Interim Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
Relocated Shumway Hollow Road Over CSXT Railroad
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

Prepared by:

DLZ

DLZ Ohio, Inc.
6121 Huntley Road
Columbus, OH 43229
Phone: (614) 888-0040
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DLZ Job No. 0121-3070.03

November 20, 2006

INTERIM REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

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SCI-823-0.00 PORTSMOUTH BYPASS

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1.0 INTRODUCTION

This report includes the interim findings of evaluations for foundations and mechanically stabilized earth (MSE) retaining walls for the structure at the relocated Shumway Hollow Road over the CSXT Railroad. The findings included in this report pertain to the structure at relocated Shumway Hollow Road over the CSXT railroad only. The findings of other structure evaluations will be submitted in separate documents.

This portion of the project consists of placing one structure to carry traffic on relocated Shumway Hollow Road over the CSXT Railroad. It is anticipated that the proposed structure over the CSXT railroad will be a single-span bridge. See structure plan and profile drawing in Appendix I. It is anticipated that the proposed rear abutment will be founded on a fill section, contained using an MSE wall. The forward abutment will be founded on the slope of a soil/rock cut near station 37+90 (Shumway Hollow stationing).

The findings and recommendations presented in this report should be considered preliminary until the final borings and evaluations have been completed. A final boring plan for this structure has been prepared. Currently, efforts are being made to coordinate the final drilling program with representatives of CSXT. Upon the completion of these final borings, the evaluations and recommendations for this structure will be revised as necessary and finalized.

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

The currently proposed structure is shown on the provided plan and profile drawing in Appendix I. It is understood that an MSE wall will be placed at approximately station 36+75 to contain the rear abutment and hold back the roadway embankment of relocated Shumway Hollow Road. Based upon the structure plan and profile drawing, it is assumed that the maximum height of the fill at station 36+70 (rear abutment) will be approximately 43 feet. It is understood that the forward abutment will be founded on the slope of a soil/rock cut near station 37+90. The proposed roadway grade at the structure and approach embankment is approximately elevation 662.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concepts 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

Two preliminary borings, TR-27 and TR-28, were drilled for the proposed structure on August 25, 2004 and February 2, 2005, respectively. The borings were drilled to depths of 17.5 to 30.5 feet and were extended into bedrock, which was verified by rock coring. It should be noted that the results of borings drilled for pavement design and retaining wall evaluations along SR 335 were also considered in the evaluation of the forward abutment foundation. Boring logs for borings TR-27, TR-28, B-1342, and B-1333 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II. A boring plan is presented in Appendix I.

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

4.0 FINDINGS

4.1 Geology of the Site

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

The genesis of the soils varies across the site. Residual and colluvial soils are found on the ridge tops and the hillsides near the site. These soils are generally thin to moderately deep, covering moderate to steep slopes. 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. In the area of soils of the Minford Complex generally overlie a layer of sand and gravel which is directly above bedrock. In this area, the Minford Complex is characterized by clays of high plasticity and high moisture content.

Although borings drilled for this structure did not encounter soils of the Minford Complex, several other borings drilled for the Shumway Hollow / SR 823 interchange did encounter these soils.

Bedrock within the structure area is primarily sandstone of the Logan Formation that is 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 860.

4.2 Field Reconnaissance

The proposed structure location lies in a shallow railroad cut located immediately west of SR 335. A visual inspection of the cut slope near the forward abutment was performed on September 15, 2006. A log of the exposed rock was created and is included in Appendix II. The cut consists of moderately steep to steep slopes of soil and rock, which are approximately 30 feet high. Elevations cited in the field reconnaissance should be considered approximate due to the accuracy of elevations reported by our field equipment.

At the eastern slope, soil is relatively thin, and consists primarily of residual and colluvial soils. Under the soil, exposed sandstone is evident, beginning at approximately elevation 645. The exposed rock is highly weathered and highly fractured. Bands of interbedded shale or siltstone are present in the sandstone south of the proposed structure, below approximately elevation 638. Areas of isolated seepage were evident in this layer south of the proposed structure. Additionally, several high angle fractures were noted in the rock face, however, no appreciable lateral movement of the rock mass was apparent.

Reconnaissance of the site at the bottom of the cut confirmed that a very soft and wet environment currently exists at the proposed rear abutment location. Drainage channels have been established along the railroad cut, which currently run near the rear abutment location. These drainage paths have deposited significant amounts of soil. Although it has not been verified by borings, it is believed that this recently deposited soil layer is approximately 3 to 5 feet deep.

4.3 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.

4.3.1 Soil Conditions

Boring TR-28 encountered 8 inches of asphalt concrete pavement at the surface. Underlying the pavement, the boring encountered very stiff to hard silt and clay (A-6a) and loose to medium dense coarse and fine sand (A-3a) to a depth of 15.5 feet or elevation 644.2, where bedrock was encountered. Boring TR-27 was drilled off the road, but did not encounter topsoil. Underlying the surface, the boring encountered hard sandy silt to a depth of 7.5 feet where bedrock was encountered.

4.3.2 Bedrock Conditions

Bedrock encountered in borings TR-27 and TR-28 at the proposed structure location was composed primarily of medium hard to hard sandstone that was generally slightly fractured to intact. Recovery of the core samples ranged from 50 to 100 percent and RQD values ranged from 12 to 100 percent with an average RQD of 76 percent. It should be noted that the first five-foot core run in boring TR-28 only recovered 50 percent of the core, which generally indicates poor quality rock. Furthermore, boring B-1342, drilled for the proposed retaining wall on SR-335 and considered for the forward abutment location, encountered bedrock at approximately 21.7 feet below the ground surface, or elevation 639.1.

4.3.3 Groundwater Conditions

Minor seepage was encountered at depths between 14.0 and 18.5 feet below the ground surface in boring TR-28. No seepage was encountered in boring TR-27. Prior to coring rock, no water was present in either of these borings. At completion of drilling, the water level in TR-28 was 10.0 feet including coring water. Boring TR-27 collapsed at a depth of 6.0 feet. It should be noted that the final water levels include drilling water and consequently may not be representative of the actual groundwater conditions.

It should also 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

5.1 General Information

Based upon the amount of embankment fill and the approximate depth to bedrock, spread footings, drilled shafts, or CIP piles socketed into bedrock are considered suitable to support the rear abutment. Additionally, the forward abutment is understood to be located on a rock/soil slope, with the centerline of bearing at approximately station 37+90. Given the highly weathered nature of the bedrock near the face of the slope, and the abutment location with respect to the slope, drilled shafts socketed into bedrock are best suited to support the forward abutment.

The following sections contain recommendations and information for the design of the proposed structural foundations and MSE wall. Table 1, on the following page, summarizes the site conditions and foundation recommendations. The approximate bearing elevations presented below indicate both the elevations at the borings and the approximate elevations at the abutment locations. The information provided in the summary tables will be revised as necessary based upon soil and bedrock conditions encountered in the final borings.

Table 1-Summary of Foundation Recommendation

Structural Element	Structure Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	TR-27	646.3^{*} / 625.0^{+}	CIP Piles	615.0^{+**}	Allowable Capacity^{++}
			Drilled Shafts	615.0^{+**}	80 ksf^{+++}
			Spread Footings	In MSE Fill	4 ksf
Forward Abutment	TR-28	659.7^{*} / 650.0^{+}	Drilled Shafts	634.0^{+**}	40 ksf^{+++}

* Elevation at preliminary structural boring location
** Includes 5-foot embedment into competent rock
$^{+}$ Ground surface elevation at abutments was estimated from the established topographic mapping
$^{++}$ Pile capacity should conform to ODOT BDM 202.2.3.2
$^{+++}$ End bearing capacity only

5.2 Bridge Foundation Recommendations

5.2.1 Rear Abutment (Sta. 36+70)

The rear abutment of the proposed structure lies in a shallow railroad cut. The preliminary boring drilled for the rear abutment of the structure was drilled outside of the rock cut. Therefore, these recommendations are based on the reconnaissance of the site conditions as well as the results of the preliminary boring.

It is understood through previous comments from the ODOT Office of Structural Engineering (OSE) that CIP piles could to be used to support the rear abutment. It is understood that the abutments could be supported by steel CIP piles placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment 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 CIP piles compared to drilled shafts, the steel CIP piles are anticipated to provide low resistance to lateral earth pressures that can be induced in high embankment fills. 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 rear abutment. Due to the large amount of embankment fill, drilled shafts socketed a minimum of 5 feet into competent rock are recommended to support the proposed rear abutment. This corresponds to an approximate bearing elevation of 615 at the rear abutment. The bearing elevation may be revised based upon the results of the final borings. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

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.

Spread footings bearing in the MSE wall fill may also be considered to support the rear abutment. 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 wall, as proposed, will be founded on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be negligible.

5.2.2 Forward Abutment (Sta. 37+90)

The forward abutment of the proposed structure lies on the eastern slope of the shallow railroad cut. The preliminary boring drilled for the forward abutment of the structure was drilled outside of the rock cut. Therefore, these recommendations are based on the reconnaissance of the site conditions as well as the results of the preliminary boring. Reconnaissance of the site included a visual inspection of the railroad cut at the forward abutment location. A log of exposed rock near this location was recorded and is included in Appendix II.

From the borings, it is anticipated that competent bedrock will be encountered within 3 to 5 feet of the soil-rock interface at the proposed centerline of bearing for the forward abutment. This corresponds to an approximate elevation of 639.

It is recommended that drilled shafts be used to support the forward abutment. Using drilled shafts at the forward abutment, a minimum 5-foot deep socket into competent rock is required. This corresponds to an approximate bearing elevation of 634 at the forward abutment. The bearing elevation may be revised based upon the results of the final borings. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). If significant lateral loading is anticipated, longer rock sockets may be required for stability.

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 5,000 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.

5.2.3 Drilled Shaft Foundations: General Recommendations

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

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 drainage channel level (rear abutment) 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.

Special considerations need to be given to the use of drilled shaft foundations with MSE walls. If drilled shafts are to be used, consideration of the diameter, spacing, and location of the drilled shafts will be of concern. The drilled shafts should be set back from the MSE wall panel a sufficient distance to allow reinforcing straps to be splayed around the shafts at an angle of 15 degrees or less.

5.3 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that an MSE wall will be used to construct the embankment and contain the rear abutment at station 36+75. Recommendations for this MSE wall are presented in the following sections. The MSE wall should be constructed according to the recommendations presented in this report and in conformance with the manufacturer's specifications.

5.3.1 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.

The parameters required to perform the stability analyses are presented in Table 2, below. As outlined in section 5.3.2, the existing soils at the rear abutment location are to be removed and replaced to the top of bedrock. Consequently, the properties of compacted granular fill are used for the stability calculations of the MSE wall. 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. Additionally, 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 2-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

5.3.2 MSE Wall Evaluations and Recommendations

The MSE fill at the rear abutment is understood to have a maximum height of approximately 43 feet. Due to the rear abutment location of the proposed structure being located in a railroad rock cut, it is anticipated that bedrock is very shallow (approximately 3 to 5 feet). Very soft and wet soils are present at the proposed rear abutment location. Consequently, it is recommended that soils overlying bedrock be removed, and the leveling pad be placed on bedrock. Additionally, provisions for the routing of water near the proposed MSE wall should be made to prevent any scour of the MSE wall materials.

If the MSE wall is founded on bedrock, the bearing capacity, global stability, and settlement of the wall are assumed to be adequate and thus calculations are not necessary. Calculations for stability (sliding and overturning) are included in Appendix IV. Other external and internal stability analyses are required for the design of an MSE wall, but are considered outside the scope of this report. For stability, calculations have shown that a minimum reinforcement length of 0.7 times the full wall height, or 30.1 feet, must be used for the proposed MSE wall at this location.

A summary of soil properties, summary of the results of calculations, and MSE retaining wall parameters are presented in Table 3, below.

Table 3-MSE Retaining Wall Parameters and Results of Analyses (Rear Abutment)
Boring TR-27

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on F = 30°)
Sliding along base of MSE wall Sliding Coefficient (m)(0.67) = tan 34°(0.67) = 0.45 Use (m)(0.67) = 0.55 as a maximum value as per AASHTO, BDM, 303.4.1.1
Allowable Bearing Capacity – Undrained Condition q_{all} = 20 ksf (Bearing on rock)
Allowable Bearing Capacity – Drained Condition q_{all} = 20 ksf (Bearing on rock)
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 = 43.0 feet Approximate Embedment Depth = 0.0 feet (Bedrock) Minimum Length of Reinforcement for External Stability = 30.1 feet

5.4 General Earthwork Recommendations

Borings drilled for the proposed structures did not encounter any organic soils. However, since organic or very soft soils may be encountered at locations other than where the borings were drilled, the contractor should be prepared to perform overexcavation of any poor soils at other locations and replace the overexcavated soil with compacted engineered fill as needed. Additionally, all topsoil, organic soil within 3 feet of subgrade level, and vegetation should also be removed prior to placing fill or pavement materials.

Durable sandstone is evident at the rear abutment location in the rock cut. Significant rock excavation may be required to accommodate the reinforcing straps of the MSE retaining wall. If necessary, the contractor should be prepared to excavate hard, durable sandstone by blasting or other means. In places where fill is to be placed on bedrock, a level bench should be cut into the bedrock prior to the placement of fill for stability purposes.

5.5 Groundwater Considerations

Minor seepage was encountered in boring TR-28. No groundwater was noted prior to adding drill water in either boring. Representative, final water levels could not be obtained due to the use of water during rock coring operations. The use of drilling water in rock coring operations also masked any seepage zones that may be present in the rock. Excavations for shafts extending below the soil-rock interface may encounter significant seepage through fractured zones in the rock. The contractor should be prepared to deal with any seepage or precipitation that may enter any excavations.

