Report of:

Subsurface Exploration
SR 823 Bridge Over SR 335 & Little Scioto River
(Bridge No. SCI-823-0248 L & R)
SCI-823-0.00 Portsmouth Bypass (PID 77366)
Scioto County, Ohio

Prepared for:

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DLZ Job No. 0121-3070.03

February 20, 2008

Prepared by:

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REPORT

OF

SUBSURFACE EXPLORATION

FOR

SR 823 BRIDGE OVER SR 335 & LITTLE SCIOTO RIVER

(BRIDGE NO. SCI-823-0248 L & R)

PROJECT SCI-823-0.00 PORTSMOUTH BYPASS (PID 77366)

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TABLE OF CONTENTS

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1.0 INTRODUCTION

This report includes the findings of evaluations and recommendations for foundations of the proposed SR 823 bridge over the Little Scioto River and SR 335 between approximate stations 130+73 and 139+02. This bridge is planned as part of the Portsmouth Bypass project. Subsurface explorations performed for other features of the project are presented in separate reports.

Structure borings were planned and drilled for the "approved" type-study configuration based upon correspondence from the Office of Structural Engineering (OSE), dated January 31, 2007. The approved configuration consisted of twin four-span structures, with a 50-foot wall retaining wall at the forward abutment. Subsequent analyses and discussions led to changes in the design/layout of the proposed bridge. Based upon correspondence from OSE dated December 13, 2007, the preferred configuration will now be a four-span left and five-span right configuration. A retaining wall, which is approximately 30 feet tall, is proposed at the forward abutment of the left bridge.

Because of the change in the bridge layout, the borings drilled for the foundations of the "approved" type-study configuration are no longer located where the currently proposed foundations are planned. Due to the erratic nature of the subsurface conditions, particularly north of SR 335, it is strongly recommended that additional borings be drilled for the proposed foundation elements at Pier 4 and the Forward Abutment before final design is complete. As a result, the recommendations contained in this report for Pier 4 and the Forward Abutment of the right structure should be considered preliminary and interim until the subsurface conditions and recommendations can be verified by additional borings.

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

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

2.0 GENERAL PROJECT INFORMATION

The project consists in part of placing twin structures to carry proposed SR 823 over the Little Scioto River and SR 335. The left structure will be comprised of four spans while the right structure will consist of five spans. Due to the sloping bedrock surface at the forward abutment location, the abutment of the right (northbound) structure will be placed further up-station to eliminate the need for the retaining wall for the right structure.

The currently proposed configuration uses spill through slopes at the rear abutment while a retaining wall will be required at the forward abutment of the left structure to retain fill material and contain the abutment.

It is assumed that the maximum height of the embankment at station 130+75 (rear abutment) will be approximately 46 feet. This height is based upon the maximum difference between the proposed grade of SR 823 and the existing grade, as indicated on the Structure Site Plan, presented in Appendix I.

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

3.0 FIELD EXPLORATION

The field exploration consisted of drilling a total of thirteen borings for the proposed structure. Structure borings TR-30 through TR-35 and TR-35A were drilled for the initial/preliminary design configuration. These borings were drilled between February 24, 2005 and January 12, 2006. Borings B-39 through B-44 were drilled for the foundations of the "approved" type study configuration. As mentioned previously, the currently proposed bridge configuration differs from the "approved" type study configuration. These borings were drilled between May 9 and 21, 2007. The boring locations are presented on the Structure Site Plan, presented in Appendix I. Boring logs are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

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

4.0 FINDINGS

4.1 Geology of the Site

The project area in Highland Bend generally has gently rolling terrain and is bounded on either end by steep slopes. The main drainage feature in the valley is the Little Scioto River, located at approximately station 136+00. The ordinary high water elevation is reported to be 498.6 feet. The soil consists primarily of alluvial and lacustrine deposits. The overburden in this area is generally fine-grained soils, which are seventy to ninety feet deep. The area is located in the Shawnee-Mississippian Plateau, and can be found on the Minford 7.5-minute Quadrangle.

Bedrock is of the Mississippian Logan Formation. Generally, this formation consists of primarily sandstone or sandy siltstone with occasional areas of interbedded shale. However, the lithology of the sandstones varies both laterally and vertically. Within this area the Logan Formation typically consists of thick, massive sandstone units.

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. Results of Atterburg limits and moisture test results are presented on the boring logs and results from strength and consolidation testing are presented in Appendix III.

4.2.1 Soil Conditions

Rear Abutment
Boring TR-35A, drilled for the rear abutment of the proposed structure encountered 3 inches of gravel (driveway) at the ground surface. Below the surface material, cohesive soils, consisting of silt (A-4b) to clay (A-7-6) were generally encountered to a depth of 72.0 feet below the ground surface. Below these layers, fine sand (A-3) was encountered to a depth of 81.0 feet, at the top of bedrock.

Pier 1
Boring TR-35, drilled for the Pier 1 foundations generally encountered 4 inches of topsoil at the ground surface. Below the topsoil, cohesive soils, consisting of silt and clay (A-6a) to clay (A-7-6) were encountered to a depth of 63.0 feet below the ground surface. Below these layers, cohesionless soils consisting of silt (A-4b) and sandy silt (A-4a) were encountered to a depth of 80.0 feet, at the top of bedrock.

Pier 2
Borings TR-33 and B-39, drilled for the Pier 2 foundations generally encountered 2 to 5 inches of topsoil at the ground surface. Below the topsoil, cohesive soils, consisting of silt (A-4b) and silty clay (A-6b) were encountered to depths ranging from 19.5 to 21.0 feet below the ground surface. Below these layers, cohesionless soils, consisting of silt (A-4b) to coarse and fine sand (A-3a) were encountered to depths ranging from 33.7 to 34.1 feet, at the top of bedrock.

Pier 3
Borings B-40 and B-41, drilled for the Pier 3 foundations generally encountered 4 to 5 inches of topsoil at the ground surface. Below the topsoil, cohesive soils, consisting of silt (A-4b) to silty clay (A-6b) were encountered to depths ranging from 40.0 to 48.5 feet below the ground surface. Below these layers, boring B-40

encountered fine sand (A-3) to a depth of 48.5 feet, at the top of bedrock. However, boring B-41 encountered bedrock below the cohesive soil layers, at a depth of 40.0 feet.

Pier 4 (right structure only)
Although boring B-43 was drilled for the forward abutment retaining wall of a previously proposed configuration, this boring will be used to develop interim recommendations for the Pier 4 Rt foundations. Due to the steeply sloping bedrock in this area, it is strongly suggested that additional borings be drilled to verify the location and character of the bedrock in this location prior to finalizing the bridge design.

Boring B-43 encountered 5 inches of topsoil at the ground surface. Below the topsoil, medium dense to dense gravel with sand and silt (A-2-4) was encountered to a depth of 8.0 feet below the ground surface, at the top of weathered bedrock.

Forward Abutment (left structure)
Boring B-42, drilled for the forward abutment of left structure encountered 6 inches of topsoil at the ground surface. Below the topsoil, a thin layer of sandy silt (A-4a) was encountered to a depth of 2.0 feet below the ground surface, at the top of weathered bedrock.

Forward Abutment (right structure)
Boring B-44, drilled for the forward abutment of the right structure encountered 4 inches of topsoil at the ground surface. Below the topsoil, sandy silt (A-4a) and silt (A-4b) was encountered to a depth of 23.5 feet at the top of bedrock. Due to the steeply sloping bedrock in this area and the rapidly changing subsurface conditions, it is strongly suggested that additional borings be drilled to verify the location and character of the bedrock in this location prior to finalizing the bridge design.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was confirmed by coring in all borings. The bedrock consisted of medium hard, moderately to slightly weathered sandstone. The amount of rock recovered in each core run varied between 80 and 100 percent, with an average recovery of 99 percent. The rock quality designation (RQD) of the bedrock ranged between 25 and 100 percent with an average of 87 percent, indicating "good" quality rock.

Unconfined compressive strength of tested rock cores ranged between 1,590 and 12,571 pounds per square inch (psi). A summary of the unconfined compressive strength of the tested cores is shown in Table 1. A 30-foot tall retaining wall is currently proposed at the forward abutment location to retain the fill, and support the forward abutment of the left bridge. Anticipating the possibility that the retaining wall foundation will need to be designed for lateral loading, the elastic modulus of selected cores was also measured. The results of these tests are presented in Appendix III.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-39	39.9-40.3	143.9	10,509
B-39	44.7-45.3	144.1	10,030
B-40	54.0-54.6	150.1	11,884
B-40	63.0-63.4	149.4	10,798
B-41	43.9-44.3	152.0	9,810
B-41	53.5-53.9	149.8	12,318
B-42	10.7-11.2	147.7	2,636
B-42	18.6-19.0	149.6	1,590
B-42	30.6-31.0	139.6	11,230
B-42	36.0-36.5	141.2	10,898
B-43	13.4-13.9	138.6	7,886
B-43	18.1-18.7	151.9	11,204
B-43	25.0-25.5	149.8	10,575
B-43	31.4-31.8	151.7	12,571
B-44	39.5-40.0	135.0	10,256
B-44	34.5-35.0	144.5	9,714

4.2.3 Groundwater Conditions

Borings B-39 through B-41, TR-32 through TR-35, and TR-35, drilled for the rear abutment, Pier 1, Pier 2, and Pier 3 encountered seepage. Where seepage was encountered, it was first observed at depths ranging from 4.0 to 34.7 feet below the ground surface. Water was used during rock coring, which masked any seepage zones that might exist in the rock. A measurable water level in the borings prior to rock coring was encountered in borings B-39, B-41, and TR-32 through TR-35, and TR-35A. In these borings, water levels prior to coring rock were observed from approximate depths of 7.0 and 67.8 feet below the ground surface. Measurable water levels, upon the completion of coring, were present in all borings from approximate depths of 23.5 and 43.7 feet below the ground surface. It should be noted that the final water levels included drilling water, and consequently, may not be representative of actual groundwater conditions.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

The Highland Bend area traverses a fairly wide valley with deep, highly compressible soils. A series of three bridges, including the SR 823 Bridge over the Little Scioto River and SR 335, are proposed to be constructed across the valley. Three new embankment sections are also proposed between approximate station 105+75 and station 130+73. The findings pertaining to the embankment sections are presented in a report titled "Report of Subsurface Exploration for Proposed Highland Bend Roadway Embankments", dated August 2, 2007, prepared by DLZ. This report will be referred to as "Highland Bend Report" hereafter. Note that the recommendations made pertaining to these adjacent embankment sections were also considered in the analyses of the SR 823 Bridge over the Little Scioto River and SR 335. Calculations are presented in Appendix IV.

