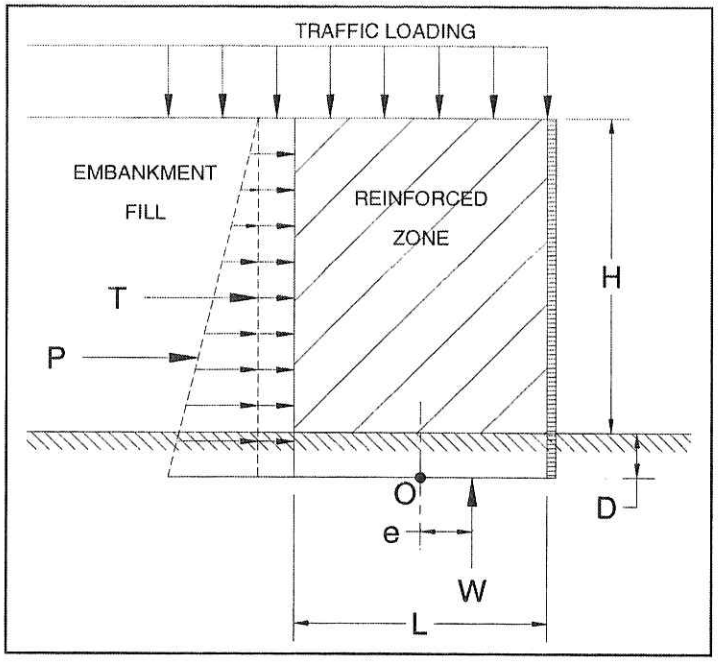
$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated $\boxed{FS = 7.73}$ Required FS = 2.00 Resistance Against Overturning is **OK**

DLZ	SUBJECT	Client	TranSystems	JOB NUMBER	0121-3070.03		
		Project	Blue Run Road	SHEET NO.	7	OF	9
		Item	Bearing Capacity (Forward Abutment)	COMP. BY	WMA	DATE	1/9/07
		Sta. 579+40.5/Borings TR-07,TR-08 and B-14		CHECKED BY	SJK	DATE	1-18-07

Compacted Granular Fill Foundation — Assumes Pile Supported Abutments

BEARING CAPACITY OF A MSE WALL

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



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
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	31	ft	Height of wall
H+D	=	34	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	24	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	11.333	ft	Moment arm
Γ Wt	=	17	ft	Moment arm
B'	=	18.12	ft	
γ'	=	57.6	pcf	
W_t		5,760	lb/ft of wall	Weight from traffic
W_{mse}	=	97,920	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 5,722 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 26,515 psf

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

q_{ALL} = 10,606 psf

Factor of Safety = **4.63** **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} = 26,515 psf

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

q_{ALL} = 10,606 psf

Factor of Safety = **4.63** **OK**

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 = 2.94 ft

Kern

e < L/6 = 4.00 ft

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=31'
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	=	34	feet
γ_{mse}	=	120	pcf
L	=	24	feet
L factor	=	0.70	
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)

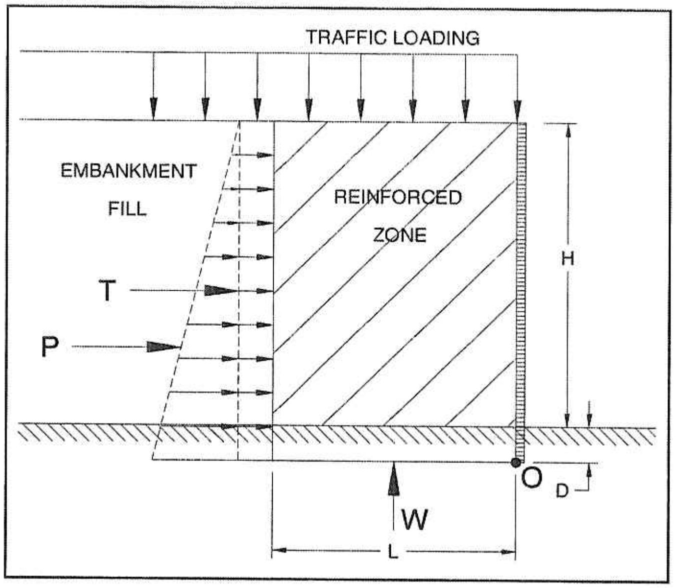
Friction Angle of Embankment Fill

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 = 25,582 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 {ODOT, Bridge Design Manual, 303.4.1.1}

P_r = 44,064 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 $\boxed{FS = \mathbf{1.72}}$ 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

$\Sigma M_{resisting}$ = 1,175,040 lb-ft

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

$\Sigma M_{overturning}$ = 305,184 lb-ft

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

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated $\boxed{FS = \mathbf{3.85}}$ Required FS = 2.00 Resistance Against Overturning is **OK**

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT TranSystems Corp / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth Bypass Project SHEET NO. 9 OF 9
SUBJECT Drilled Shaft End Bearing & Side Friction COMP. BY SJK DATE 1-18-07
S.R. 823 over Blue Run Road CHECKED BY WNT DATE 1-18-07

* From lab testing rock core samples (lower bound) $q_u = 5,840$ psi

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

End Bearing * For lower bed rock RQD between 70-100 & $q_u > 5.2$ tsf

$q_u = 5,840 \text{ psi} = 40.3 \text{ MPa}$

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

$q_{max} = 4.83[40.3 \text{ MPa}]^{0.51} = 31.8 \text{ MPa} = 4,615 \text{ psi} = 665 \text{ ksf}$

$q_a = \frac{q_{max}}{FS} = \frac{665 \text{ ksf}}{3.0} = 221.7 \text{ ksf}$

* For this type of Sandstone, we typically use;

$\boxed{\text{Use } q_{allow} = 80 \text{ ksf}}$ for Competent Rock

[FHWA-IF-99-025] Eq. 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 * Assumes Smooth Rock Socket

$f_{max} = 0.65(14.7 \text{ psi})\left[\frac{5840}{14.7 \text{ psi}}\right]^{0.5} \leq 0.65(14.7 \text{ psi})\left[\frac{4500}{14.7 \text{ psi}}\right]^{0.5}$

$f_{max} = 190 \text{ psi} \leq 167.2 \text{ psi}$ Use $f_{max} = 167$ psi

$f_{allow} = \frac{167}{3.0} = 55.7 \text{ psi} = 8,016 \text{ psf}$

$\boxed{* \text{ Use } f_{allow} = 7,500 \text{ psf}}$ for Competent Rock