6.0 CLOSING REMARKS

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

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Arthur (Pete) Nix, P.E.
Geotechnical Engineering Division Manager

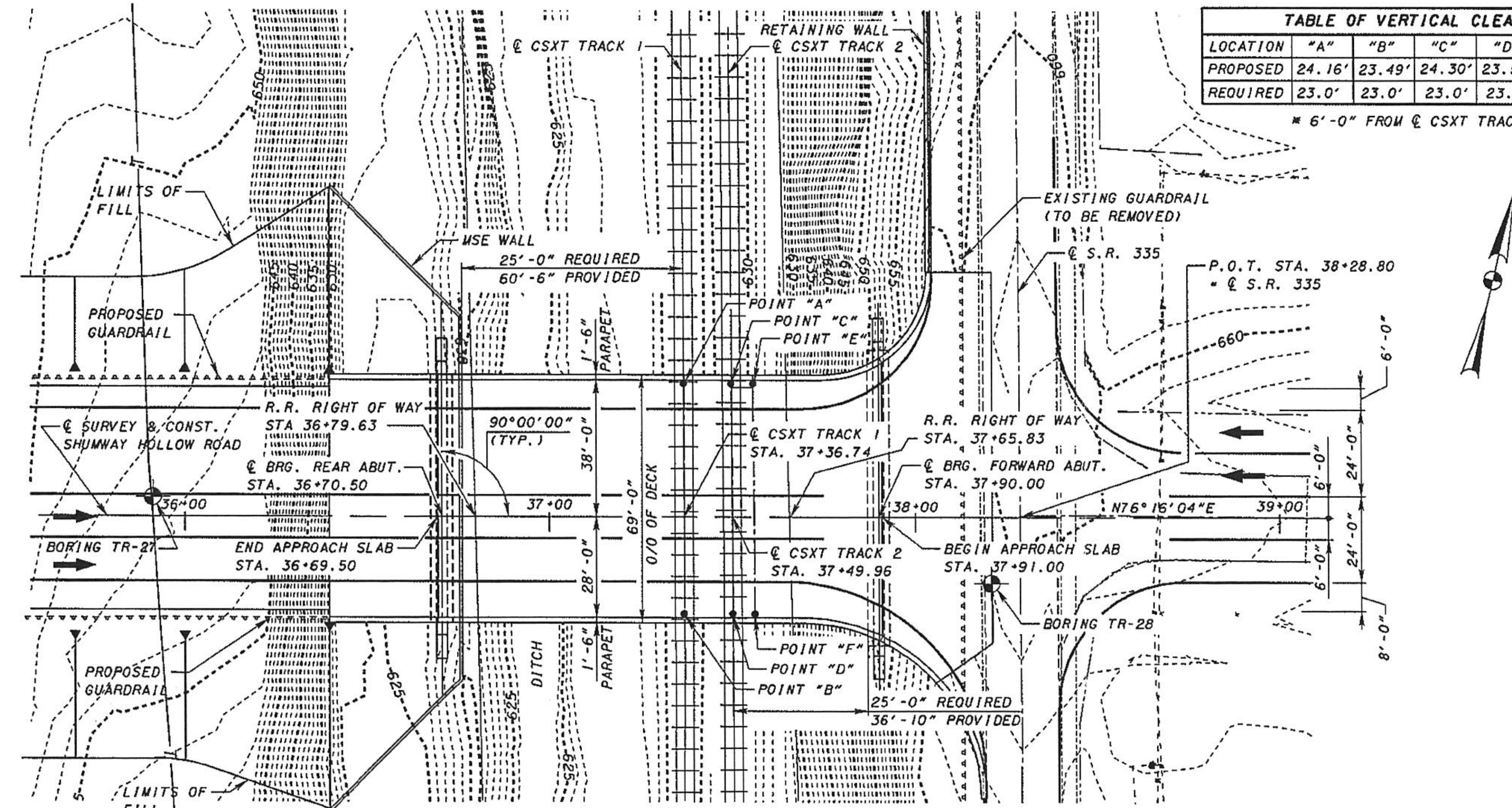
sjr

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\Shumway Hollow Road over CSXT RR\Shumway Hollow Rd over CSXT Joint Interim Report.doc

APPENDIX I

Structure Plan and Profile Drawing – 11"x17"

Boring Plan – 11"x17"



PLAN

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"	"C"	"D"	"E"*	"F"*
PROPOSED	24.16'	23.49'	24.30'	23.52'	25.58'	24.91'
REQUIRED	23.0'	23.0'	23.0'	23.0'	23.0'	23.0'

* 6'-0" FROM ℄ CSXT TRACK 2

BORING LOCATIONS

BORING No.	STATION	OFFSET
TR-27	35+91.27	5.31' LT.
TR-28	38+20.74	18.43' RT.

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

TRAFFIC DATA

(SHUMWAY HOLLOW ROAD)
CURRENT YEAR ADT (2010) = 3,800
DESIGN YEAR ADT (2030) = 7,800
CURRENT YEAR ADTT (2010) = 228
DESIGN YEAR ADTT (2030) = 468

PROPOSED STRUCTURE

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

SPANS: 119'-6" C/C BEARINGS

ROADWAY: 66'-0" TOE TO TOE OF PARAPETS

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

SKEW: NONE

CROWN: VARIES

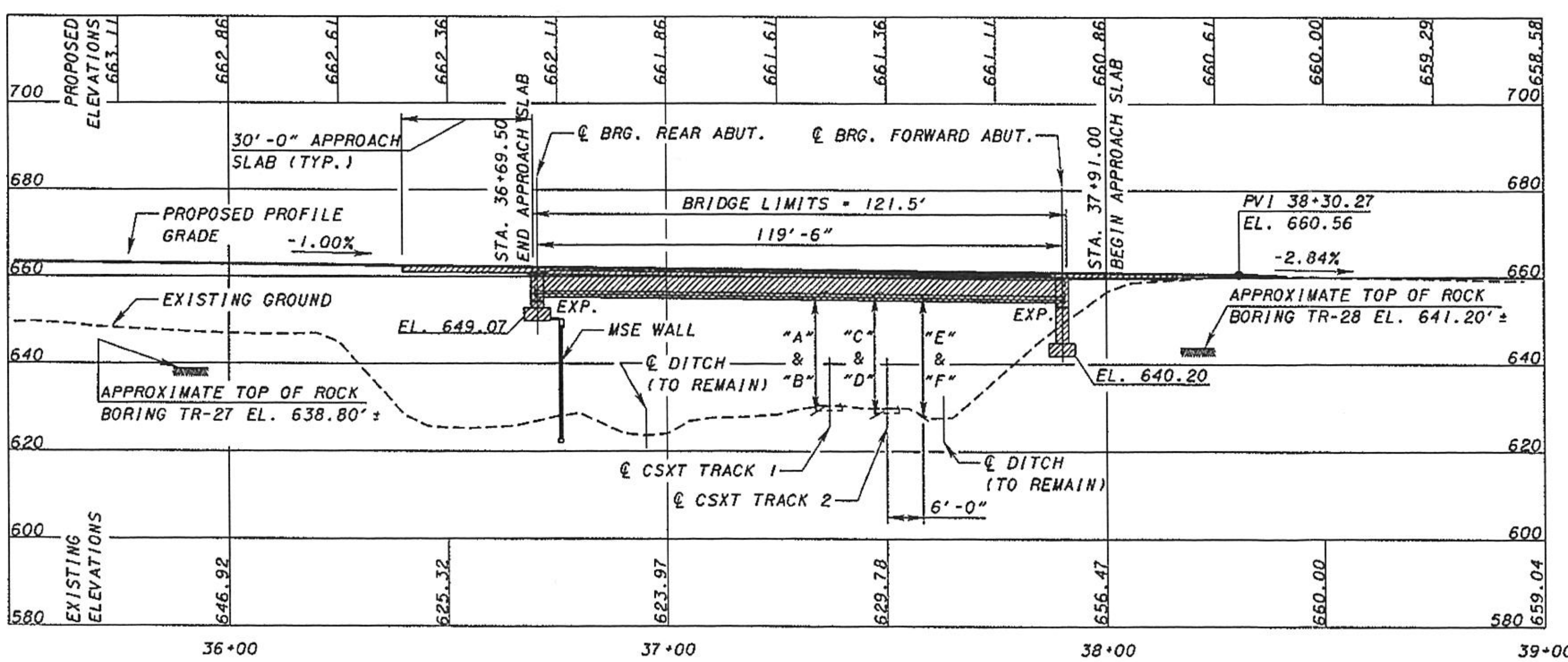
ALIGNMENT: TANGENT

WEARING SURFACE: 1" MONOLITHIC CONCRETE

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

LATITUDE:

LONGITUDE:



ELEVATION ALONG ℄ SURVEY AND CONSTRUCTION SHUMWAY HOLLOW ROAD

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:

REAR AND FORWARD ABUTMENTS SHALL BE ON SPREAD FOOTINGS. USE ALLOWABLE BEARING CAPACITY OF 4 KSF AT REAR ABUTMENT, 30 KSF AT FORWARD ABUTMENT.

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED DURING THE TS&L SUBMITTAL

DESIGN AGENCY
TRANSYSTEMS CORPORATION

DESIGNED	CHECKED	DRAWN	REVISED	REVIEWED	DATE
MSL	MSL	MTN		JRC	04/27/06

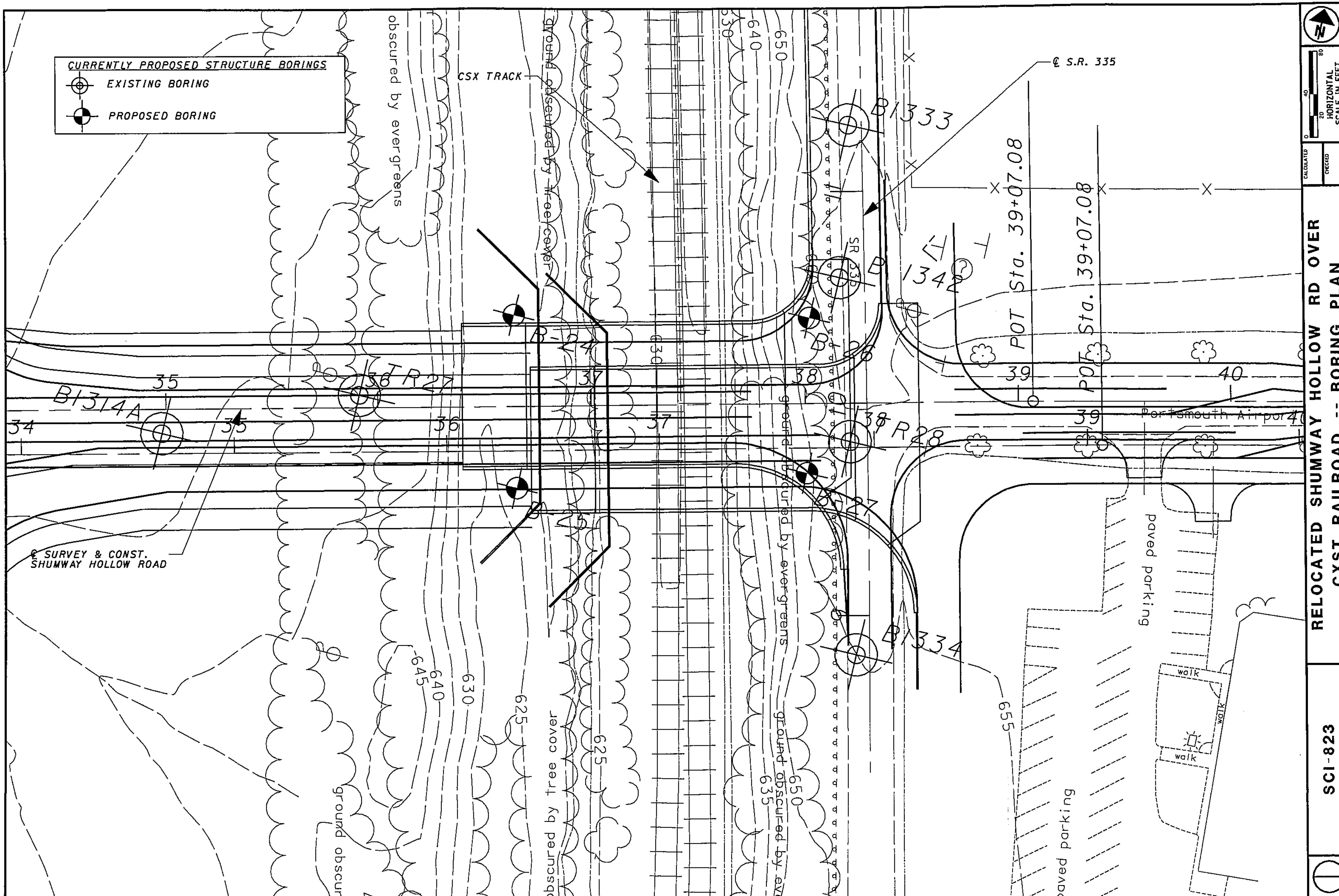
STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 36+69.50
STA. 37+91.00

PRELIMINARY SITE PLAN - ALTERNATIVE 1
BRIDGE NO. SCI-823-XXXX
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-0.00
PID 19415

1/3


CURRENTLY PROPOSED STRUCTURE BORINGS
EXISTING BORING
PROPOSED BORING
CSX TRACK
℄ S.R. 335
B1333
B1342
B1334
B1314A
TR27
TR28
B-24
B-25
B-26
B-27
POT Sta. 39+07.08
POT Sta. 39+07.08
SR 335
Portsmouth Airport
℄ SURVEY & CONST.
SHUMWAY HOLLOW ROAD
obscured by evergreens
ground obscured by tree cover
ground obscured by evergreens
ground obscured by evergreens
obscured by tree cover
ground obscure
paved parking
paved parking
walk
walk
walk
34
35
35
36
36
37
37
38
38
39
39
40
625
625
630
630
635
640
640
645
650
650
655
HORIZONTAL SCALE IN FEET
0
20
40
80
CALCULATED
CHECKED
RELOCATED SHUMWAY HOLLOW RD OVER
CXST RAILROAD -- BORING PLAN
SCI-823

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Four (4) Borings
Log of Rock Cut – Eastern Slope

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

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

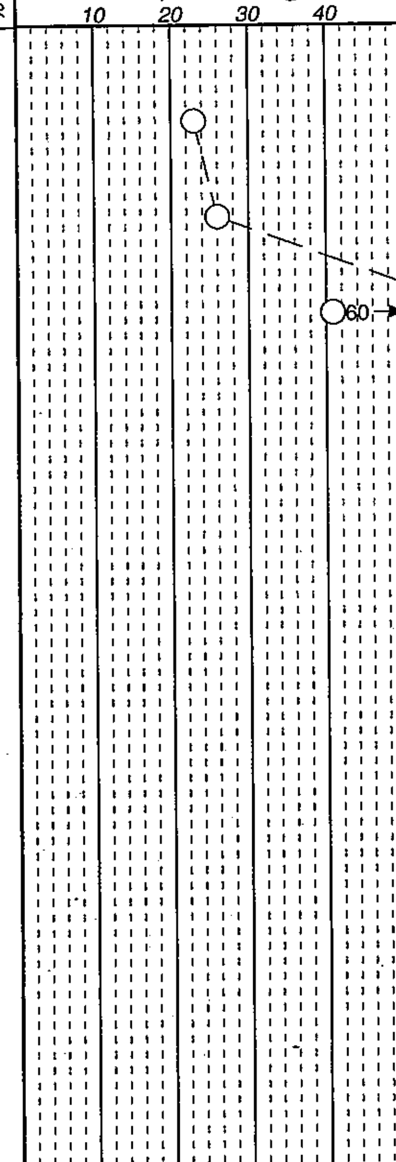
Client: TranSystems, Inc. **Project:** SCI-823-0.00 **Job No.** 0121-3070.03