Because of the change in the bridge layout, the borings drilled for the foundations of the "approved" type-study configuration are no longer located where the currently proposed foundations are planned. Due to the erratic nature of the subsurface conditions, particularly north of SR 335, it is strongly recommended that additional borings be drilled for the proposed foundation elements at Pier 4 and the Forward Abutment before final design is complete. As a result, the recommendations contained in this report for Pier 4 and the Forward Abutment of the right structure should be considered preliminary and interim until the subsurface conditions and recommendations can be verified by additional borings.

5.1 General Foundation Recommendations

After the type-study review, it was determined that twin, four-span structures should be used to traverse the Little Scioto River and SR 335. However, it is understood that the high estimated construction costs for the tall (approximately 50 feet) abutment retaining wall and associated foundations led to a design change. Due to the sloping bedrock near the forward abutment location, the right (northbound) structure was lengthened in order to reduce the required height of a retaining wall to be positioned at the forward abutment. Consequently, the left (southbound) structure will be 4-span, while the right (northbound) structure will be 5-span. This newly proposed design places the forward abutment of the left bridge at approximate station 138+99. Similarly, for the right bridge, Pier 4 and the forward abutment will be placed at approximate stations 138+75 and 140+27, respectively.

It is understood that driven piles are preferred to support the rear abutment and Pier 1 of the proposed bridge. However, drilled shafts, founded in sandstone bedrock are preferred to support Piers 2 and 3 of the proposed bridge. Pier 4 of the right bridge may be supported using either a spread footing or system of drilled shafts, bearing on bedrock.

At the forward abutment for the left bridge, it is understood that drilled shafts will be used to support the abutment as well as resist lateral loads applied to the proposed abutment retaining wall.

The retaining backwall, running essentially parallel to the alignment, will be situated on a sloping bedrock surface. Based upon the available boring information, it is likely that a portion of the retaining wall may be supported on spread footings founded in rock, while the remaining portion will be supported on drilled shafts, founded in rock.

A summary of the bridge foundation recommendations is presented in Table 2. It should be noted that the bedrock surface varies widely across the project area. If the subsurface conditions encountered while installing the foundations are different than those assumed, the actual bearing elevations may differ from those cited in Table 2.

Table 2 - Summary of Foundation Recommendations

Structural Element	Structure / Boring	Existing Ground Surface Elevation *(Feet)	Foundation Type	Approximate Bearing Elevation *(Feet)	Allowable Bearing Capacity
Rear Abutment	Left / TR-35A	554.6	HP 14X73 piles	473.6	95 tons
	Right / TR-35A	554.6	HP 14X73 piles	473.6	95 tons
Pier 1	Left / TR-35	552.8	HP 14X73 piles	472.8	95 tons
	Right / TR-35	552.8	HP 14X73 piles	472.8	95 tons
Pier 2	Left / TR-33	502.7	Drilled Shafts	(1)463.6	80 ksf[+]
	Right / B-39	508.0	Drilled Shafts	(1)466.7	80 ksf[+]
Pier 3	Left / B-40	529.0	Drilled Shafts	(2)471.5	80 ksf[+]
	Right / B-41	518.9	Drilled Shafts	(2)470.9	80 ksf[+]
Pier 4	Right / B-43	Variable	Drilled Shafts	(3) Variable	80 ksf[+]
			Spread Footings[++]	557.6	20 ksf
Forward Abutment	Left / B-42	Variable	Drilled Shafts	(4) Variable	(4) Variable
	Right / B-44	Variable	Drilled Shafts	(5) Variable	(5) Variable

* Cited elevations are encountered at the boring locations except where cited.

+Allowable end bearing capacity only. Refer to subsequent sections for additional details.

++Elevation based upon interpolation using all available boring and contour information.

(1)Minimum rock socket length of 8 ft. Refer to Section 5.4.

(2)Minimum rock socket length of 9 ft and 8 ft, left and right structures, respectively. Refer to Section 5.5.

(3)Minimum rock socket length of 7 ft. Refer to Section 5.6.

(4)End bearing and side friction varies with depth. Refer to Sections 5.7 and 5.9.

(5)Minimum rock socket length of 7 ft. End bearing and side friction varies with depth. Refer to Section 5.8.

5.2 Bridge Foundation Recommendations – Rear Abutment (L & R)

Based upon the Structure Site Plan drawings, it is understood that the rear abutment of the proposed bridge will be placed at station 130+75.

With respect to the foundation and settlement analyses, the subsurface conditions encountered by boring TR-35A were assumed to be representative of conditions in the vicinity of the rear abutment. Based upon the subsurface conditions encountered by the borings, it is recommended that driven piles be used to support the rear abutment of the proposed structure. The use of HP 12x53, HP 14x73, 14-inch CIP, and 16-inch CIP piles were analyzed. The results of static analyses indicate that the piles will develop the required ultimate resistance as friction piles. However, because bedrock is within 10 to 16 feet below the estimated pile tip elevations, it is recommended that consideration be given to driving piles to the top of bedrock, at approximate elevation 473.6. If piles are driven to bedrock, it is recommended that HP 12x53 or HP 14x73 piles be used. At this time, uplift is not anticipated at the rear abutment location. However, if required, the allowable uplift capacity of the piles can be provided upon request.

Although the underlying bedrock is hard, because the piles will be driven through new embankment fill as well as approximately 81 feet of in-situ soils, the use of reinforced piles points may not be necessary according to ODOT's Bridge Design Manual.

It should also be noted that the borings encountered fine sand and silt layers. When saturated, these layers may produce exaggerated blow counts during pile driving which do not reflect the actual load carrying ability of the strata. Therefore, piles should be driven to refusal, and then redriven to refusal after the excess pore pressures near the pile tip have had time to dissipate (usually less than 24 hours).

It is assumed that spill-through slopes characterized by 2.5H:1V or flatter slopes will be used at the rear abutment location. Based upon the provided Structure Site Plans, the maximum height of the embankment at the proposed rear abutment is assumed to be approximately 46 feet. The proposed embankment should be built using staged construction as mentioned in Section 5.12 of this report, and according to the recommendations cited in the Highland Bend Report.

The in-situ foundation soils will consolidate under the influence of the new embankment fill. Settlement calculations indicate that approximately 37 inches of settlement will occur at the rear abutment location. Time-rate of settlement calculations indicate that the in-situ foundation soils would consolidate to ninety percent (U=90%) in approximately 6 years without wick drains or other means to accelerate the consolidation process. However, due to the need for staged construction of the Highland Bend mainline embankments, it is anticipated that wick drains will be used under the embankment at the proposed rear abutment.

To prevent downdrag forces from reducing the allowable capacity of the piles, it is recommended that a waiting period be observed prior to driving piles at the rear abutment location. Calculations indicate that piles should not be driven until approximately 99 percent of total primary consolidation has occurred. Estimates of the time to 90 and 95 percent consolidation (with wick drains) are presented in Table 3. However, it should be noted that these values are estimates only. Time-rate of consolidation estimates, particularly with the use of wick drains beyond 90 percent are unreliable. Additionally, the staged construction of this embankment further complicates the determination of the time required to reach a specified percentage of consolidation. The ODOT construction representative responsible for monitoring the settlement platforms and pore pressures in the foundation soils should determine when approximately 99 percent consolidation has occurred, and modify the required waiting period accordingly.

Table 3 – Time rate of Consolidation Estimates Using Wick Drains

Location	Total Settlement (in)+	Wick Drain Spacing (ft)*	t_{90} (days)	t_{95} (days)
Rear Abutment	37	5	35	50
		7	65	95
		9	105	150

* *Assumes triangular grid spacing. See Highland Bend Embankment Report (8-2-07) for details.*
\+ *Settlement due to embankment loading only.*

5.3 Bridge Foundation Recommendations – Pier 1 (L & R)

Based upon the Structure Site Plan drawings, it is understood that Pier 1 of the proposed bridges will be placed at approximate stations 132+38 and 132+25 for the left and right structures, respectively.

Borings TR-35 and TR-35A were both considered in the evaluation of the Pier 1 foundations. However, with respect to the foundation and settlement analyses, the subsurface conditions encountered by boring TR-35A were assumed to be representative of conditions in the vicinity of the Pier 1. Based upon the subsurface conditions encountered by the borings, it is recommended that driven piles be used to support Pier 1 of the proposed left and right structures. The use of HP 12x53, HP 14x73, 14-inch CIP, and 16-inch CIP piles were analyzed. The results of static analyses indicate that the piles will develop the required ultimate resistance as friction piles. However, because bedrock is within 9 to 15 feet below the estimated pile tip elevations, it is recommended that consideration be given to driving piles to the top of bedrock, at approximate elevation 472.8. If piles are driven to bedrock, it is recommended that HP 12x53 or HP 14x73 piles be used. At this time, uplift is not anticipated at the rear abutment location. However, if required, the allowable uplift capacity of the piles can be provided upon request.

Although the underlying bedrock is hard, because the piles will be driven through new embankment fill as well as approximately 80 feet of in-situ soils, the use of reinforced piles points may not be necessary according to ODOT's Bridge Design Manual.

Because no embankment fill will be placed near the Pier 1 location, the effect of downdrag forces acting on the piles is considered to be negligible, and does not need to be accounted for. Consequently, no waiting period will be required prior to driving piles at the Pier 1 location.

It should be noted that the borings encountered fine sand and silt layers. When saturated, these layers may produce exaggerated blow counts during pile driving which do not reflect the actual load carrying ability of the strata. Therefore, if piles are not driven to bedrock, piles should be driven to refusal, and then redriven to refusal after the excess pore pressures near the pile tip have had time to dissipate (usually less than 24 hours).

5.4 Bridge Foundation Recommendations – Pier 2 (L & R)

Based upon the Structure Site Plan drawings, it is understood that Pier 2 of the proposed bridges will be placed at approximate stations 134+41 and 134+25 for the left and right structures, respectively.

Based upon the subsurface conditions encountered by the borings and the anticipated loading conditions at Pier 2, it is recommended that a system of drilled shafts be used to support Pier 2 of the proposed left and right structures. The drilled shafts should be founded in the underlying moderately to slightly weathered sandstone bedrock. Assuming a minimum rock socket length of 8 feet for both the left and right piers, the drilled shafts may be designed using an allowable bearing pressure of 80 ksf (40 tsf). A layer of highly weathered, argillaceous sandstone was encountered at the top of rock. The required minimum 8-foot embedment into the rock is intended to place the drilled shaft tips approximately 1 foot below the weathered, argillaceous zone encountered in the borings. The recommended embedment depth (for end-bearing support) was determined by plotting the top of rock and weathered zones within the rock on cross section drawings. Considering the sloping bedrock and variable thickness of the weathered zone, a minimum recommended embedment was selected to ensure uniform bearing material is encountered at the proposed drilled shaft tip elevations.