LOG OF: Boring TR-27 **Location:** Sta. 35+91.3, 5.9 ft. LT of Rel. Shumway Hollow CL **Date Drilled:** 8/25/04

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (boring collapsed @ 6.0')

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	646.3												
0.4	645.9						Topsoil - 5"						
		7 10 13	18	1		4.5+	Hard brown SANDY SILT (A-4a), trace clay, trace to little gravel; damp.						
		8 13 13	18	2		4.5+							
		4 10 50	16	3		4.5+	@ 6.0'-7.5', contains sandstone fragments.						
7.5	638.8	Core 120"	Rec 120"	RQD 98%	R-1		Medium hard to hard brown and gray SANDSTONE; very fine to fine grained, slightly to highly weathered, argillaceous, micaceous, massive, slightly fractured. @ 7.5'-10.0', rust stained. @ 7.8',8.9', 15.6', low angle fractures. @ 14.9'-15.2', high angle fracture.						
17.5	628.8						Bottom of Boring - 17.5'						

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



FILE: 0121-3070-03 [11/17/2006 9:27 AM]

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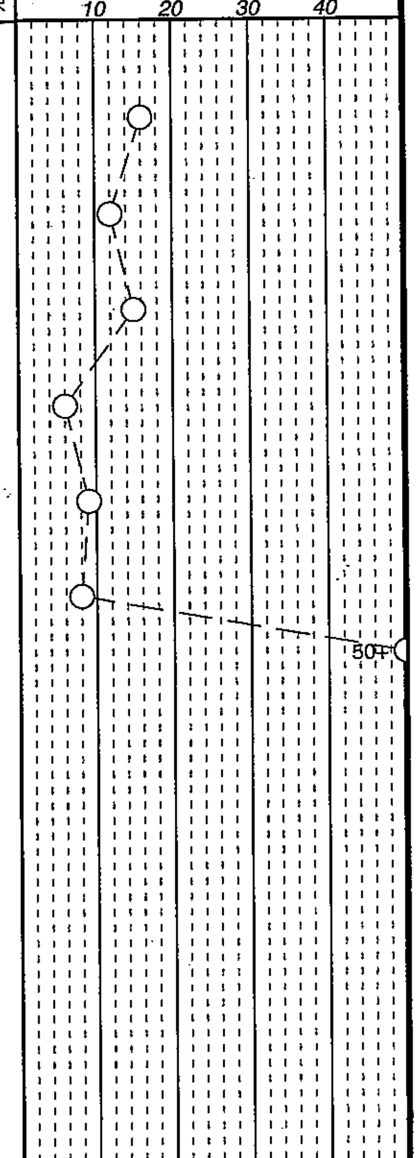
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-28 | Location: Sta. 38+20.7, 17.8 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: 14.0', 18.5'
Water level at completion: 10.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	659.7						Asphalt Concrete Pavement - 8"						
0.7	659.0	4 8 8	16	1		4.0	Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand; damp.						
3.0	656.7	5 5 7	15	2			Medium dense reddish brown COARSE AND FINE SAND (A-3a); moist. (residual soil)						
		8 8 7	18	3									
		6 4 2	18	4									
		3 5 4	14	5									
		1 4 4	13	6									
15.5	644.2	50/2	2	7			Severely weathered gray SANDSTONE argillaceous.						
18.5	641.2	Core 60"	Rec 30"	RQD 12%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, massively bedded, slightly fractured. @ 18.5'-24.0', broken.						
		Core 84"	Rec 84"	RQD 100%	R-2								



FILE: 0121-3070-03 [11/17/2006 9:27 AM]

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-28 | Location: Sta. 38+20.7, 17.8 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: 14.0', 18.5'
Water level at completion: 10.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ (10 20 30 40)
30	629.7													
30.5	629.2						Bottom of Boring - 30.5'							
35														
40														
45														
50														
55														
60														

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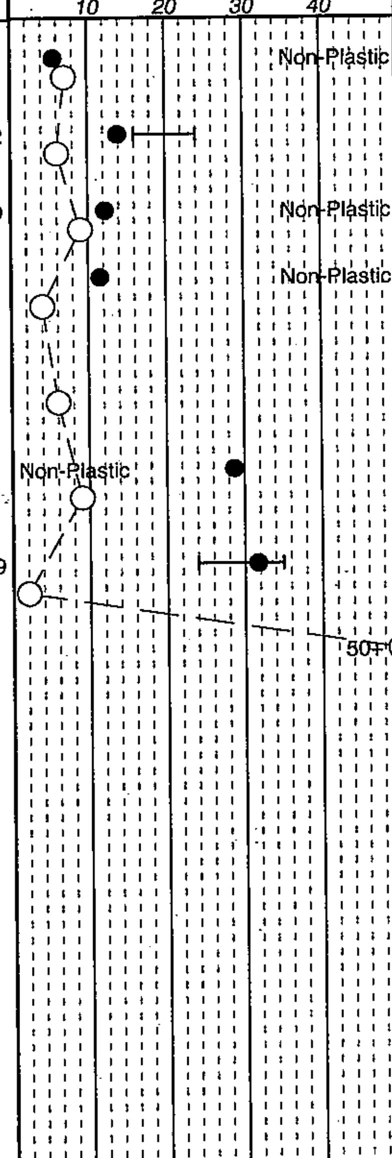
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1333 | Location: Sta. 11+30.6, 6.3 ft. LT of SR 335 CL | Date Drilled: 09/13/05

WATER OBSERVATIONS: Water seepage at: 12.0'
Water level at completion: 2.6' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	660.1												
0.3	659.8						Topsoil - 4"						
		3 3 4 6	8	1			Loose brown GRAVEL WITH SAND AND SILT (A-2-4), little to some clay, little to some silt; trace to some gravel gravel; damp.	32	18	--	21	22	7
		3 2 4 6	18	2				0	7	--	61	10	22
4.0	656.1	3 4 5 6	18	3			Loose brown COARSE AND FINE SAND (A-3a), trace to little silt, little to some clay; damp to moist.	0	2	--	69	9	20
		3 2 2	16	4				0	15	--	68	17	
8.0	652.1	2 3 3	17	5			Loose brown FINE SAND (A-3), some coarse sand, trace silty clay; moist.						
		1 1 8	13	6			@ 11.0'-12.5', wet.	0	24	--	72	4	
13.0	647.1	1 1 1	18	7		<0.25	Very ~~soft~~ brown SILT AND CLAY (A-6a), trace fine sand; moist.	0	0	--	1	60	39
16.0	644.1	50/4		8			Dense to very dense brown COARSE AND FINE SAND (A-3a) trace silty clay; damp to moist.						
18.0	642.1	Core 72.0"	Rec 62.5"	RQD 72%	R-1	*376	Medium hard to hard gray SANDSTONE ; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, medium bedded to thickly bedded, slightly to moderately fractured. @ 19.6'-19.8', 20.2'-20.5', broken zones.						
		Core 72"	Rec 72"	RQD 80%	R-2	*323	@ 25.0'-25.5', lost recovery.						
30.0	630.1						Bottom of Boring - 30.0'						

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



FILE: 0121-3070-03 [11/17/2006 9:27 AM]

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

LOG OF: Boring B-1333 | Location: Sta. 11+30.6, 6.3 ft. LT of SR 335 CL | Date Drilled: 09/13/05

WATER OBSERVATIONS: Water seepage at: 12.0'
Water level at completion: 2.6' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	630.1													
35														
40														
45														
50														
55														
60														

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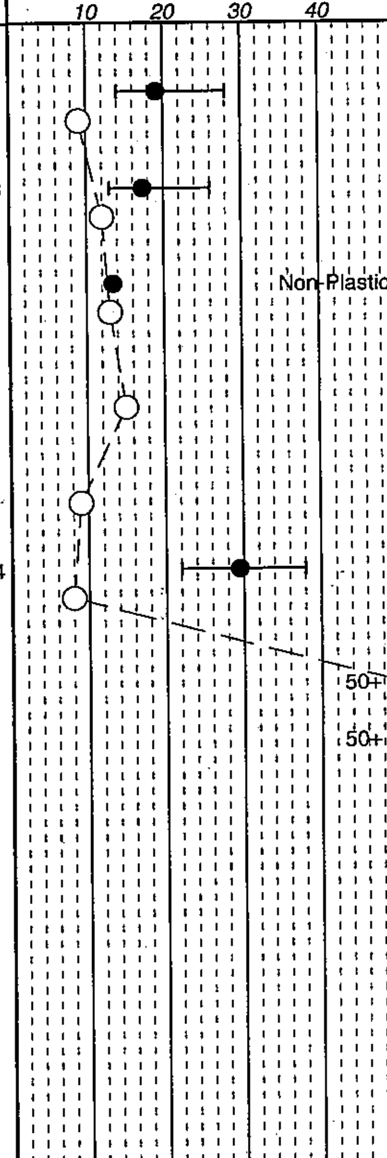
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1342 | Location: Sta. 10+59.4, 11.7 ft. LT of SR 335 CL | Date Drilled: 6/21/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
13.0' (includes drilling water, inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	660.8						Stiff brown SILT AND CLAY (A-6a), trace coarse sand, trace gravel, some fine sand; moist.						
		4 4 5	15	1		1.5		1	2	--	26	30	42
		6 6 6	16	2		-	@ 4.0', "and" fine sand.	0	4	--	56	14	26
5													
6.5	654.3	3 6 7	18	3		-	Medium dense brown COARSE AND FINE SAND (A-3a), little silty clay; damp.	0	8	--	79	14	
		3 7 8	16	4		-							
10													
		3 5 4	18	5		-	@ 11.5', moist to wet.						
14.0	646.8	5 4 4	18	6		1.0	Medium stiff to stiff brown SILTY CLAY (A-6b), trace fine to coarse sand, trace organic clay, trace gravel; moist.	1	1	--	2	63	34
15													
15.5	645.3	7 35 50/3	14	7			Dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace organic clay; damp to moist.						
		50/3	3	8									
19.5	641.3						Medium hard to hard light gray SANDSTONE; fine to medium grained, moderately weathered, thinly bedded to medium bedded, highly to moderately fractured.						
20							@ 19.5' to 21.7', possible core loss (coring in dense sand and gravel).						
25		Core 120"	Rec 88"	RQD 56%	R-1		@ 25.0', qu= 13,229 psi, Er= 2,759,623 psi.						
							@ 27.0' to 27.4', possible core loss.						
							@ 27.5' lost water return.						
							@ 29.0' to 29.5', high angle fracture, broken.						
30													

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



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

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

LOG OF: Boring B-1342 | *Location:* Sta. 10+59.4, 11.7 ft. LT of SR 335 CL | *Date Drilled:* 6/21/06

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
13.0' (includes drilling water, inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	630.8						@ 29.5' to 30.9', possible core loss.							
30.9	629.9	Core 120"	Rec 104"	RQD 83%	R-2		Hard gray SANDSTONE; fine grained, slightly to moderately weathered, medium bedded to thickly bedded, moderately fractured. @ 32.0', qu= 13,226 psi, Er= 2,493,963 psi.							
		Core 120"	Rec 120"	RQD 100%	R-3		@ 44.5', dark gray, moderately to slightly fractured.							
49.5	611.3						Bottom of Boring - 49.5'							

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ⊢——⊣ LL
Blows per foot - ○
10 20 30 40

Depth scale: 30, 35, 40, 45, 50, 55, 60

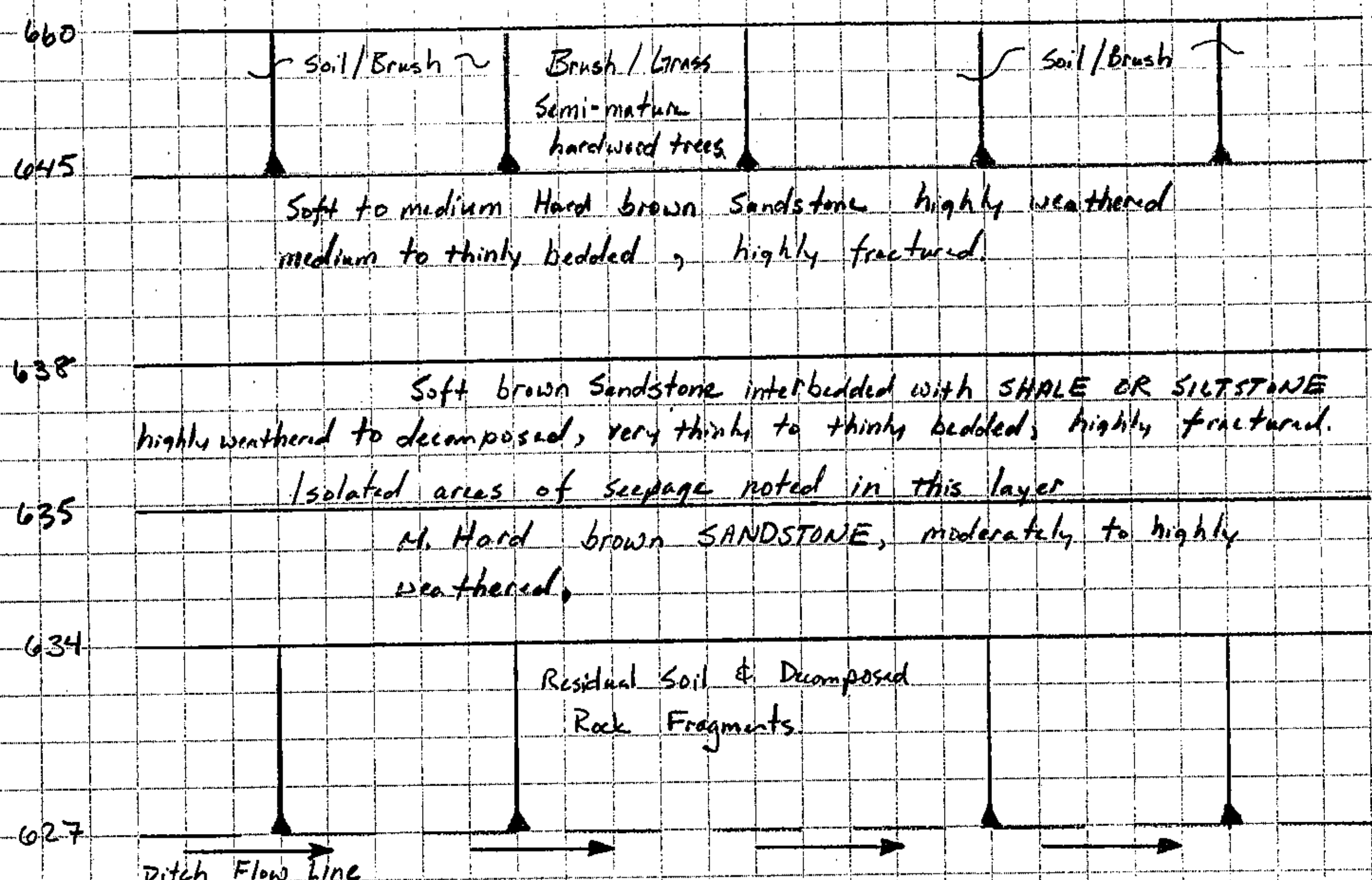
FILE: 0121-3070-03 [11/17/2006 9:27 AM]

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Shumway Hollow Rd over CSX RR
Log of Railroad Rock Cut

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 1
COMP. BY SJK DATE 11-14-06
CHECKED BY ______ DATE ______

Log of Railroad Rock Cut: East Cut - Forward Abutment Location
At structure location rock obscured by soil and rock fragments. Slope located approximately 50' South of proposed structure centerline was logged.