Alternatively, drilled shafts may be designed as friction-type shafts. Neglecting the overburden and upper two feet of the rock socket, an allowable sidewall shear stress/adhesion of 7,500 pounds per square foot (psf) may be used for the rock socket.

5.5 Bridge Foundation Recommendations – Pier 3 (L & R)

Based upon the Structure Site Plan drawings, it is understood that Pier 3 of the proposed bridges will be placed at approximate stations 136+96 and 136+75 for the left and right structures, respectively.

Based upon the subsurface conditions encountered by the borings and the anticipated loading conditions at Pier 3, it is recommended that a system of drilled shafts be used to support Pier 3 of the proposed left and right structures. The drilled shafts should be founded in the underlying moderately weathered sandstone bedrock. Assuming a minimum rock socket length of 9 feet and 8 feet for the left and right pier foundations, respectively, the drilled shafts may be designed using an allowable bearing pressure of 80 ksf (40 tsf). A layer of highly weathered, argillaceous sandstone was encountered at the top of rock. The required minimum embedment into the rock is intended to place the drilled shaft tips approximately 1 foot below the weathered, argillaceous zone encountered in the borings. The recommended embedment depth (for end-bearing support) was determined by plotting the top of rock and weathered zones within the rock on cross section drawings. Considering the sloping bedrock and variable thickness of the weathered zone, a minimum recommended embedment was selected to ensure uniform bearing material is encountered at the proposed drilled shaft tip elevations.

If adequate axial capacity cannot be developed with reasonable shaft diameter, drilled shafts could be designed as friction-type shafts. Neglecting the overburden and upper two feet of the rock socket, an allowable sidewall shear stress/adhesion of 7,500 pounds per square foot (psf) may be used for the rock socket.

Given the Pier 3 plan locations, it is anticipated that significant excavations will be required to construct the foundations and pier columns. Due to the proximity of Pier 3 to the Little Scioto River and SR 335, a temporary retaining wall will likely be required. For information concerning any temporary retaining structure at Pier 3, refer to Section 5.11.

5.6 Bridge Foundation Recommendations – Pier 4 (R)

It is understood that Pier 4 of the right bridge will be placed at approximate station 138+75. It should be noted that the previous bridge configuration placed the forward abutment near this location. In addition, a retaining wall approximately 50 feet tall was proposed to retain the fill at the right corner of the abutment. The foundations needed to support the abutment and resist the lateral loads generated by such a high wall needed to be very large, and heavily reinforced against shear failure. As a result, representatives of ODOT's Office of Structural Engineering and Transystems decided to extend the right bridge structure to reduce the height of the abutment retaining wall.

Based upon subsurface conditions encountered by the borings and the anticipated loading conditions, a spread footing founded on moderately weathered sandstone bedrock could be considered to support Pier 4 of the right bridge. The Structure Site Plan, provided by Transystems indicates that the proposed bottom of footing should be placed at elevation 565.0. Although a boring was not drilled specifically for this new pier location, an approximate top of rock profile was estimated based upon available boring information and contour mapping. Based upon the available information, the right-rear corner of the footing will encounter the top of rock at the lowest elevation, at approximately elevation 559.6. Based upon the information from the adjacent borings, it is anticipated that a layer of severely weathered bedrock, which is approximately 2 feet thick is present at the Pier 4 location. If the footing is not stepped, the bottom of the footing should be placed at approximate elevation 557.6. Consequently, the recommended footing elevation reflects an embedment into the bedrock of approximately 2 feet. The spread footing at the anticipated bearing elevation (557.6) may be designed using an allowable bearing pressure of 20 ksf (10 tsf). For resistance against sliding, the coefficient of friction between the rock and concrete may be taken to be 0.55. Higher values of the friction coefficient could be used for this type of rock. However, due to the sloping bedrock surface and the potential for laminations and clay filled factures to control the sliding condition, a lower value was conservatively selected.

Note that due to sloping bedrock surface, the bearing depth, below the top of rock, could vary from approximately 2 feet to over 20 feet. As a result, the weathering and strength characteristics of the rock will vary across the footing.

It should be anticipated that due to the sloping ground surface excavations in rock as deep as 23 feet or more may be required at the left-forward corner of the proposed footing. This deep could reduce the geotechnical resistance of the drilled shafts supporting the proposed forward abutment retaining wall of the left structure. If spread footings are considered, additional borings at the Pier 4 location are necessary to determine the top of bedrock elevations.

Alternatively, it is recommended that Pier 4 be supported using a system of drilled shafts. The use of drilled shafts to support the pier would avoid the detrimental effect of possible deep footing excavations as indicated in the previous paragraph. The drilled shafts should be founded in the underlying sandstone bedrock. Assuming a minimum rock socket length of 7 feet, the drilled shafts may be designed using an allowable bearing pressure of 80 ksf (40 tsf). A 2-foot thick layer of weathered sandstone was encountered at the top of rock. The required minimum 7-foot embedment into the rock is intended to place the drilled shafts 5 feet into the competent bedrock.

If adequate axial capacity cannot be developed with reasonable shaft diameter, drilled shafts could be designed as friction-type shafts. Neglecting the overburden and upper two feet of the rock socket, an allowable sidewall shear stress/adhesion of 7,500 pounds per square foot (psf) may be used for the rock socket.

5.7 Bridge Foundation Recommendations – Forward Abutment (L)

It is understood that the forward abutment of the left bridge will be placed at approximate station 138+99. Also, a retaining wall will be required at the abutment to contain the new embankment fill (at the proposed abutment) as the roadway transitions into a rock cut. From the plans, it is understood that the maximum height of the currently proposed retaining walls is approximately 30 feet. Given the sloping bedrock surface, and the anticipated high lateral loads acting on the walls, drilled shafts socketed into bedrock are considered well suited to support the proposed retaining walls.

Because of the forces generated by the retained fill, the drilled shafts needed to be analyzed under the influence of the lateral loading. The results of the GROUP analyses and the detailed drilled shaft recommendations are presented in Section 5.10 of this report.

5.8 Bridge Foundation Recommendations – Forward Abutment (R)

It is understood that the forward abutment of the right bridge will be placed at approximate station 140+25. While most of the slope in this area is under a thin soil cover (less than 5 feet), boring B-44 encountered slightly cohesive soils consisting of sandy silt (A-4a) and silt (A-4b) to a depth of 23.5 feet.

Based upon the available boring information and contour mapping it is recommended that the forward abutment for the right bridge structure be supported using drilled shafts, socketed into the sandstone bedrock. The use of spread footings was considered to support the abutment. However, the bedrock surface slopes downward, moving from left to right along the proposed abutment. The anticipated bedrock contact from the left to right along the abutment indicates a change in elevation of approximately 50 feet. Consequently, spread footings are not well suited for the site conditions found at the proposed forward abutment location.

The drilled shafts should be founded in competent sandstone bedrock. Due to the sloping and irregular bedrock surface, it is recommended that end-bearing drilled shafts be embedded 7.0 feet into the competent bedrock Assuming a minimum rock socket length (in competent rock) of 7.0 feet, the drilled shafts may be designed using an allowable bearing pressure of 80 ksf (40 tsf).

If adequate axial capacity cannot be developed with reasonable shaft diameter, drilled shafts could be designed as friction-type shafts. If designed as friction-type shafts, the side resistance in the upper two feet of the rock socket should be ignored. For portions of drilled shaft rock sockets above elevation 582.5, the drilled shaft capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 4500 psf. Similarly, below elevation 582.5, the drilled shaft capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 7500 psf. The side resistance in the overburden and upper two feet of the rock socket should be neglected.

For drilled shafts subject to uplift (tension), the drilled shafts should use an allowable uplift sidewall resistance relative to the shaft elevation. For portions of drilled shaft rock sockets above elevation 582.5, the drilled shaft uplift capacity may be calculated based upon an allowable sidewall shear stress/adhesion 3,000 psf. Similarly, below elevation 582.5, the drilled shaft uplift capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 5,000 psf. See attached calculations presented in Appendix IV.

5.9 Forward Abutment Retaining Wall (L)

It is understood that a retaining wall system will be required at the forward abutment location of the proposed left bridge. Based upon the provided drawings, it is understood that the maximum height of the proposed retaining wall system will be approximately 30 feet. Furthermore, it is understood that additional separation has been provided between the left and right bridges to allow modest amounts of deflection at the top of the proposed retaining wall. The intention is to mobilize the active earth pressure acting on the proposed retaining wall system and limit the size and number of foundation elements required to support the proposed wall. For the purposes of the analyses, it was assumed that the retaining wall was flexible enough to mobilize active earth pressures.

Based on information obtained from the borings and wall details provided by Transystems, it was acknowledged that the drilled shafts at the intersection of the forward abutment retaining wall and forward abutment backwall will penetrate approximately 5 feet of weathered sandstone before encountering competent bedrock. It should be noted that the strength characteristics of the sandstone bedrock vary with elevation across the site. Boring B-42 encountered broken, weaker sandstone from elevation 603.6 to 582.5. Below elevation 582.5, borings B-42 and B-43 generally encountered more competent sandstone. Please refer to the boring logs presented in Appendix II for more information.

Previously, analyses were performed to determine the required size and spacing of a single row of drilled shafts to support the proposed 50-foot tall retaining walls (from previous "type-study" configuration). These analyses indicated that a single row of drilled shafts (of reasonable size) could not resist the lateral earth pressures that are generated by the high fills. It is understood that the limiting condition in this case was structural capacity of the drilled shafts in shear resistance.

Analysis of this drilled shaft foundation system was performed by DLZ using GROUP Version 6.0 from Ensoft, Inc. GROUP is a program well suited for the analysis of multiple laterally loaded piles and/or drilled shafts and takes into account the interaction between axial loads and bending moments in the foundation elements. Loads from the actual bridge superstructure were provided by TranSystems; other loads, such as lateral earth pressures, were developed by DLZ based in part on information provided by TranSystems (i.e. wall dimensions).

For the purposes of these analyses, it was assumed that Pier 4, of the right structure would be supported on drilled shafts, socketed into bedrock. If spread footings were used to support Pier 4, the footing excavation could potentially destabilize the up-slope rock mass, which the drilled shafts supporting the retaining wall and left abutment are to be socketed into. Consequently, if spread footings are used to support Pier 4, DLZ should be notified so that the pertinent conclusions and recommendations may be modified as required.

Tables 4 and 5 outline the assumed parameters for the backfill material and the sandstone bedrock. For additional information, refer to the boring logs and the results of the laboratory testing, presented in Appendices II and III, respectively.

Table 4 –Assumed Fill Parameters, Forward Abutment (L)

Soil	Unit Weight (pcf)	Submerged Unit Weight (pcf)	Effective Friction Angle (deg)	Active Pressure Coefficient	At-rest Pressure Coefficient
Retained Fill	125	62.6	30	0.33	0.50

Table 5 –Assumed Bedrock Parameters, Forward Abutment (L)

Soil	Unit Weight (pcf)	Submerged Unit Weight (pcf)	Sliding μ	q_u (psi)	E_r (psi)
+Upper Bedrock	145	82.6	0.55	1600	3.99×10^5
++Lower Bedrock	145	82.6	0.55	10000	2.2×10^6

+ Upper Bedrock is considered above elevation 582.5.

++ Lower Bedrock is considered below elevation 582.5.

Results of Abutment Backwall Analyses – Drilled Shafts

Based upon the geotechnical resistance, the results of these analyses indicate that 48-inch diameter drilled shafts using 2.5D or 10-foot center-to-center spacing may be used to support the abutment backwall (perpendicular to the alignment). The analyses were performed for the design proposed by Transystems, which includes two rows of five drilled shafts. Analyses indicate that a maximum unfactored shear force of 131 kips is anticipated in each drilled shaft supporting the abutment retaining wall. Similarly, the maximum unfactored bending moment per shaft supporting the abutment retaining wall is anticipated to be 204 kip-ft. The unfactored axial load per shaft was found to range from approximately 811 to 102 kips, for the down-station and up-station rows, respectively. Based on the GROUP analyses, to resist the applied moment and shear forces, the minimum required embedment into rock should be 18 feet for the drilled shafts supporting the abutment backwall.

Results of Retaining Wall Analyses – Drilled Shafts

Analyses were also performed for the retaining wall (parallel to alignment) from station 139+00 to 139+65 for the left bridge. Because the larger (48-inch) drilled shafts supporting the abutment backwall will also support the corner of the retaining wall, it was assumed the retaining wall section begins at approximately station 139+15. Analyses indicated that 36-inch diameter drilled shafts using 3.33D or 10-foot center-to-center spacing may be used to support the highest portions of the retaining wall (parallel to the alignment). The analyses were performed for the proposed design, which includes two rows of five drilled shafts. Analyses indicate that a maximum unfactored shear force of 137 kips is anticipated in each drilled shaft supporting the retaining wall. Similarly, the maximum unfactored bending moment per shaft supporting the abutment retaining wall is anticipated to be 111 kip-ft. The unfactored axial load per shaft was found to range from approximately 584 to -4 kips (uplift), for the left and right rows, respectively. Based on the GROUP analyses, to resist the applied moment and shear forces, the minimum required embedment into rock should be 15 feet for the drilled shafts supporting the retaining wall.

Results Stability Analyses – Wall on Spread Footings

From the provided plans, it is understood that a portion of the retaining wall (parallel to the alignment) is planned to be supported using spread footings, founded in sandstone bedrock. This portion of the retaining wall, from approximate stations 139+65 to 140+28 has been analyzed for stability (overturning and sliding). The maximum height of this portion of wall (section A-A) was assumed to be 20.3 feet, which includes the footing thickness. The plans included in Appendix I present the details of the proposed wall section. The factor of safety against overturning was found to be 3.1, which is well above the minimum required factor of safety of 2.0. Similarly, the factor of safety against sliding was found to be 1.7, which is also above the minimum required factor of safety of 1.5. These analyses indicate that the stability of the proposed wall is adequate.

Spread footings supporting this retaining wall may be designed using an allowable bearing pressure of 20 ksf (10 tsf). It is recommended that a minimum footing width of 4 feet be used to bridge over any discontinuities that may be present in the bedrock surface.

Additional Drilled Shaft Recommendations – Abutment Retaining Wall System

For drilled shafts in compression and founded in competent rock, an allowable bearing pressure of 80 ksf may be used. However, in cases where the allowable bearing pressure is exceeded, designing the drilled shafts as friction-type drilled shafts may provide the axial capacity. The previously cited minimum required embedment depth is based upon geotechnical resistance to the applied lateral loads. Sufficient axial capacity should be attained using this required socket length. However, the required socket length may be increased if necessary to achieve adequate axial capacity.

If designed as friction-type shafts, the side resistance in the upper five feet of the rock socket should be ignored. For portions of drilled shaft rock sockets from elevation 603.6 to 582.5, the drilled shaft capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 4500 psf. Similarly, from elevation 582.5 and below, the drilled shaft capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 7500 psf.

As planned, some of the drilled shafts are expected to be subject to small amounts of permanent uplift. For drilled shafts subject to uplift (tension), the drilled shafts should use an allowable uplift sidewall resistance relative to the shaft elevation. The side resistance in the upper five feet of the rock socket should be ignored. For portions of drilled shaft rock sockets above elevation 582.5, the drilled shaft uplift capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 3,000 psf. Similarly, below elevation 582.5, the drilled shaft uplift capacity may be calculated based upon an allowable sidewall shear stress/adhesion of 5,000 psf.

It should be noted that while the proposed foundation/retaining wall system was analyzed for geotechnical stability, the structural design of the foundation elements will be performed by others. Due to the iterative nature of the design process, it was necessary for DLZ to make certain assumptions regarding the rigidity of the drilled shafts. The structural engineer is responsible for determining whether or not the various structural components (e.g. drilled shafts, footings, retaining walls, etc.) can be designed to resist the applied loads.

5.10 Drilled Shaft Foundations – General Recommendations

For drilled shafts founded in rock, FHWA guidelines allow the resistance provided by shear stress developed along the length of the drilled shaft, or side resistance, to be combined with the resistance provided by end bearing when determining the total axial capacity of the drilled shaft. It is reasonable to add portions of side resistance and end bearing when it can be demonstrated by load testing that the rock along the sides of the socket behaves in a ductile manner. Because site-specific dilatometer or load test data is not available, it is recommended that the designer assume that the rock will behave in a brittle manner. Consequently, the drilled shaft capacity should be based upon end bearing or side friction whichever is greater.

For drilled shafts subject to uplift, unit side resistance may be used to calculate the allowable uplift capacity of the drilled shaft. The allowable uplift capacity can be calculated by multiplying the unit side resistance by 0.7.

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

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

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Significant seepage was encountered by several of the borings. In addition, groundwater levels indicate that groundwater would flow into excavations into rock through granular layers overlying bedrock. 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 to prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

5.11 Temporary Retaining Wall - Pier 3

Because of the proximity of Pier 3 to existing SR 335, it is anticipated that a temporary retaining wall will be required to facilitate the construction of the foundations and pier columns in order to leave SR 335 open for traffic. Due to the anticipated function of the retaining wall, it will likely be temporary (service life of 3 years or less) in nature.

It is understood that the bottom of the footing / cap will be approximately elevation 504.3, and 500.0 for the left and right bridges, respectively. From the provided site plan and contour mapping, the elevation of SR 335 near Pier 3 is approximately 535. Consequently, a retained height of at least 31 feet is anticipated. Boring B-40, drilled in the area of Pier 3 encountered the top of sandstone bedrock at approximate elevation 480.5. As a result, approximately 55 feet of soil overburden is anticipated at the wall location. Soil parameters for the design of the retaining wall system are presented in Table 6. These parameters are provided based upon conditions encountered in borings B-40, B-41, and TR-32.

Table 6 – Sheet Pile Retaining Wall Parameters, Pier 3

Elevation (ft)	Moist Unit Weight (pcf)	Submerged Unit Weight (pcf)	Undrained Cohesion (psf)	Effective Friction Angle (deg)	+Active Pressure Coefficient	At-rest Pressure Coefficient	++Passive Pressure Coefficient
535 - 502	125	62.6	750	28	0.33	0.53	2.77
502 - 480	125	62.6	1500	28	0.33	0.53	2.77

+ Lateral Earth Pressure Coefficient calculated as per Coulomb including wall friction.
++ Lateral Earth Pressure Coefficients calculated as per Rankine.

Because of the anticipated height of the retaining structure, it may be necessary to "tie-back" the wall using rock anchors to control the deflections and provide stability to the wall system. Based upon published guidelines from the U.S. Army Corps of Engineers (EM 1110-1-2907) and the Post-Tensioning Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors", the ultimate bond stress in competent sandstone generally ranges from 120 to 250 pounds per square inch (psi). Preliminary design should use a tentative ultimate bond stress of 185 psi. This value is provided as an estimate for preliminary design only. As per PTI, the working bond stress used to determine the bond length is normally 25 to 50 percent of the ultimate bond stress. Also, for normal applications, the bond length should not be less than 10 feet and the unbonded (free) length should not be less than 15 feet. The Contractor must verify the final design assumptions by conducting performance tests prior to installation of the production anchors. It is recommended that special provisions be produced for the design, field-testing, and installation of the tieback anchors which will likely be required for this retaining wall. Once a design has been finalized, special provisions should be produced for the proposed retaining wall and tieback anchors. DLZ should also have the opportunity to review the special provisions.

It should be noted that the top of bedrock slopes upward to the northwest of the Pier 3 location. Although the approximate top of bedrock can be estimated from the available borings at the Pier 3 location (B-40 and B-41) and at the forward abutment/Pier 4 location (B-42 and B-43), it is recommended that additional borings be drilled to determine the bedrock contact location/elevation along the length of the anchors.

5.12 Embankment Stability Analysis

In the area of the proposed rear abutment, the terrain slopes into the riverbed of the Little Scioto River. The in-situ soils consist of primarily fine-grained soils consisting of silt (A-4b) and clay (A-7-6). Near the river, borings indicate that the soil strata are discontinuous with varying consistency, shear strengths, and moisture contents.