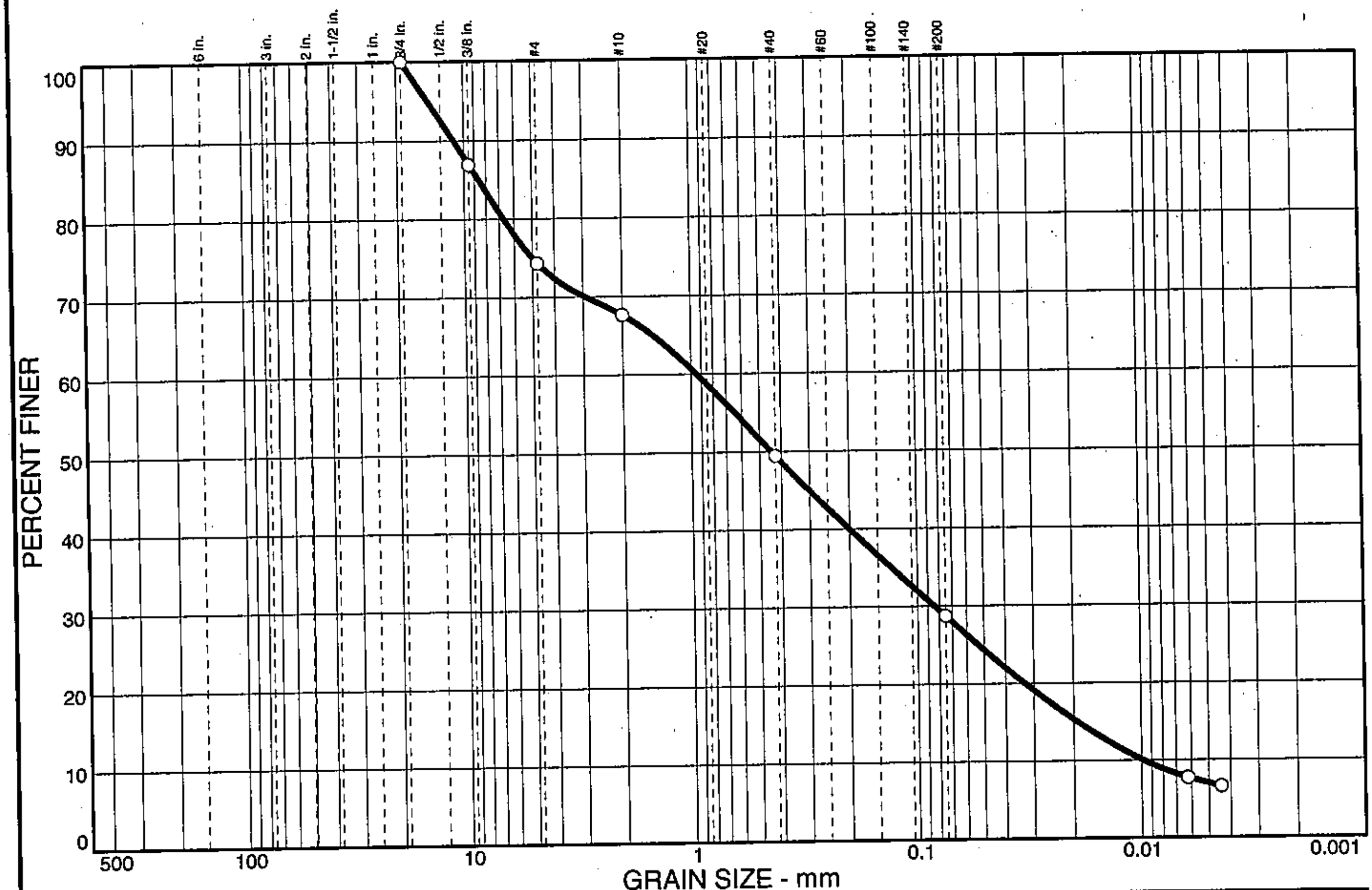


Vertical fractures were noted in the exposed rock. No significant lateral movement is evident from visual inspection of these fractures.

Isolated areas of seepage are evident in rock layers between approximately elevations 635 and 638.

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	25.7	6.7	18.2	20.7	21.7	7.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	86.9		
#4	74.3		
#10	67.6		
#40	49.4		
#200	28.7		

* (no specification provided)

Soil Description

Silty sand with gravel

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 8.66 D_{60}= 0.946 D_{50}= 0.445
D_{30}= 0.0843 D_{15}= 0.0193 D_{10}= 0.0098
C_u= 96.20 C_c= 0.76

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Moisture Content= 5.6%
F.M.=0.39

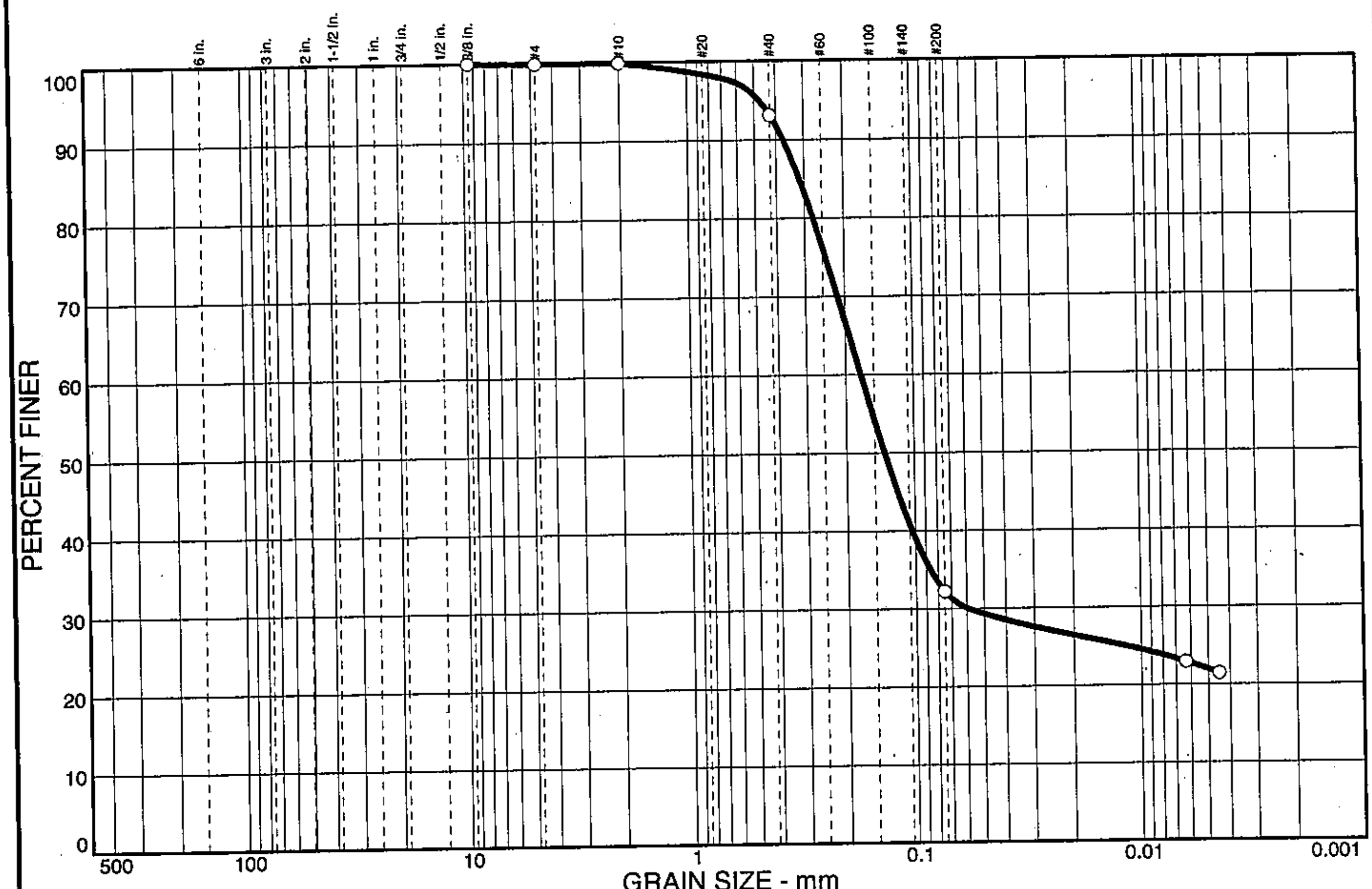
Sample No.: 1 **Source of Sample:** B-1333 **Date:** 7/21/06
Location: **Elev./Depth:** 0.0

DLZ

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

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.1	0.0	6.8	60.9	10.3	21.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.9		
#10	99.9		
#40	93.1		
#200	32.2		

* (no specification provided)

Soil Description

Clayey sand

Atterberg Limits

PL= 16 LL= 24 PI= 8

Coefficients

D_{85}= 0.317 D_{60}= 0.172 D_{50}= 0.136
D_{30}= 0.0591 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-4(0)

Remarks

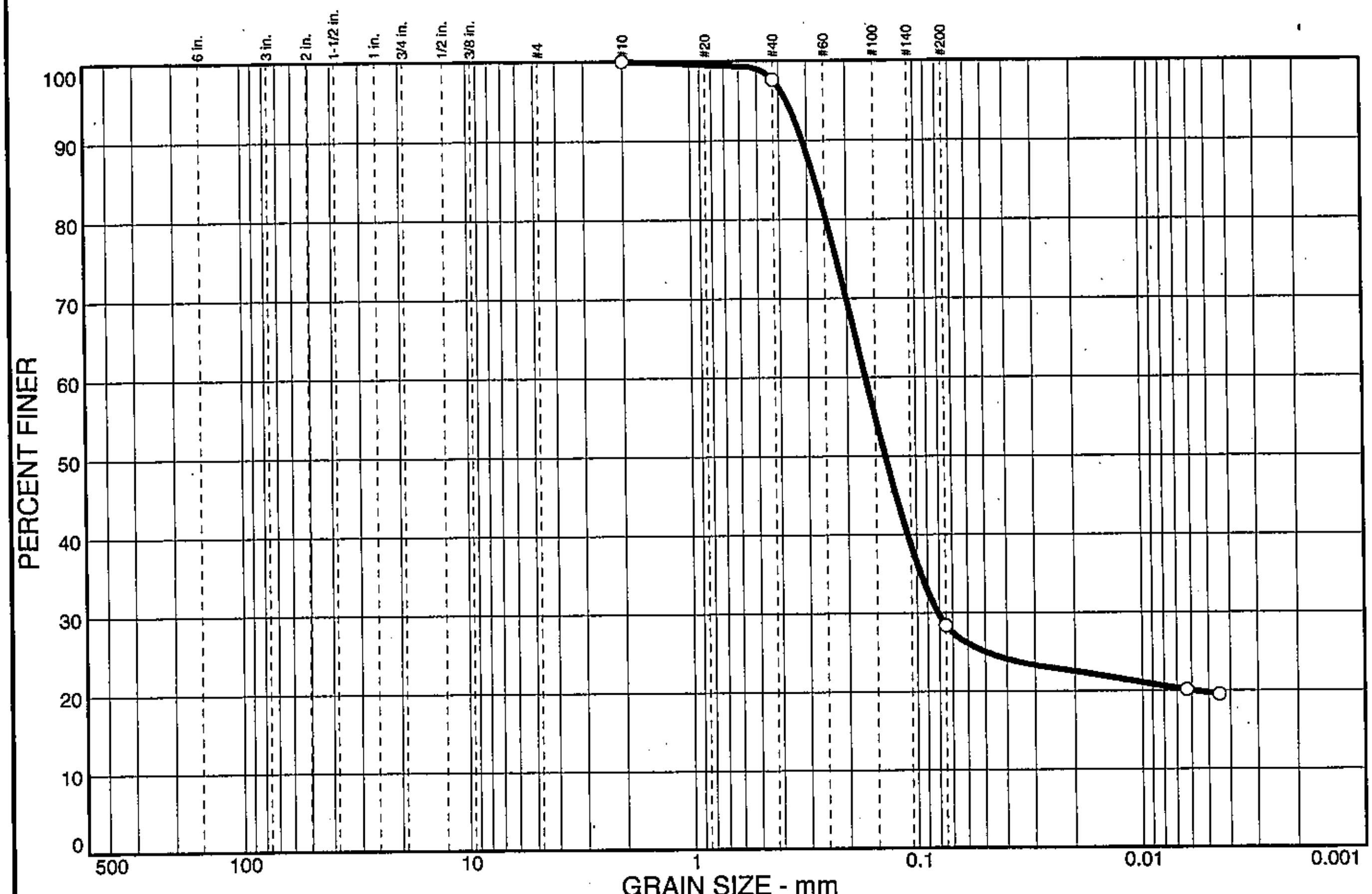
Moisture Content= 13.9%
F.M.=0.00

Sample No.: 2 **Source of Sample:** B-1333 **Date:** 1/17/06
Location: **Elev./Depth:** 2.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	2.4	69.2	8.7	19.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	97.6		
#200	28.4		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.277 D_{60}= 0.167 D_{50}= 0.137
D_{30}= 0.0813 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

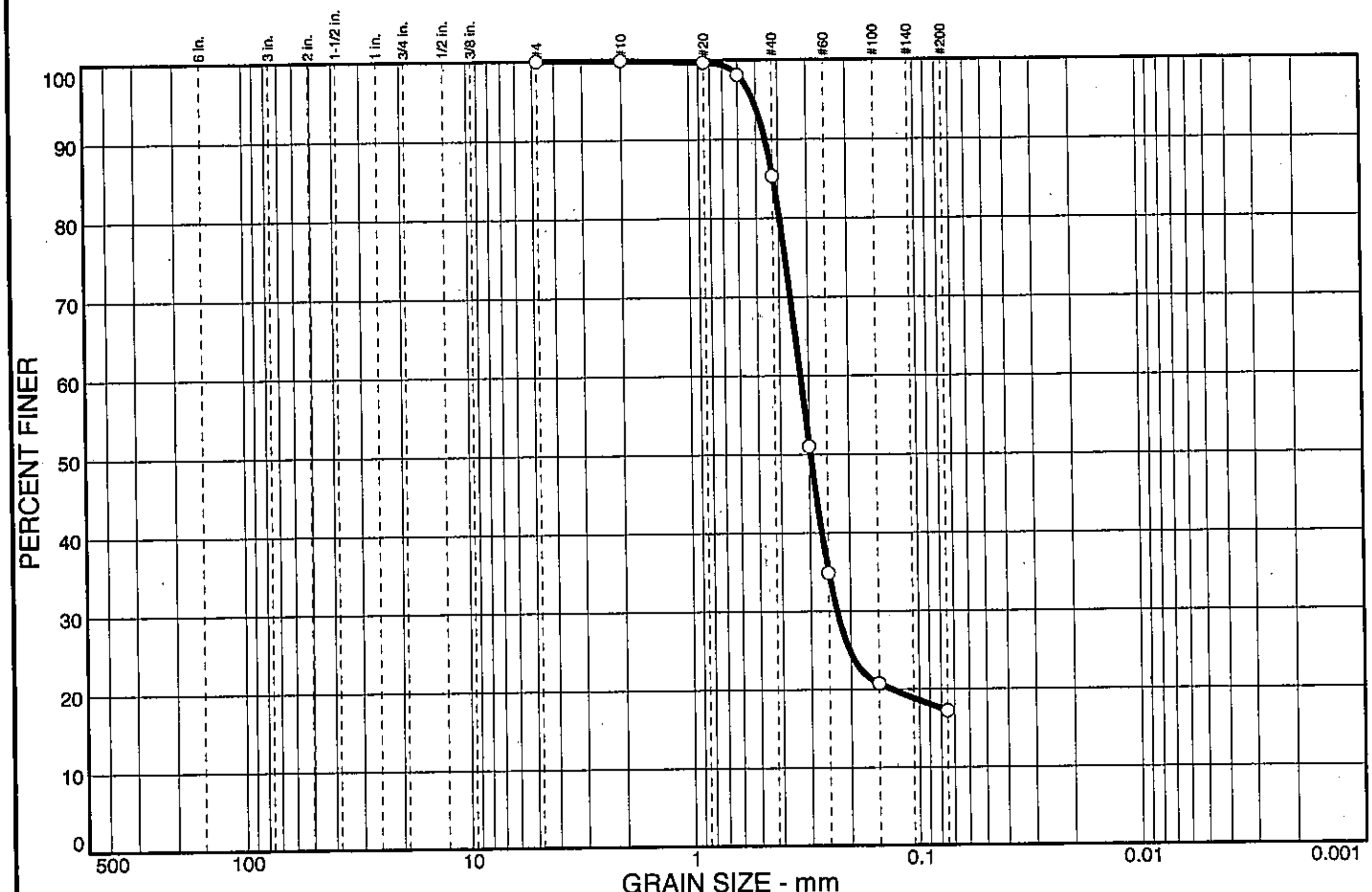
Moisture Content= 12.2%

Sample No.: 3 **Source of Sample:** B-1333 **Date:** 1/17/06
Location: **Elev./Depth:** 4.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	14.7	68.1	17.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.7		
#30	98.1		
#40	85.3		
#50	51.0		
#60	34.9		
#100	20.7		
#200	17.2		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.423 D_{60}= 0.327 D_{50}= 0.297
D_{30}= 0.231 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

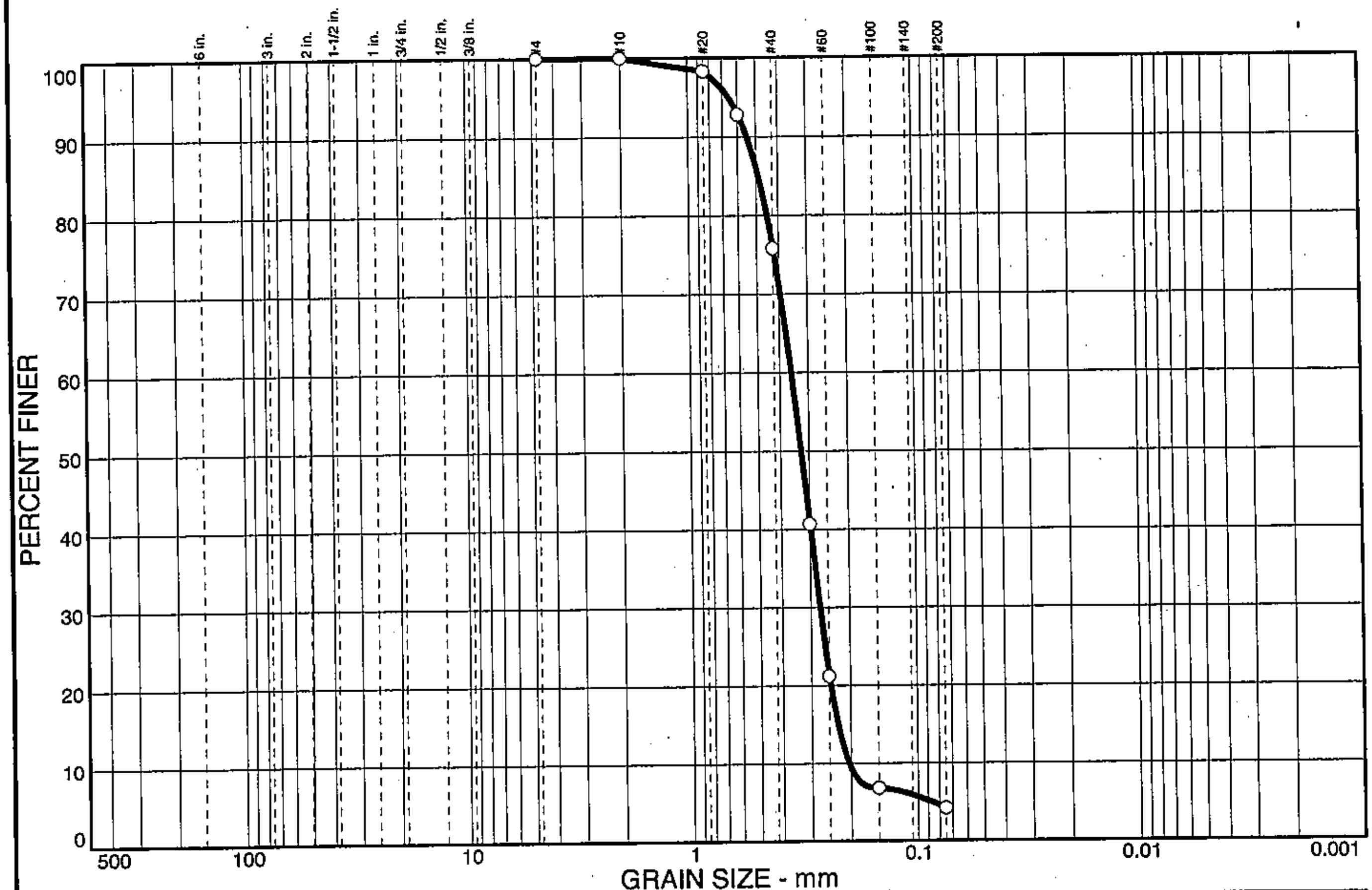
Moisture Content= 11.5%
F.M.=1.30

Sample No.: 4 **Source of Sample:** B-1333 **Date:** 7/21/06
Location: **Elev./Depth:** 6.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	24.2	71.6	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	98.3		
#30	92.8		
#40	75.8		
#50	40.7		
#60	21.2		
#100	6.8		
#200	4.2		

* (no specification provided)

Soil Description

Poorly graded sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.491 D_{60}= 0.357 D_{50}= 0.326
D_{30}= 0.273 D_{15}= 0.230 D_{10}= 0.205
C_u= 1.74 C_c= 1.02

Classification

USCS= SP AASHTO= A-3

Remarks

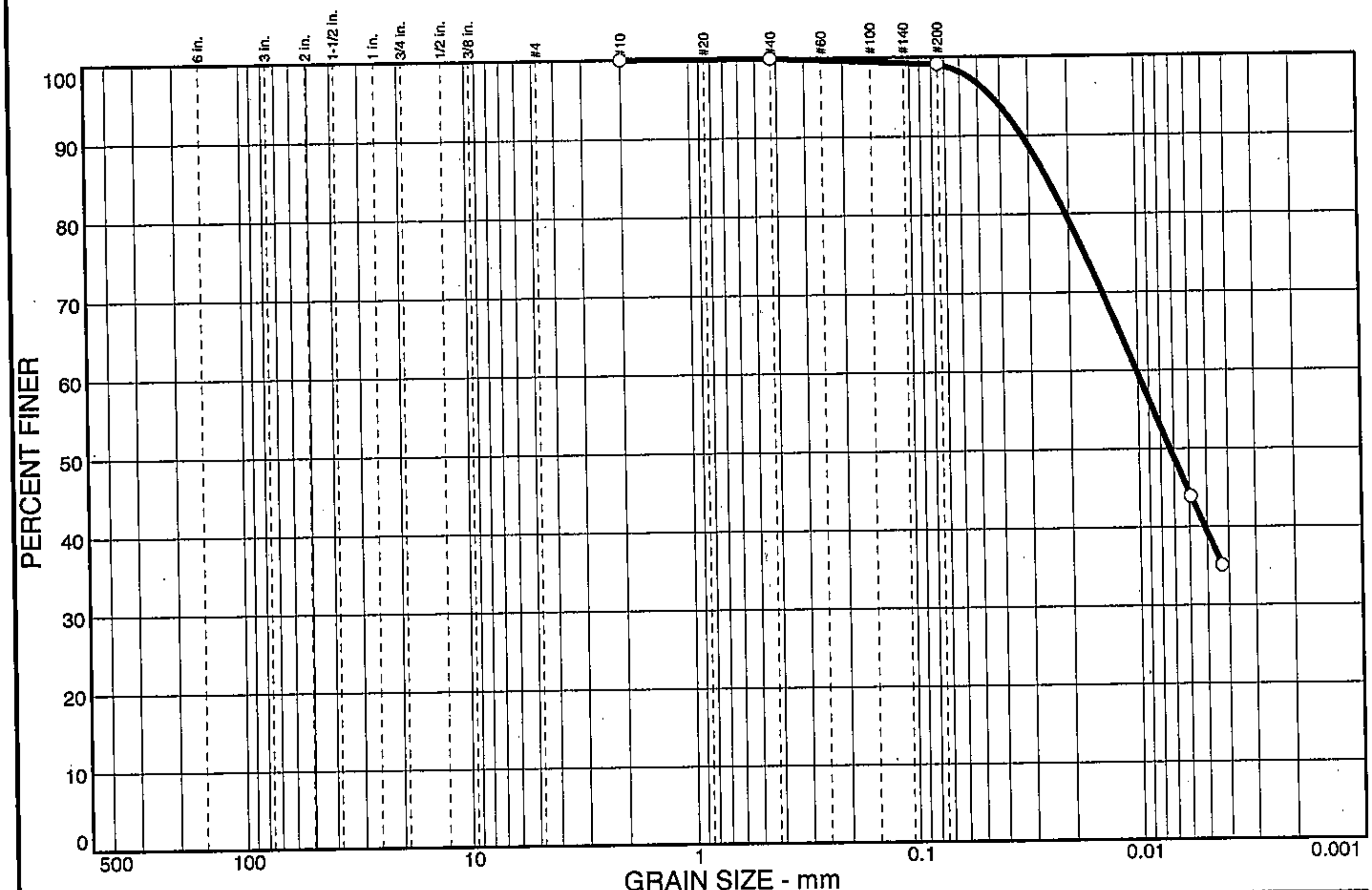
Moisture Content= 28.7%
F.M.=1.60

Sample No.: 6 **Source of Sample:** B-1333 **Date:** 1/17/06
Location: **Elev./Depth:** 11.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.0	0.9	59.9	39.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	100.0		
#200	99.1		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 24 LL= 35 PI= 11

Coefficients

D_{85}= 0.0254 D_{60}= 0.0102 D_{50}= 0.0073
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(12)

Remarks

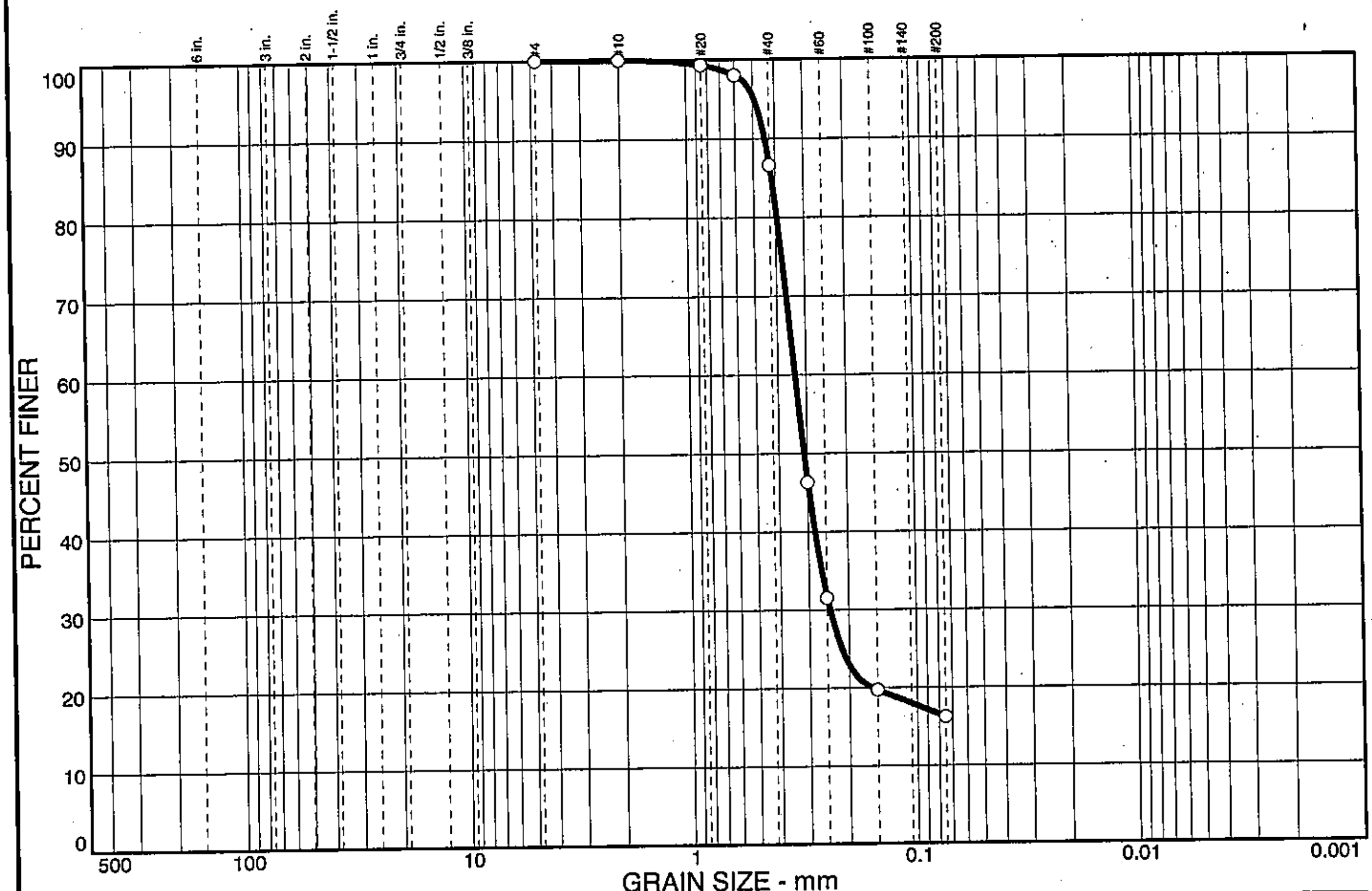
Moisture Content= 31.7%

Sample No.: 7 **Source of Sample:** B-1333 **Date:** 1/17/06
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	0.0	0.0	13.4	70.4	16.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.3		
#30	98.0		
#40	86.6		
#50	46.3		
#60	31.5		
#100	19.7		
#200	16.2		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.417 D_{60}= 0.337 D_{50}= 0.310
D_{30}= 0.243 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