It should be noted that previous design configurations consisted of a 44-foot embankment with 2:1 slopes. The proposed rear abutment would be placed at approximately station 132+20. The proposed toe of the roadway embankment would be at approximately station 132+80. This will place the toe at the crest of the existing bank, which stability is a concern. Analyses indicate that the previous configurations have a factor of safety below the recommended minimum value for undrained and drained global stability. Consequently, several embankment configurations were evaluated to ensure that a stable embankment would be provided at the rear abutment. Based upon geotechnical analyses, representatives of ODOT and Transystems have indicated that the preferred option is to reposition the rear abutment down-station to approximate station 130+73 in order to provide a stable approach embankment. The spill through and side slopes should be constructed using 2.5H:1V slopes. In addition, the embankments should be constructed using staged construction as outlined in the Report of Subsurface Exploration for Proposed Highland Bend Roadway Embankments, dated August 2, 2007. Additional details regarding these preliminary global stability analyses can be found in a letter titled "Addendum to Proposed Highland Bend Embankment Report, Preliminary Retaining Wall Options and Spill Through Slopes, Little Scioto River Structure", dated June 29, 2006.

The currently proposed configuration places the rear abutment at station 130+75. See the Structure Site Plan included in Appendix I. The currently proposed embankment configuration is acceptable provided that the approach embankments are constructed as per the recommendations provided in the Report of Subsurface Exploration for Proposed Highland Bend Roadway Embankments, dated August 2, 2007. The spill through and side slopes should be constructed using 2.5H:1V slopes. Staged construction of the embankments is required for the embankments contained in the Highland Bend Valley. The associated details are presented in the above-referenced report.

The in-situ foundation soils will consolidate under the influence of the new embankment fill. Settlement calculations indicate that approximately 37 inches of settlement will occur at the rear abutment location. Time-rate of settlement calculations indicate that the in-situ foundation soils would consolidate to ninety percent (U=90%) in approximately 6 years without wick drains or other means to accelerate the consolidation process. However, due to the need for staged construction of the Highland Bend mainline embankments, it is anticipated that wick drains will be used under the embankment at the proposed rear abutment. Time-rate of consolidation estimates using wick drains are provided in Table 3.

It is recommended that pore pressures and settlements be monitored via vibrating wire piezometers and settlement platforms during construction operations. An instrumentation plan and associated details are presented in the Highland Bend Report. Please refer to this document for additional information.

5.13 Scour Analyses

Particle-size analyses were performed on several of the samples collected within the floodplain for possible scour analysis. Piers 2 and 3 of the proposed structures are located within the Little Scioto River flood plain. Table 7, presents the sample locations and the D_{50} and D_{85} sizes from select particle size analyses. For additional results of the pertinent particle-size analyses, refer to the grain-size reports, presented in Appendix III.

Table 7 – Particle-Size Distribution Results for Scour Analyses, Pier 3

Boring Number	Elevation (ft.)	Sample Depth (ft.)	ODOT Classification	D_{85} (mm)	D_{50} (mm)
B-39	508.0	3.5	A-6b	0.0320	0.0095
		8.5	A-6a	0.0475	0.0092
		10.0	A-4b	0.0579	0.0118
		13.5	A-4a	0.2230	0.0427
		21.0	A-4b	0.1090	0.0243
		26.0	A-4b	0.2040	0.0461
B-40	529.0	3.5	A-6b	0.0318	0.0070
		8.5	A-6a	0.0373	0.0111
		13.5	A-4b	0.0800	0.0224
		18.5	A-4b	0.0538	0.0178
		20.0	A-4b	0.0447	0.0172
		26.0	A-4b	0.5190	0.0277
		38.5	A-4a	0.9670	0.0422

5.14 Groundwater Considerations

At the rear abutment, boring TR-35A first encountered seepage at a depth of 24.5 feet below the ground surface. Under the current design, little excavation, if any, is anticipated at the rear abutment location. Consequently, seepage and groundwater are not likely to impact any excavations at the rear abutment.

In the area of Pier 1, boring TR-35 first encountered seepage at a depth of 7.0 feet below the ground surface, or elevation 545.8. The bottom of the pile cap is understood to be approximate elevation 546.6. The contractor may encounter minor seepage while excavating for the pile cap at the Pier 1 location. Deeper excavations would encounter more significant seepage.

In the area of Pier 2, boring TR-33 first encountered seepage at a depth of 5.5 feet below the ground surface, or elevation 497.2. The bottom of the pile cap is understood to be approximate elevation 500.5. The contractor may encounter minor seepage while excavating for the pile cap at the Pier 2 location. However, deeper excavations will encounter significant amounts of seepage. The seepage and groundwater conditions near the river will likely be influenced by the river level and recent weather conditions. Due to the soil and groundwater conditions, excavations for the proposed drilled shaft foundations will require the use of temporary casing and/or dewatering to keep the excavation dry and to combat the potential for a "quick" condition to develop.

In the area of Pier 3, boring TR-32 first encountered seepage at a depth of 4.0 feet below the ground surface, or elevation 511.1. The bottom of the pile cap is understood to be approximate elevation 504.3. The contractor should be prepared to deal with seepage and groundwater while excavating for the pile cap at the Pier 3 location. Deeper excavations, for the installation of the drilled shafts will also encounter seepage and groundwater. The seepage and groundwater conditions near the river will likely be influenced by the river level and recent weather conditions. Due to the soil and groundwater conditions, excavations for the proposed drilled shaft foundations will require the use of temporary casing and/or dewatering to keep the excavation dry and to combat the potential for a "quick" condition to develop.

Excavations for the Pier 4 (R) and forward abutment excavations are anticipated to encounter little if any seepage above the bedrock. Excavations for drilled shafts below the bedrock may encounter seepage. Due to the use of water during rock coring operations, seepage zones in the rock could not be observed.

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. Consequently, the contractor should be prepared to deal with any unexpected seepage, water flow, and precipitation that may enter any excavations.

It is anticipated that the drilled shaft excavations will encounter significant amounts of seepage, especially within the vicinity of the Little Scioto River. The contractor should be prepared to deal with water in the drilled shaft excavations. Temporary casings may be considered to seal off the saturated overburden from the rock mass. The preferred method of construction is to place concrete in a dry hole; however, if de-watering is determined to be impractical, approved methods for concrete placement in water (CMS Item 524) can be used

5.15 General Earthwork Recommendations

The proposed alignment of SR 823 over the Little Scioto River and SR 335 traverses a relatively narrow river and flood plain. The rear abutment of the structure will be constructed on embankment fill. The maximum embankment height at the rear abutment is approximately 46 feet.

Based upon boring TR-35, approximately 4 inches of topsoil is anticipated. All topsoil and vegetation within the footprint of the new embankment and roadway should be removed prior to new fill placement. All pavement, and organic soil within three feet of subgrade level should also be removed prior to placing fill or pavement materials.

Organic soils were not encountered in the borings drilled near the rear abutment approach embankment. However, organic or very soft soils may be encountered at locations other than where the borings were drilled. Consequently, 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.

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

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

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

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

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

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

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

Timothy A. Hampshire, P.E.
Structural Engineer

Eric Tse, P.E.
Senior Geotechnical Engineer

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PROFESSIONAL ENGINEER

sjr

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SECTION 1

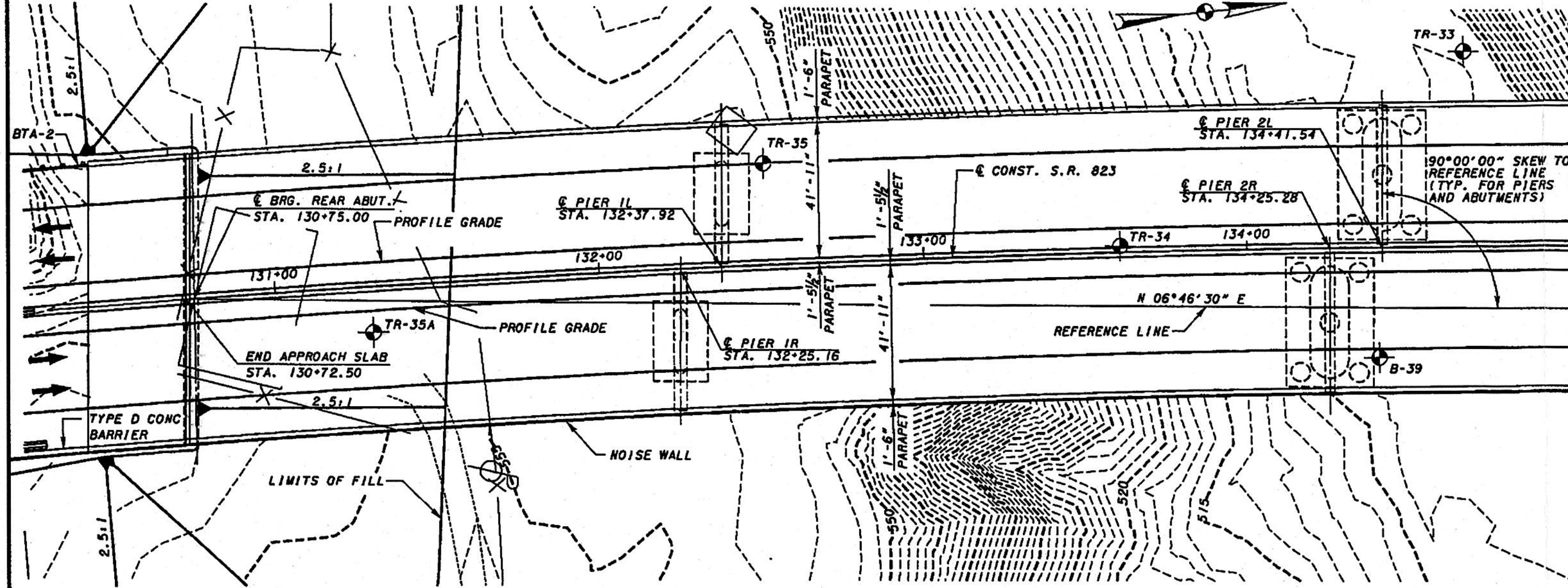
DLZ

APPENDIX I

Structure Plan and Profile Drawing – 11"x17"

Structure Type Study Review Comments, dated August 11, 2006

Structure Type Study Review Comments, dated December 13, 2007



PLAN

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS	
LOCATION	STATION
REAR ABUT. (SB)	130+41.42
FWD. ABUT. (SB)	139+32.36