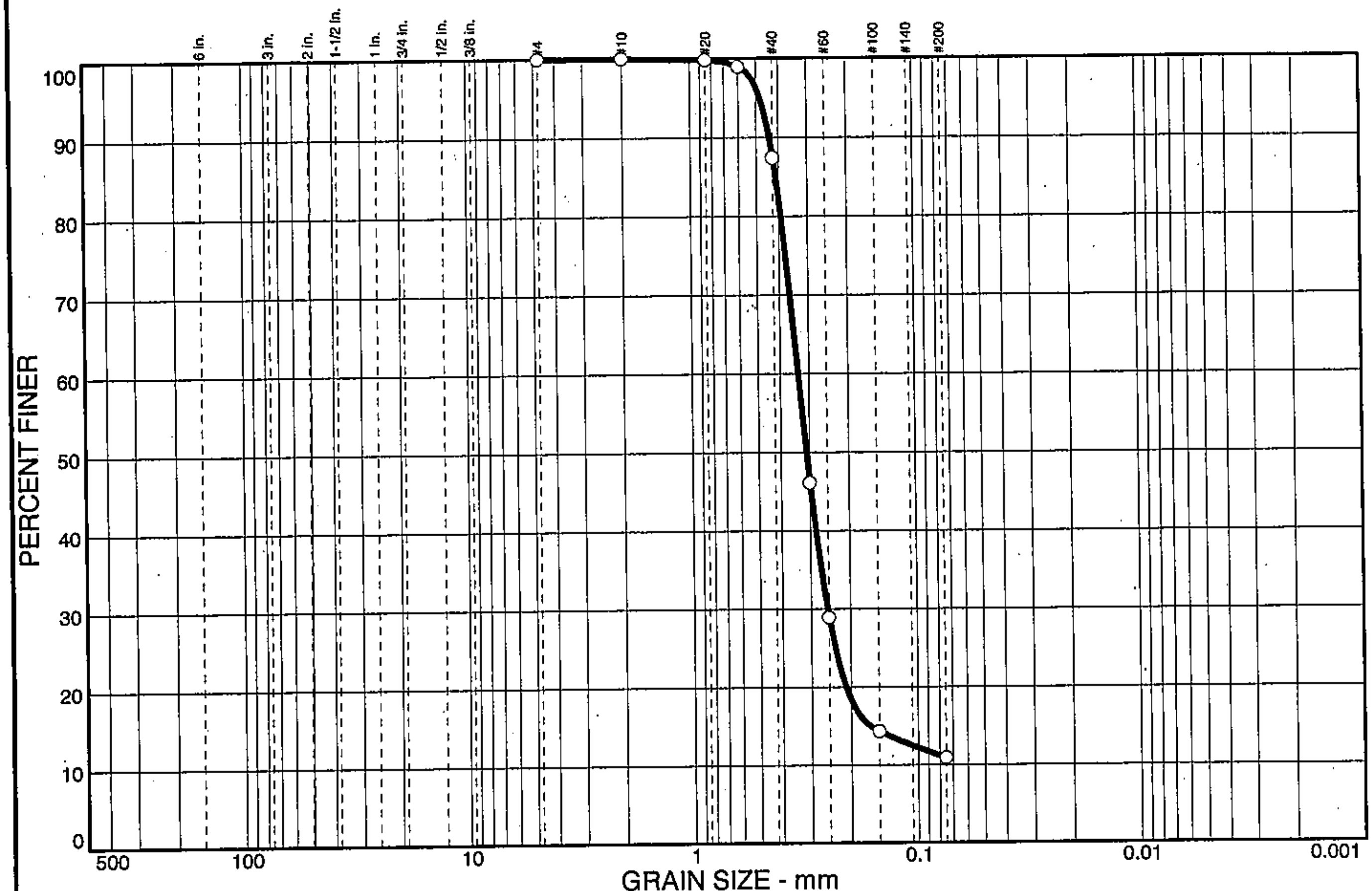
Moisture Content= 7.4%
F.M.=1.36

Sample No.: 2 **Source of Sample:** B-1334 **Date:** 1/17/06
Location: **Elev./Depth:** 2.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	12.6	76.5	10.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.8		
#30	98.9		
#40	87.4		
#50	46.1		
#60	28.9		
#100	14.3		
#200	10.9		

* (no specification provided)

Soil Description

Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.414 D_{60}= 0.335 D_{50}= 0.310
D_{30}= 0.254 D_{15}= 0.165 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO= A-2-4(0)

Remarks

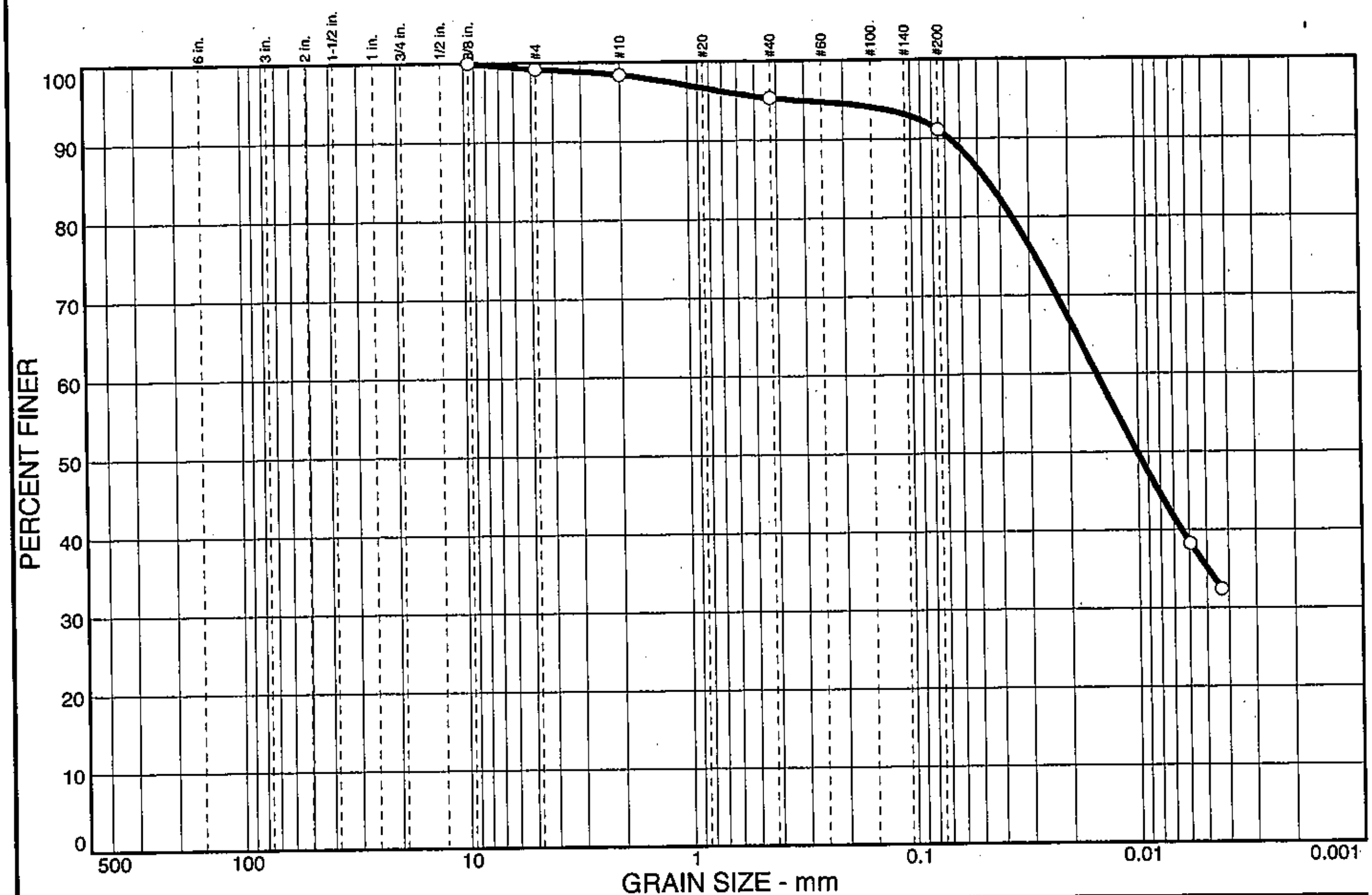
Moisture Content= 8.9%
F.M.=1.41

Sample No.: 3 **Source of Sample:** B-1334 **Date:** 1/17/06
Location: **Elev./Depth:** 4.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.8	0.8	3.1	4.2	56.0	35.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.2		
#10	98.4		
#40	95.3		
#200	91.1		

* (no specification provided)

Soil Description

Silt

Atterberg Limits

PL= 30 LL= 44 PI= 14

Coefficients

D_{85}= 0.0468 D_{60}= 0.0152 D_{50}= 0.0101
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-7-5(15)

Remarks

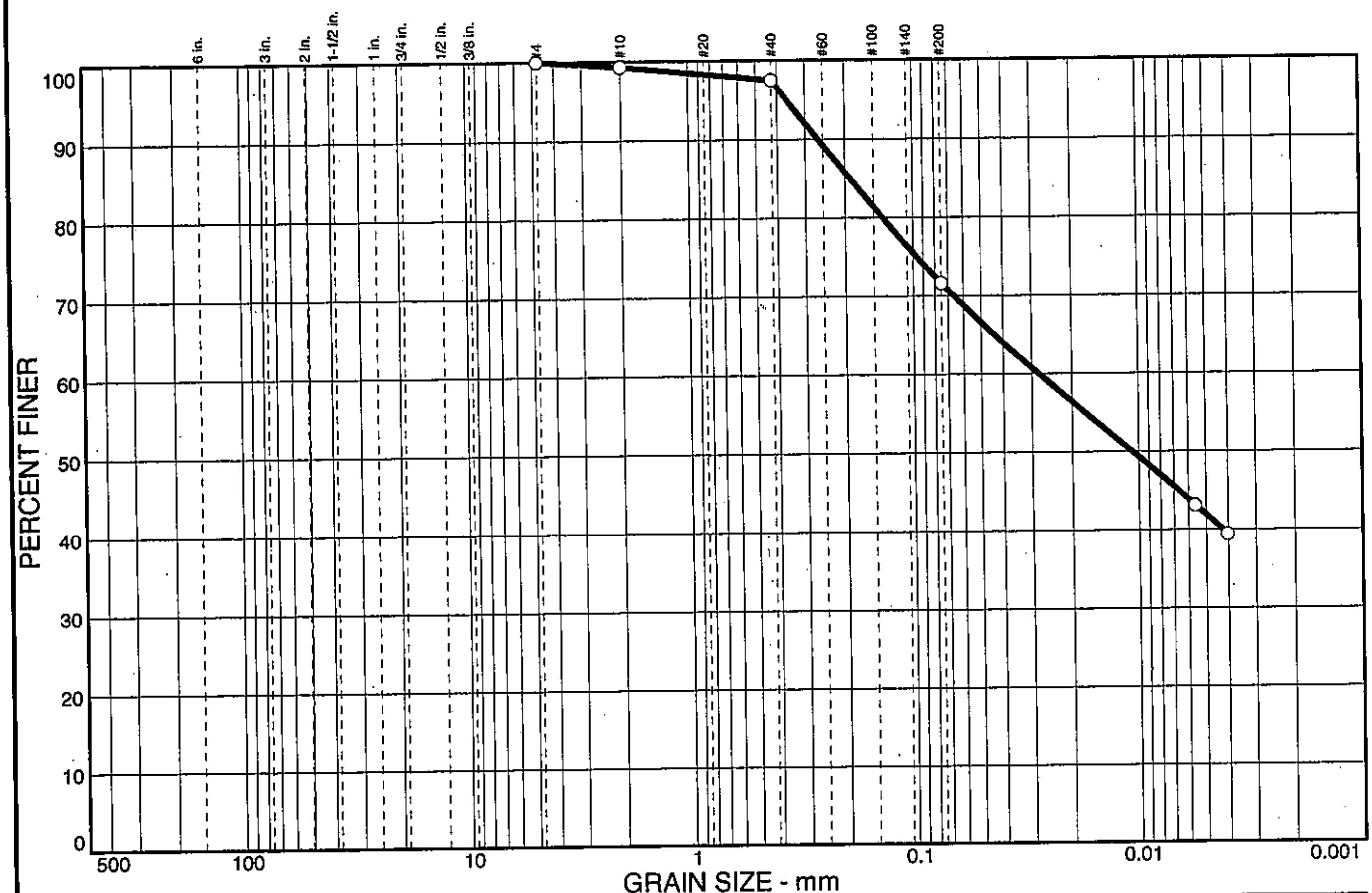
Moisture Content= 28.8%
F.M.=0.01

Sample No.: 5 **Source of Sample:** B-1334 **Date:** 1/17/06
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	0.7	1.8	25.9	29.8	41.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.3		
#40	97.5		
#200	71.6		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 14 LL= 28 PI= 14

Coefficients

D_{85}= 0.191 D_{60}= 0.0280 D_{50}= 0.0107
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(7)

Remarks

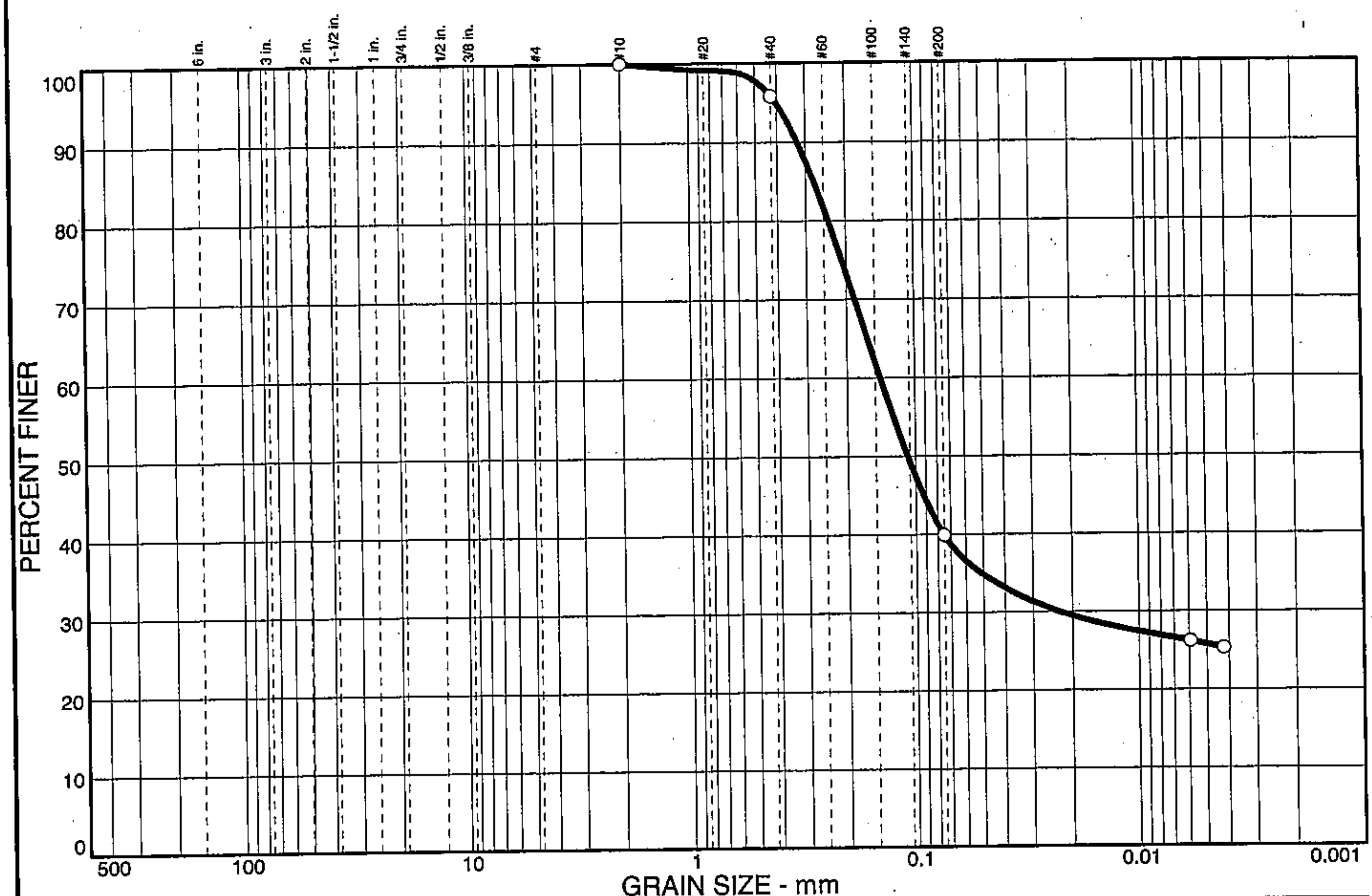
Moisture Content = 19.1%

Sample No.: 1 **Source of Sample:** B-1342 **Date:** 07/12/06
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	4.2	55.9	14.1	25.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	95.8		
#200	39.9		

* (no specification provided)

Soil Description

Clayey sand

Atterberg Limits

PL= 13 LL= 26 PI= 13

Coefficients

D_{85}= 0.277 D_{60}= 0.143 D_{50}= 0.109
D_{30}= 0.0228 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-6(1)

Remarks

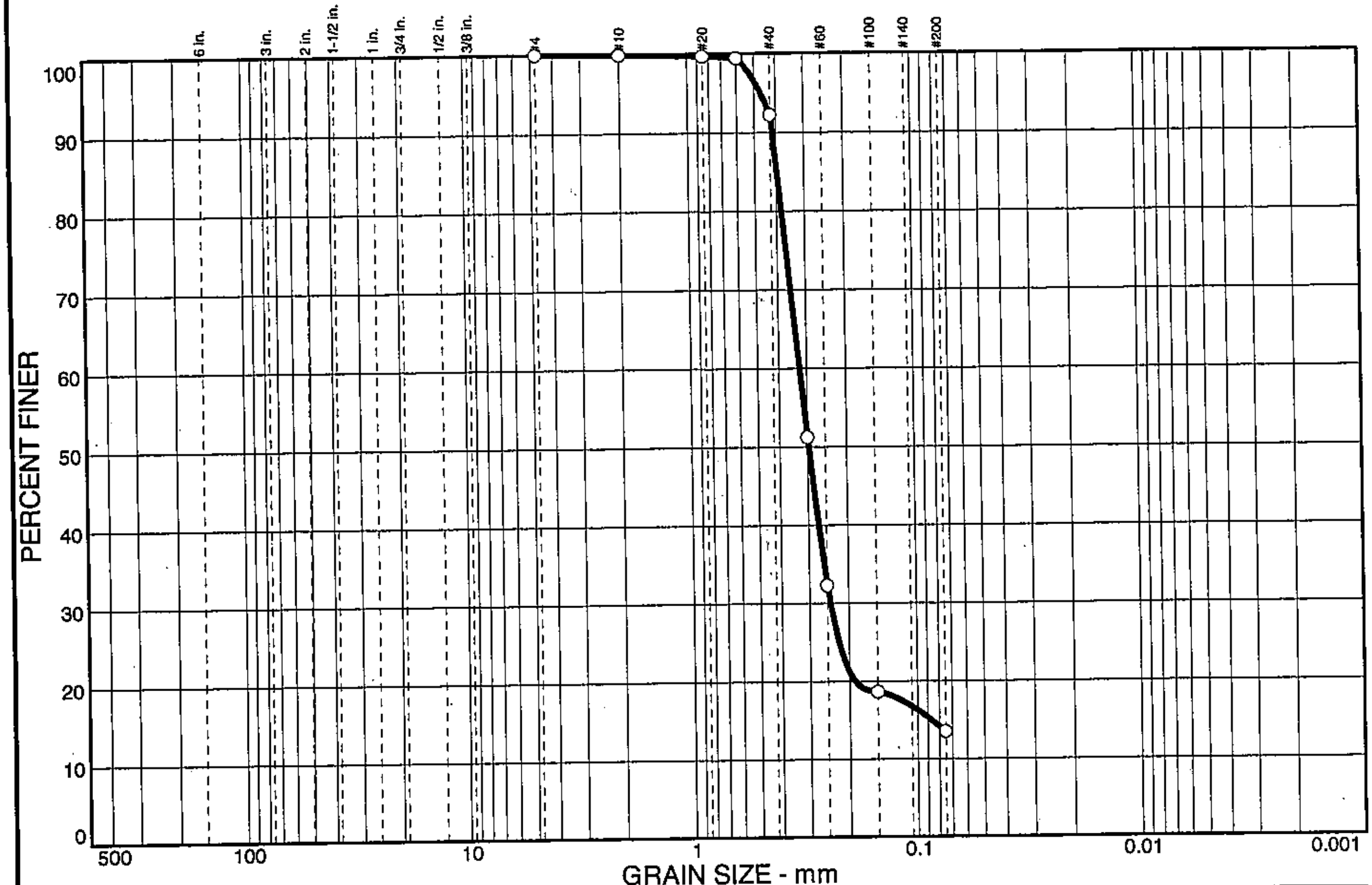
Moisture Content = 17.3%

Sample No.: 2 **Source of Sample:** B-1342 **Date:** 07/12/06
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	0.0	0.1	7.7	78.7	13.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.7		
#30	99.4		
#40	92.2		
#50	51.2		
#60	32.3		
#100	18.6		
#200	13.5		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.399 D_{60}= 0.323 D_{50}= 0.297
D_{30}= 0.243 D_{15}= 0.0873 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

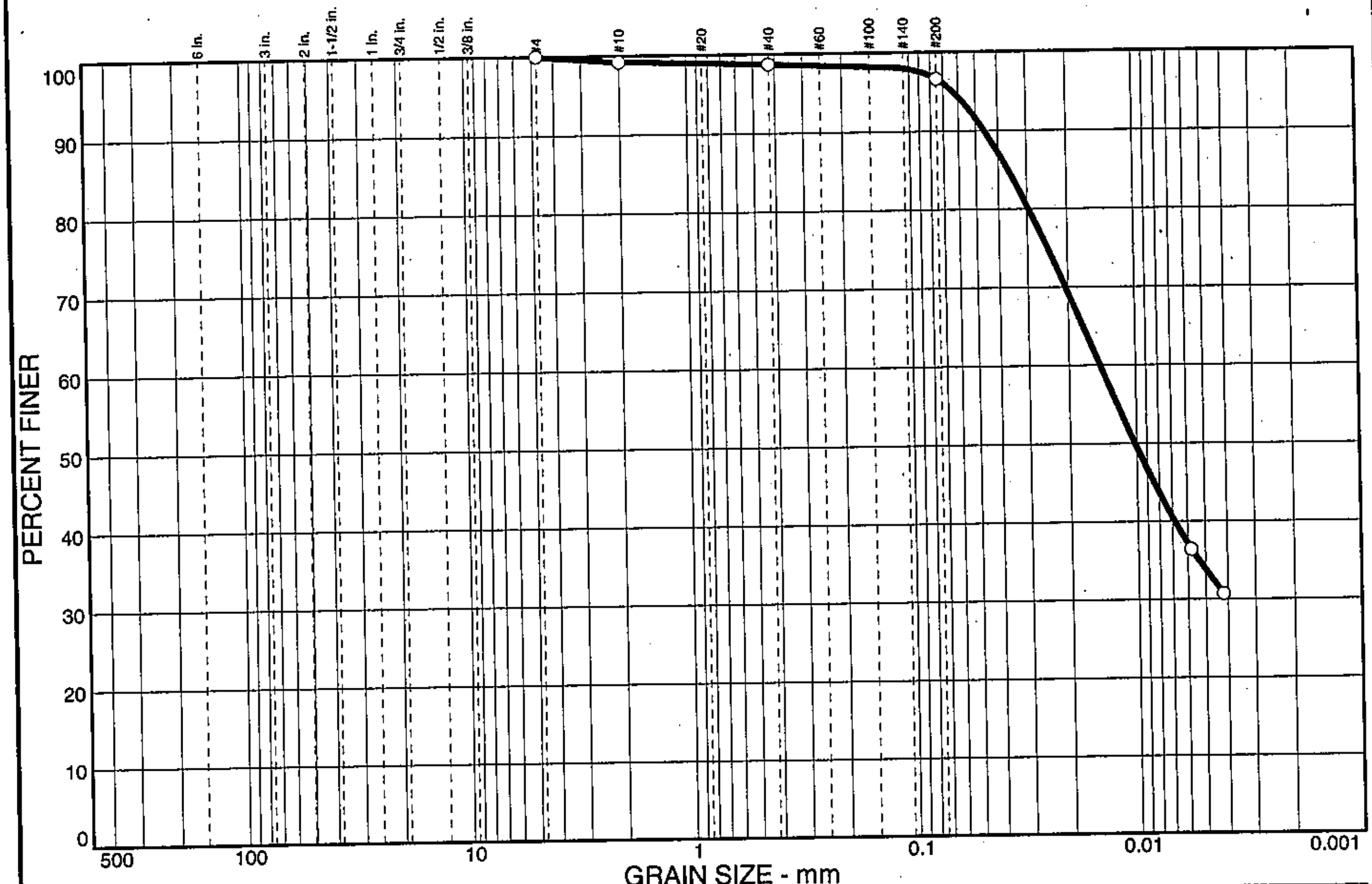
Moisture Content = 13.4%
F.M.=1.31

Sample No.: 3 **Source of Sample:** B-1342 **Date:** 07/12/06
Location: **Elev./Depth:** 6.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.8	0.5	2.1	62.9	33.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.2		
#40	98.7		
#200	96.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 22 LL= 38 PI= 16

Coefficients

D_{85}= 0.0364 D_{60}= 0.0146 D_{50}= 0.0102
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(17)

Remarks

Moisture Content = 29.5%

Sample No.: 6 **Source of Sample:** B-1342 **Date:** 07/12/06
Location: **Elev./Depth:** 13.5

DLZ

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

Project No: 0121-3070.03 **Figure**

APPENDIX IV
Calculations
Forward Abutment Profile Drawings

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	1 OF 2
Item	MSE Wall Stability	COMP. BY	SJR DATE 11/10/06
Relocated Shumway Hollow Rd over CSXT RR		CHECKED BY	DAA DATE 11-16-06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=40'

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 Assumed 3.0' depth to bedrock.

Wall Properties

$H+D$ = 43 feet

γ_{mse} = 120 pcf

L = 30.1 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)

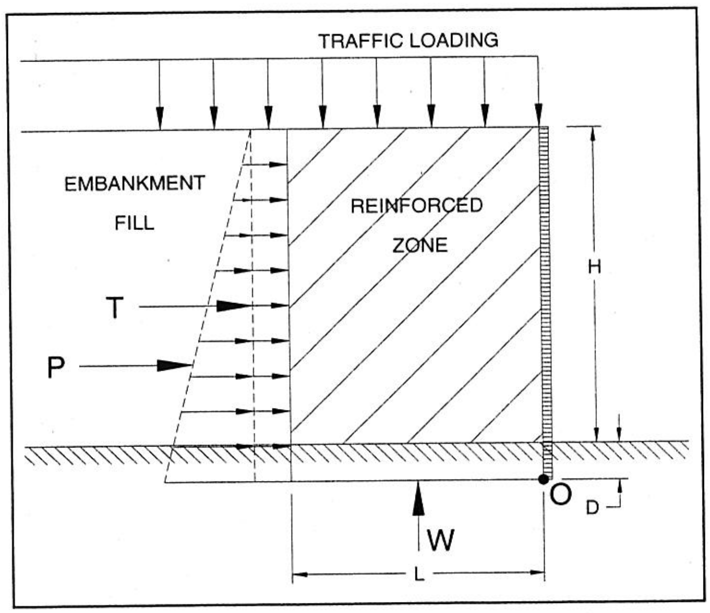
Friction Angle of Embankment Fill

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 = 40,016 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 = 69,892 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 = **1.75** 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

$\Sigma M_{resisting}$ = 2,337,506 lb-ft

$\Sigma M_{overturning}$ = 597,967 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 FS = **3.91** Required FS = 2.00

Resistance Against Overturning is **OK**

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Drilled Shaft - End Bearing & Side friction Shumway over CSXT Railroad

PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 2
COMP. BY SJR DATE 11-15-06
CHECKED BY DAA DATE 11-16-06

* From test on rock cores from boring B-1342; $q_u = 13{,}226$ psi
Rock core testing on cores from final borings will be performed to finalize these recommendations.