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

TRAFFIC DATA

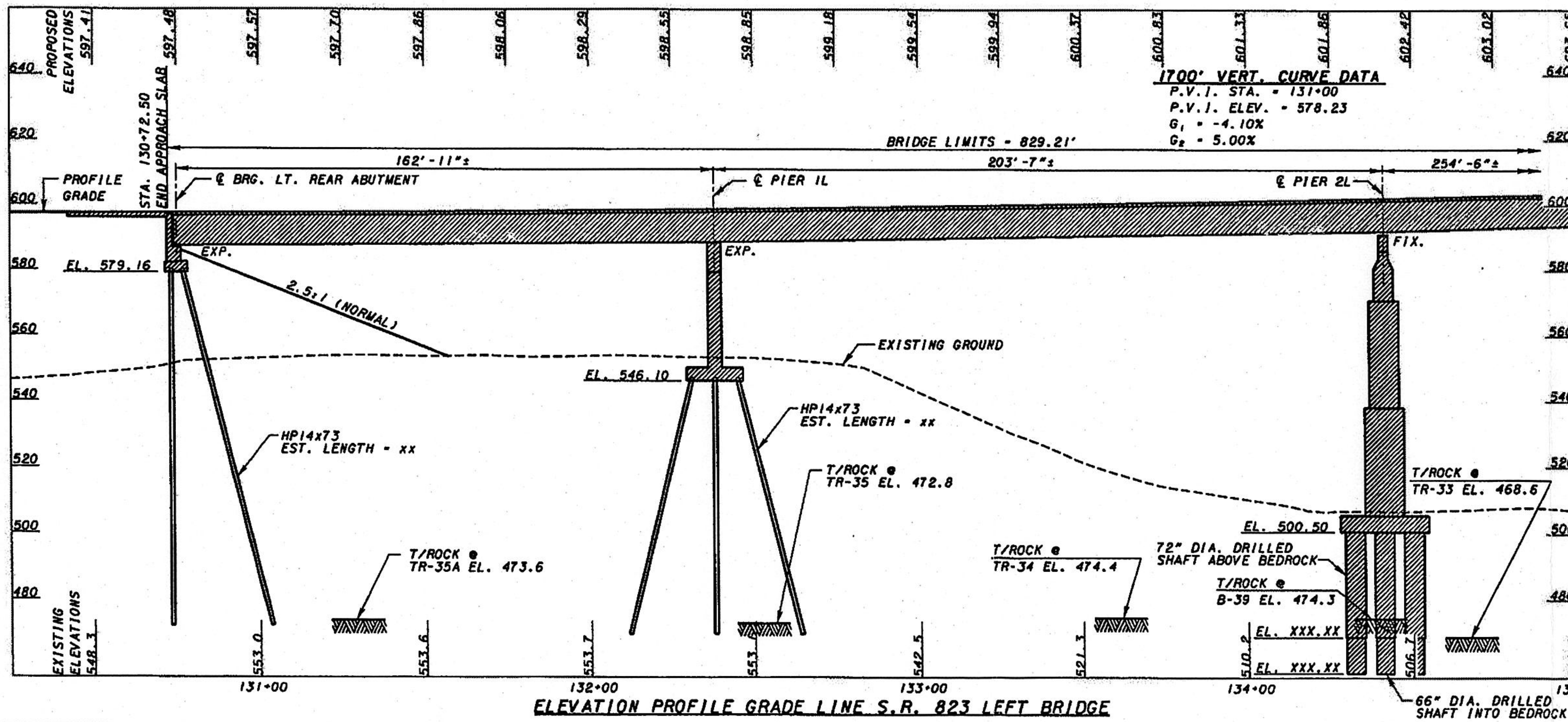
S.R. 823
CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,970
DESIGN YEAR ADTT (2030) = 4,370

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.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ = BORING LOCATION



ELEVATION PROFILE GRADE LINE S.R. 823 LEFT BRIDGE

PROPOSED STRUCTURE

TYPE: 4 SPAN CONTINUOUS STEEL PLATE GIRDER A709 GRADE 50W, DOG LEGGED AT SPLICES, WITH COMPOSITE REINFORCED CONCRETE DECK ON STUB ABUTMENTS AND T-TYPE PIERS

SPANS: 162'-11", 203'-4", 254'-6"±, 203'-2" (MEASURED ALONG ℄ CONST. S.R. 823)

ROADWAY: 2 - 41'-11" TOE TO TOE PARAPETS

LOADING: HS-25 (CASE 1) AND ALTERNATE MILITARY LOADING, FWS-60 PSF

SKEW: 0°00'00" WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)

SUPERELEVATION: 0.036 FT/FT ACROSS TRAVEL LANES

ALIGNMENT: Dc = 1°00'00" (TO THE RIGHT)

WEARING SURFACE: MONOLITHIC CONCRETE

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

LATITUDE: 38°46'28" N

LONGITUDE: 82°51'52" W

DESIGN AGENCY: Systems

DESIGNED MTN | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED MSL | DATE 01/11/08 | STRUCTURE FILE NUMBER 7306369

SCIOTO COUNTY STA. 130+72.50 STA. 139+01.71

SITE PLAN
BRIDGE NO. SCO-823-0248 L
S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

SCI-823-0.00
PID 77366

1/15

835/847

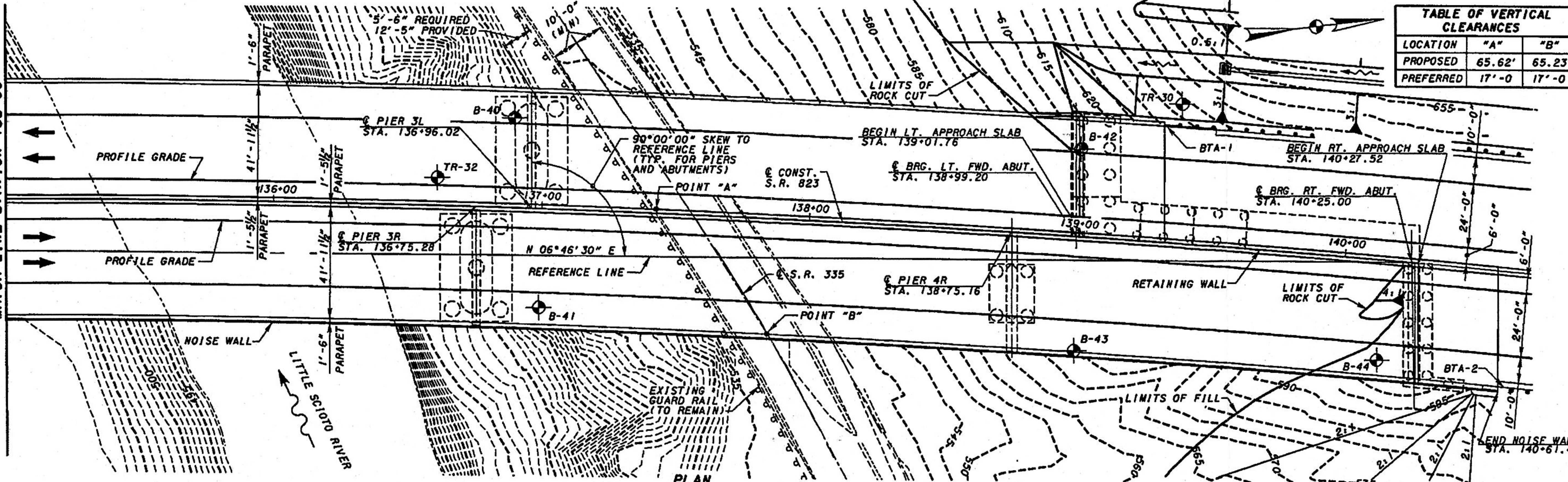
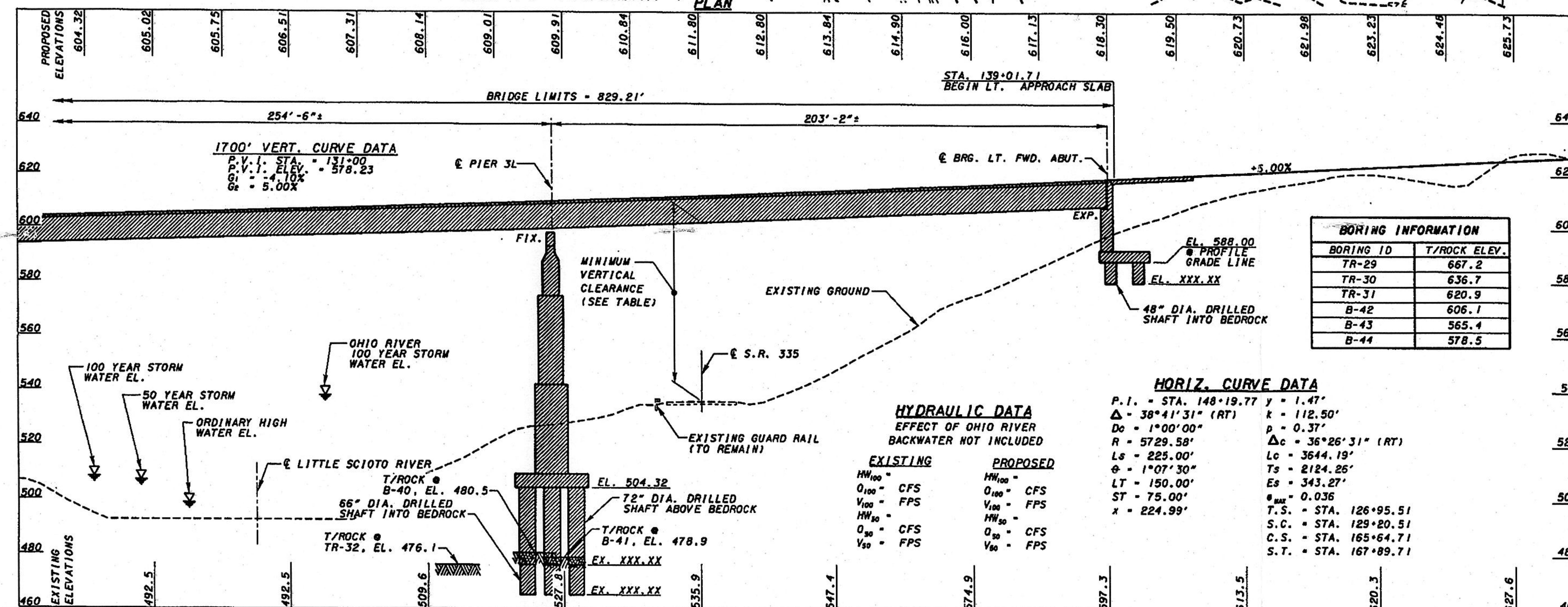


TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	65.62'	65.23'
PREFERRED	17'-0	17'-0



BORING INFORMATION

BORING ID	T/ROCK ELEV.
TR-29	667.2
TR-30	636.7
TR-31	620.9
B-42	606.1
B-43	565.4
B-44	578.5

HYDRAULIC DATA

EFFECT OF OHIO RIVER BACKWATER NOT INCLUDED

EXISTING	PROPOSED
HW_{100} =	HW_{100} =
Q_{100} = CFS	Q_{100} = CFS
V_{100} = FPS	V_{100} = FPS
HW_{50} =	HW_{50} =
Q_{50} = CFS	Q_{50} = CFS
V_{50} = FPS	V_{50} = FPS