End Bearing FHWA-IF-99-025 Eqn 11.6 $q_{max}(MPa) = 4.83\left[q_u(MPa)\right]^{0.51}$
For RQD between 70-100 percent and $q_u > 0.5$ MPa (5.2 tsf)

$q_u = 13{,}226 \text{ psi} = 91.2 \text{ MPa}$

[Eqn 11.6] $q_{max} = 4.83\left[91.2 \text{ MPa}\right]^{0.51} = 48.3 \text{ MPa} = 6999 \text{ psi} = 1{,}008 \text{ ksf}$

$q_{allow} = \dfrac{q_{max}}{FS} = \dfrac{1{,}008 \text{ ksf}}{3.0} = 336 \text{ ksf}$

* For this type & quality of rock we typically use;

$q_{allow} = 80$ ksf (40 tsf) → Rear Abutment
$q_{allow} = 40$ ksf (20 tsf) → Forward Abutment

Side Friction FHWA-IF-99-025 Eqn 11.24 $f_{max} = 0.65\,p_a\left[q_u/p_a\right]^{0.5} \leq 0.65\,p_a\left[f_c'/p_a\right]^{0.5}$
- Assumes Smooth Rock Socket

$f_c' \simeq 4500$ psi $\quad q_u \simeq 13{,}226$ psi $\quad f_c'$ governs

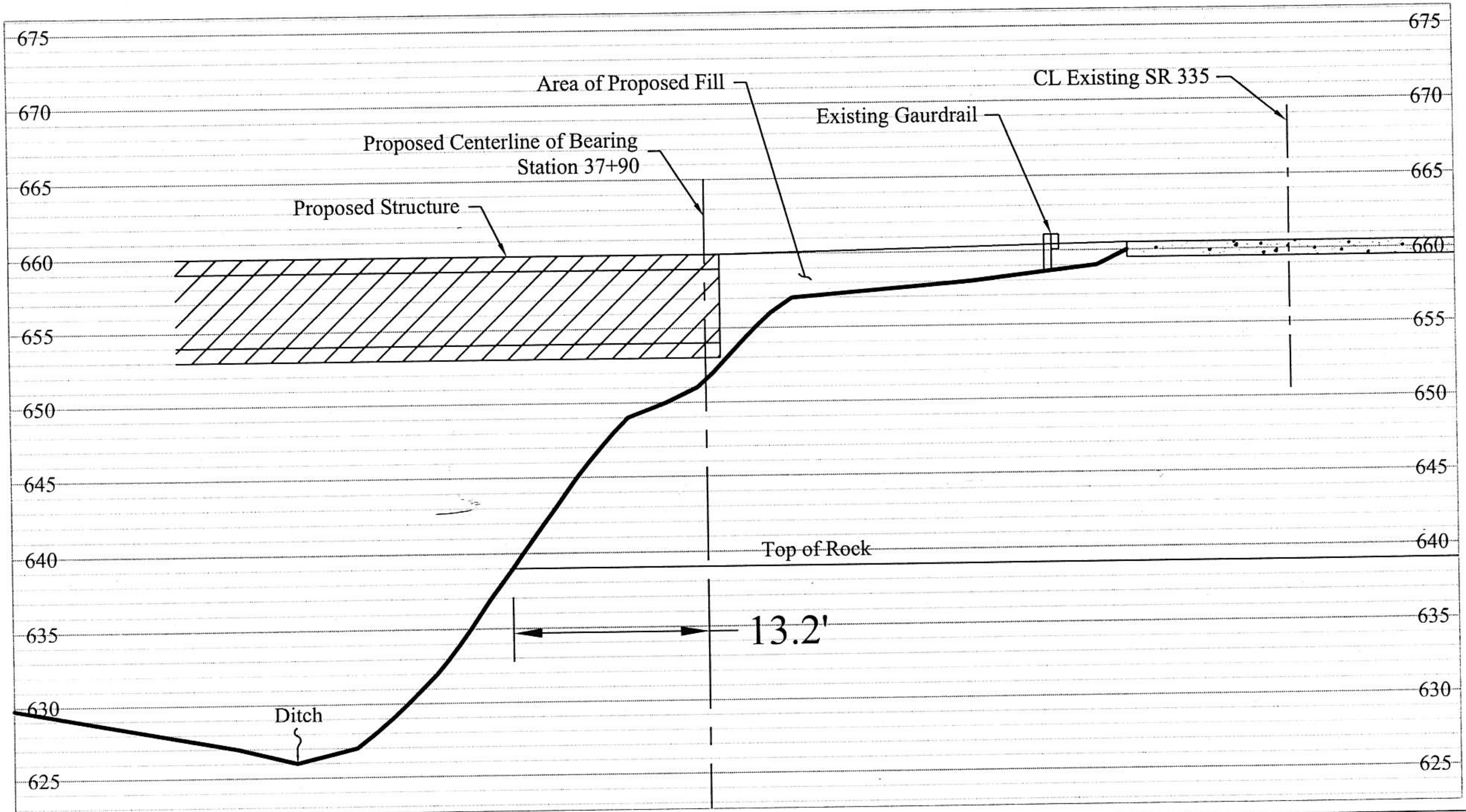
$f_{max} = 0.65\,(14.70 \text{ psi})\left[\dfrac{4500 \text{ psi}}{14.7 \text{ psi}}\right]^{0.5} = 167.2 \text{ psi}$

$f_{allow} = \dfrac{167 \text{ psi}}{3.0} = 55.6 \text{ psi} = 8006 \text{ psf}$

Use $f_{allow} = 7{,}500$ psf → Rear Abutment

Use $f_{allow} = 5{,}000$ psf → Forward Abutment

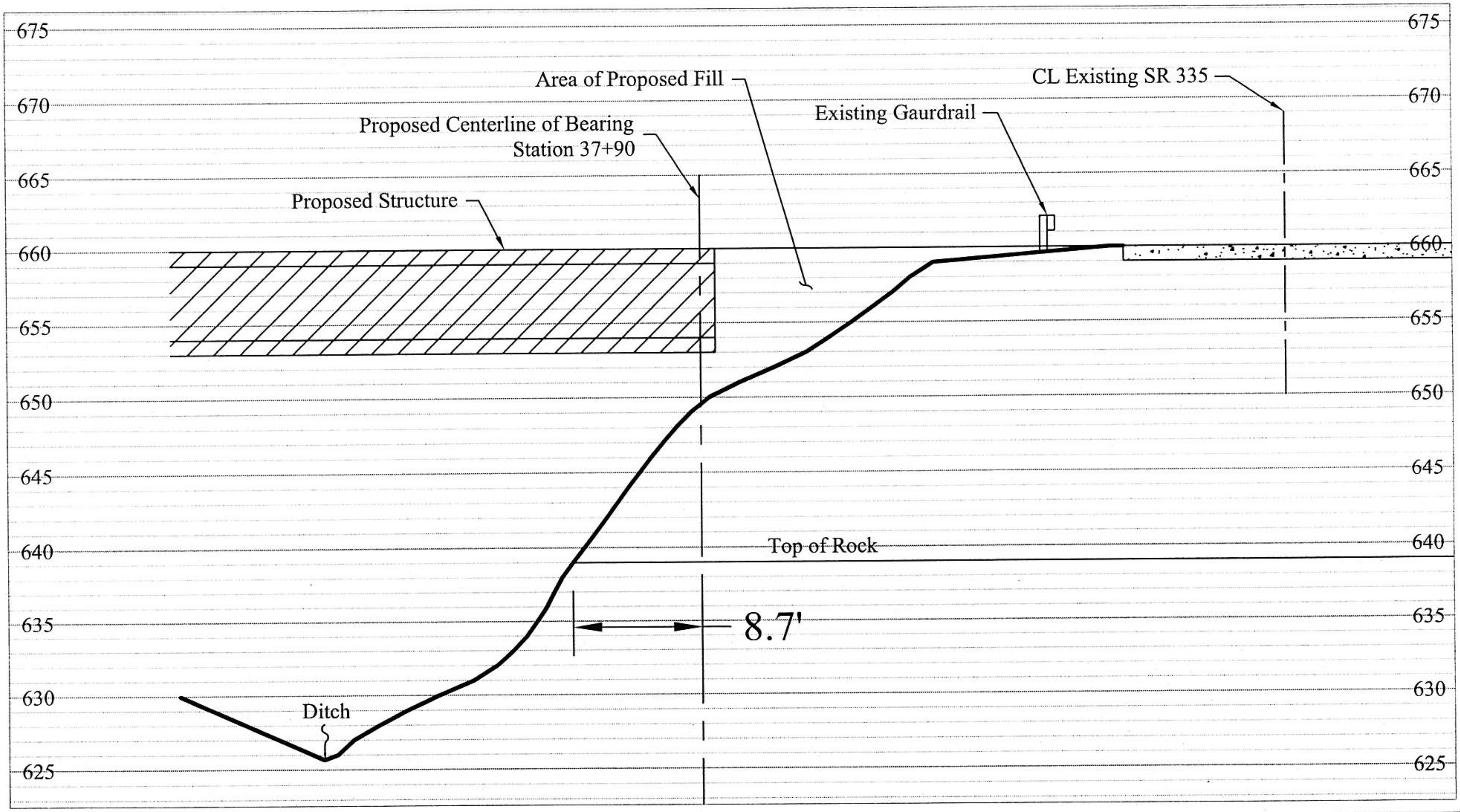
PROFILE (VIEW LOOKING NORTH)
RELOCATED SHUMWAY HOLLOW ROAD
25' RIGHT OFFSET



RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION 25' RIGHT OFFSET		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 11/10/06

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 11/17/2006 9:44:19 AM, \\Dlztree\Q_geotechhplj5100tn SCALE: 1"=10'

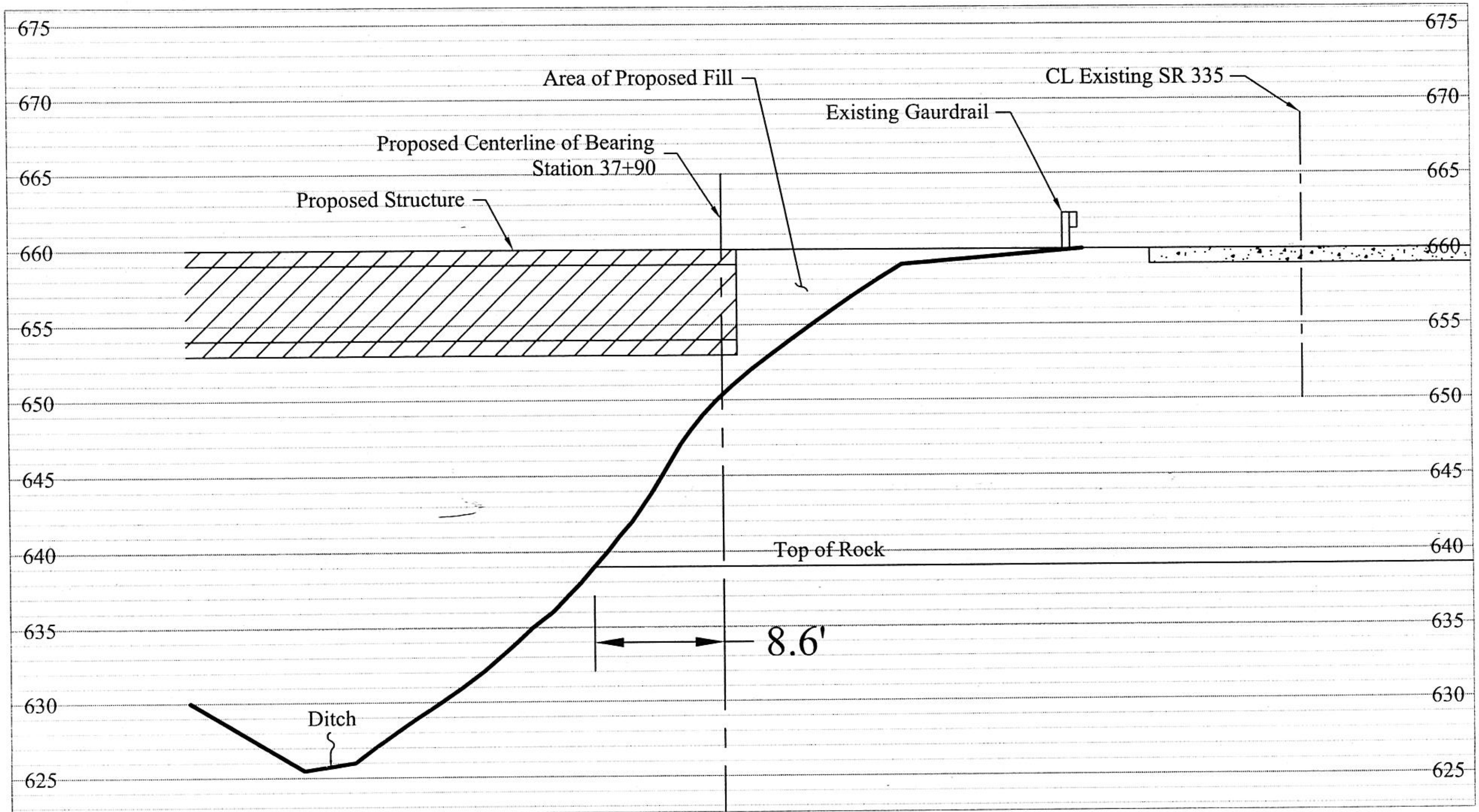
PROFILE (VIEW LOOKING NORTH)
RELOCATED SHUMWAY HOLLOW ROAD
ON BASELINE



RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION ON BASELINE		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 11/10/06

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 11/17/2006 9:44:24 AM, \\Dlztree\Q_geotechhplj5100tn SCALE: 1"=10'

PROFILE (VIEW LOOKING NORTH)
RELOCATED SHUMWAY HOLLOW ROAD
25' LEFT OFFSET



RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION 25' LEFT OFFSET		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 11/10/06

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 11/17/2006 9:44:32 AM, \\Dlztree\Q_geotechplj5100tn SCALE: 1'=10'