HORIZ. CURVE DATA

P.I. = STA. 148+19.77	y = 1.47'
Δ = 38°41'31" (RT)	k = 112.50'
Dc = 1°00'00"	p = 0.37'
R = 5729.58'	Δc = 36°26'31" (RT)
Ls = 225.00'	Lc = 3644.19'
θ = 1°07'30"	Ts = 2124.26'
LT = 150.00'	Es = 343.27'
ST = 75.00'	e_{MAX} = 0.036
x = 224.99'	T.S. = STA. 126+95.51
	S.C. = STA. 129+20.51
	C.S. = STA. 165+64.71
	S.T. = STA. 167+89.71

ELEVATION ALONG PROFILE GRADE LINE S.R. 823 LEFT BRIDGE

SITE PLAN

BRIDGE NO. SCI-823-0248 L

S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

SCIOTO COUNTY

STA. 130+72.50

STA. 139+01.71

SCI-823-0.00

PID 77366

DESIGNED MTN | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED MSL | DATE 01/11/08 | STRUCTURE FILE NUMBER 7306369

DESIGN AGENCY Systems

2/15

836/847

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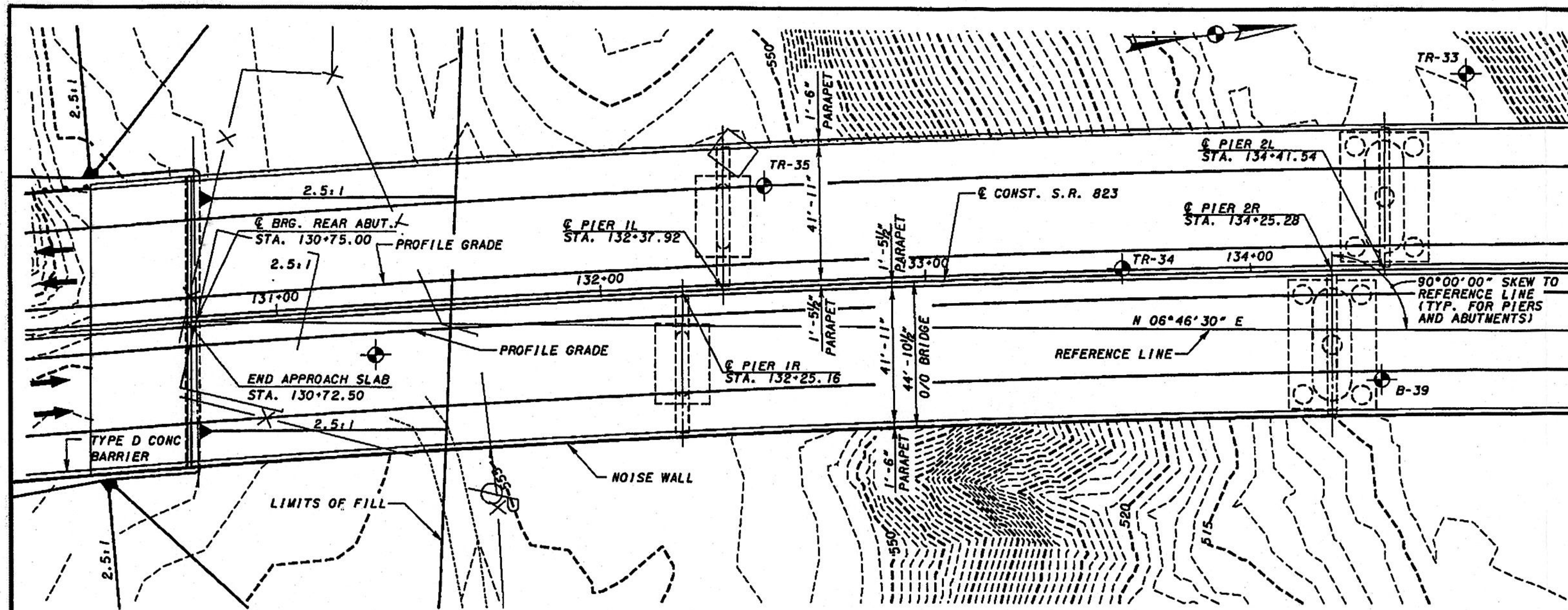
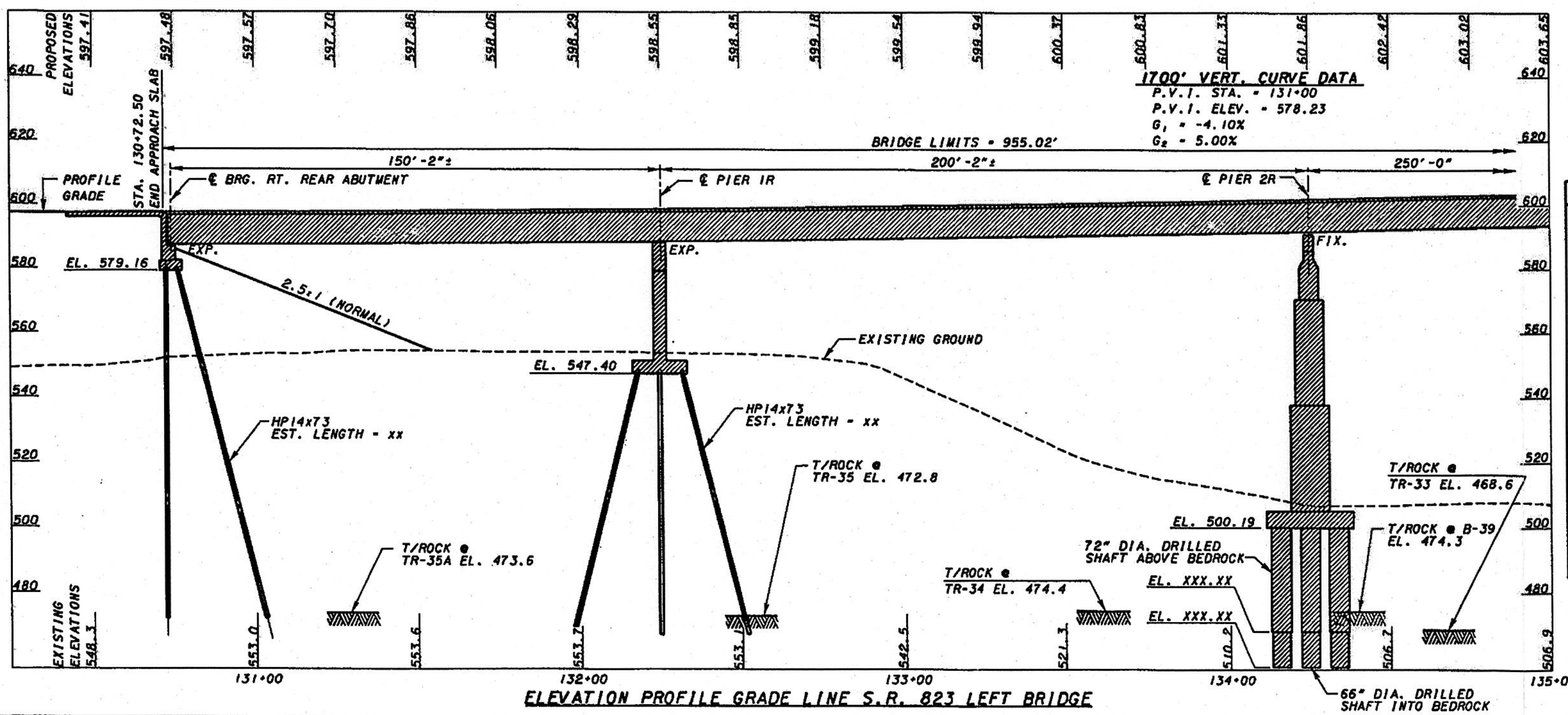


PLAN



ELEVATION PROFILE GRADE LINE S.R. 823 LEFT BRIDGE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS	
LOCATION	STATION
REAR ABUT. (NB)	109+95.85
FWD. ABUT. (NB)	140+66.33

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

TRAFFIC DATA

S.R. 823
CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,970
DESIGN YEAR ADTT (2030) = 4,370

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.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ = BORING LOCATION

PROPOSED STRUCTURE

TYPE: 5 SPAN CONTINUOUS STEEL PLATE GIRDER A709 GRADE 50W, DOG LEGGED AT SPLICES, WITH COMPOSITE REINFORCED CONCRETE DECK ON STUB ABUTMENTS AND T-TYPE PIERS
SPANS: 150'-2"±, 200'-2"±, 250'-0", 199'-10"±, 149'-10"± C/C BEARINGS (MEASURED ALONG € CONST. S.R. 823)
ROADWAY: 2 - 41'-11" TOE TO TOE PARAPETS
LOADING: HS-25 (CASE 1) AND ALTERNATE MILITARY LOADING, FWS=60 PSF
SKEW: 0°00'00" WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)
SUPERELEVATION: 0.036 FT/FT ACROSS TRAVEL LANES
ALIGNMENT: Dc = 1°00'00" (TO THE RIGHT)
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)
LATITUDE: 38°46'28" N
LONGITUDE: 82°51'52" W

DESIGN AGENCY: Systems
DESIGNED MTN | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED MSL | DATE 01/11/08 | STRUCTURE FILE NUMBER 7306377

SCIOTO COUNTY
STA. 130+72.50
STA. 140+27.52

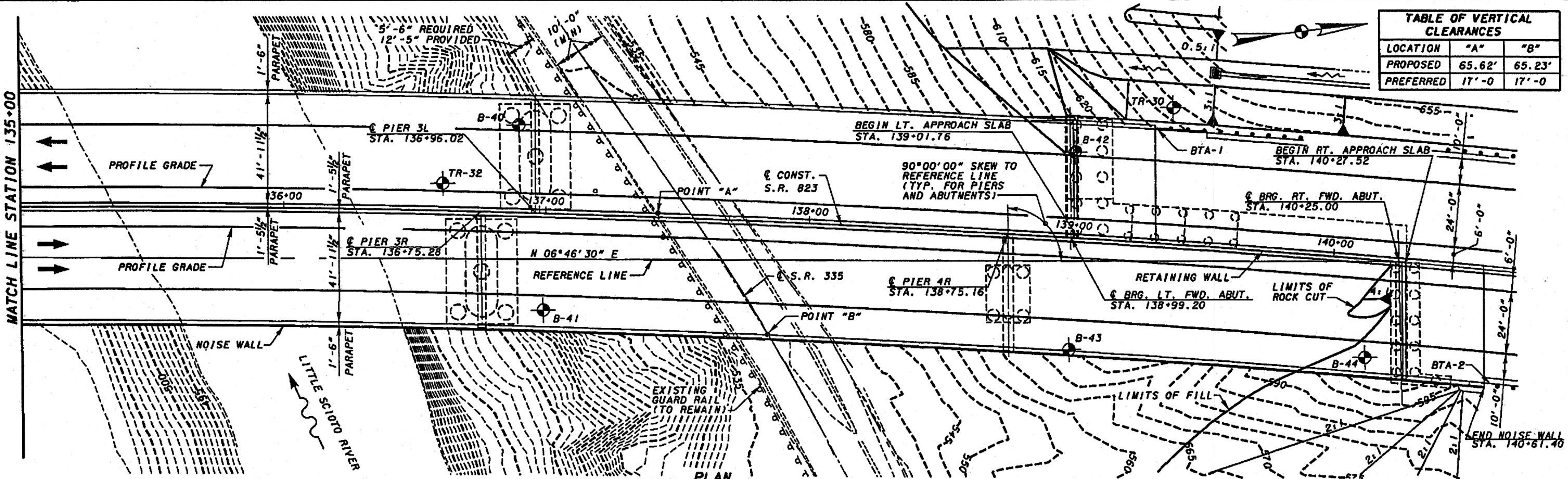
SITE PLAN
BRIDGE NO. SCO-823-0248 R
S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

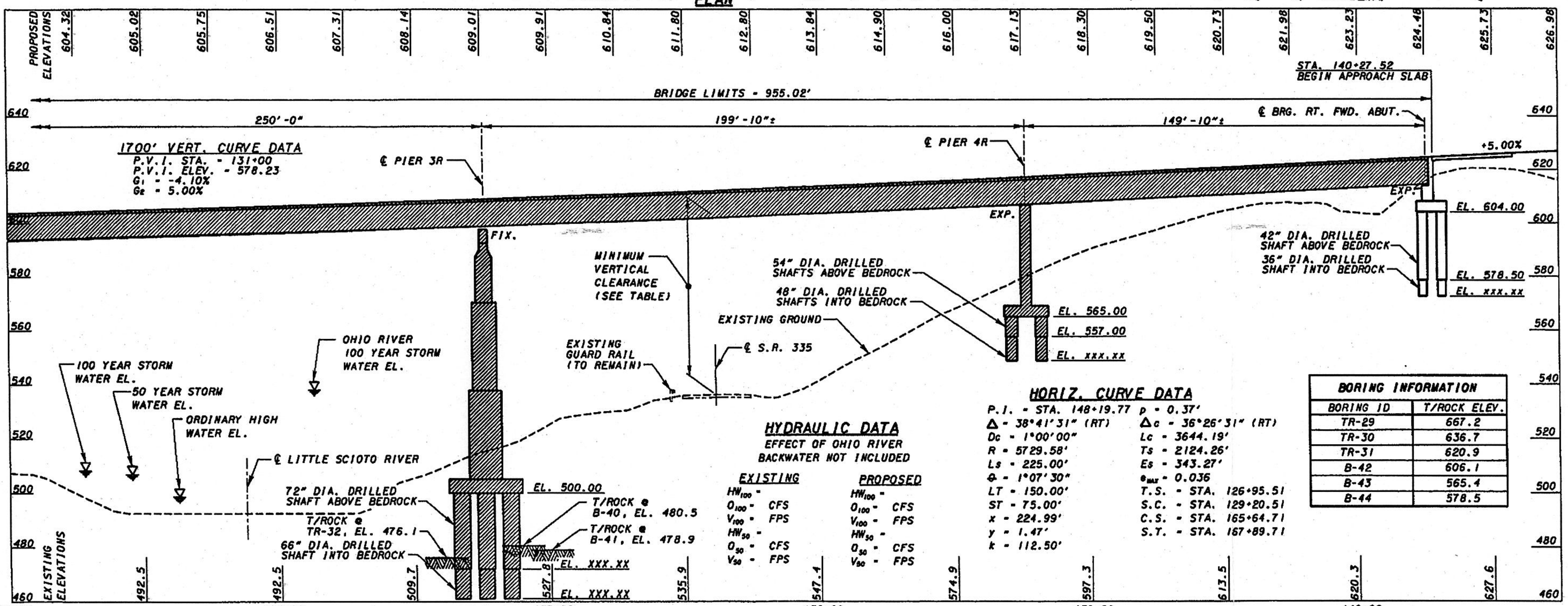
SCI-823-0.00
PID 77366

3/15

837/847

8:07:50 AM 1/6/2008 g:\oo03\006\bridge\on\bts\07-sr335\littlescioto river\tet\823_0248osp003.dgn





ELEVATION ALONG PROFILE GRADE LINE S.R. 823 LEFT BRIDGE

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	65.62'	65.23'
PREFERRED	17'-0	17'-0

HYDRAULIC DATA

EFFECT OF OHIO RIVER BACKWATER NOT INCLUDED

EXISTING	PROPOSED
HW_{100} -	HW_{100} -
Q_{100} - CFS	Q_{100} - CFS
V_{100} - FPS	V_{100} - FPS
HW_{50} -	HW_{50} -
Q_{50} - CFS	Q_{50} - CFS
V_{50} - FPS	V_{50} - FPS

HORIZ. CURVE DATA

P.I. - STA. 148+19.77	p - 0.37'
Δ - 38°41'31" (RT)	Δ_c - 36°26'31" (RT)
Dc - 1°00'00"	Lc - 3644.19'
R - 5729.58'	Ts - 2124.26'
Ls - 225.00'	Es - 343.27'
θ - 1°07'30"	e_{max} - 0.036
LT - 150.00'	T.S. - STA. 126+95.51
ST - 75.00'	S.C. - STA. 129+20.51
x - 224.99'	C.S. - STA. 165+64.71
y - 1.47'	S.T. - STA. 167+89.71
k - 112.50'	

BORING INFORMATION

BORING ID	T/ROCK ELEV.
TR-29	667.2
TR-30	636.7
TR-31	620.9
B-42	606.1
B-43	565.4
B-44	578.5

DESIGN AGENCY: Systems

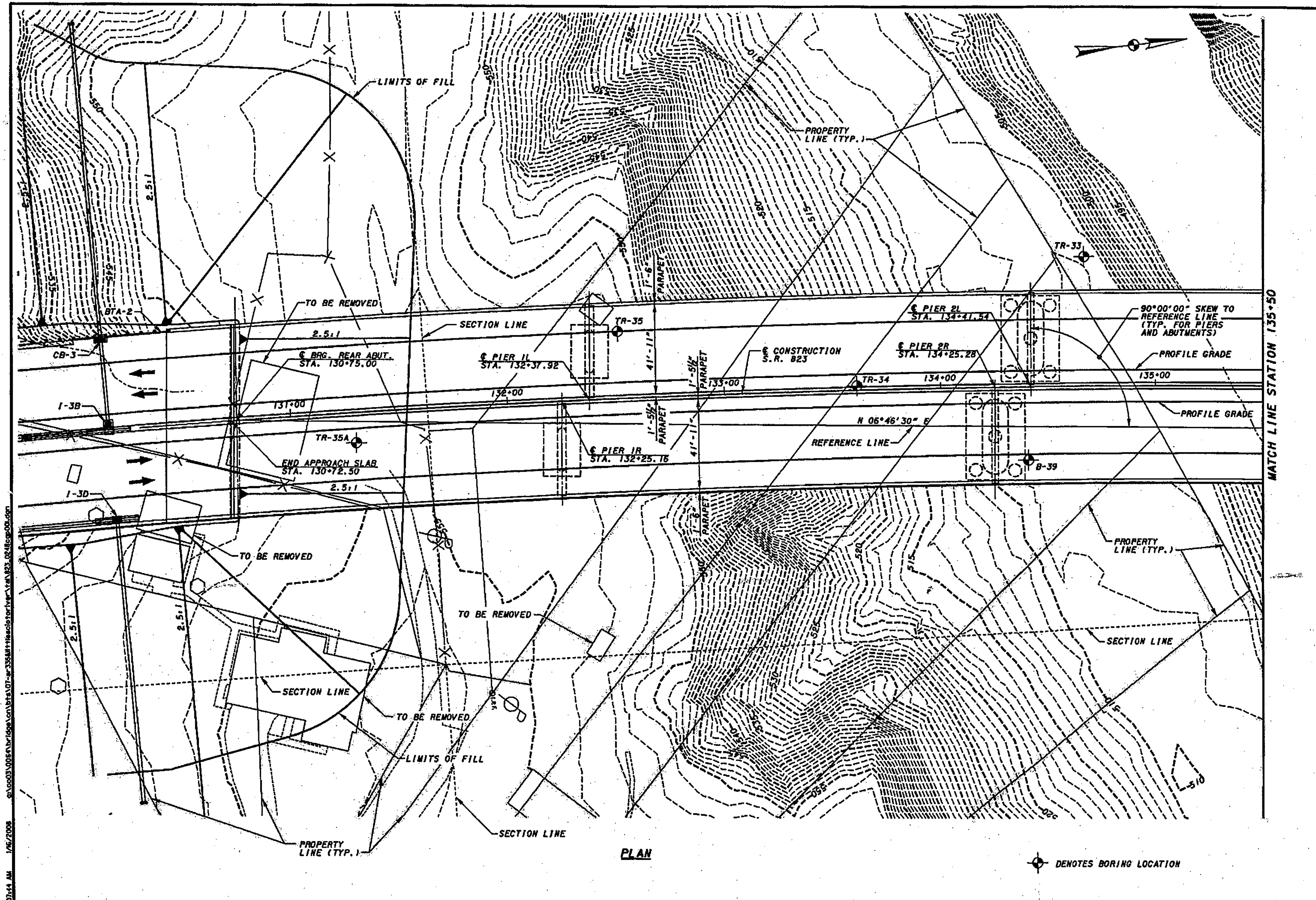
DESIGNED	DRAWN	REVIEWED	DATE
MTN	MTN	MSL	01/11/08
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP		7306369	

SCIOTO COUNTY
STA. 130+72.50
STA. 140+27.52

SITE PLAN

BRIDGE NO. SCI-823-0248 R
S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

SCI-823-0.00
PID 77366



GENERAL PLAN

BRIDGE NO. SCO-823-0248 L&R

S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

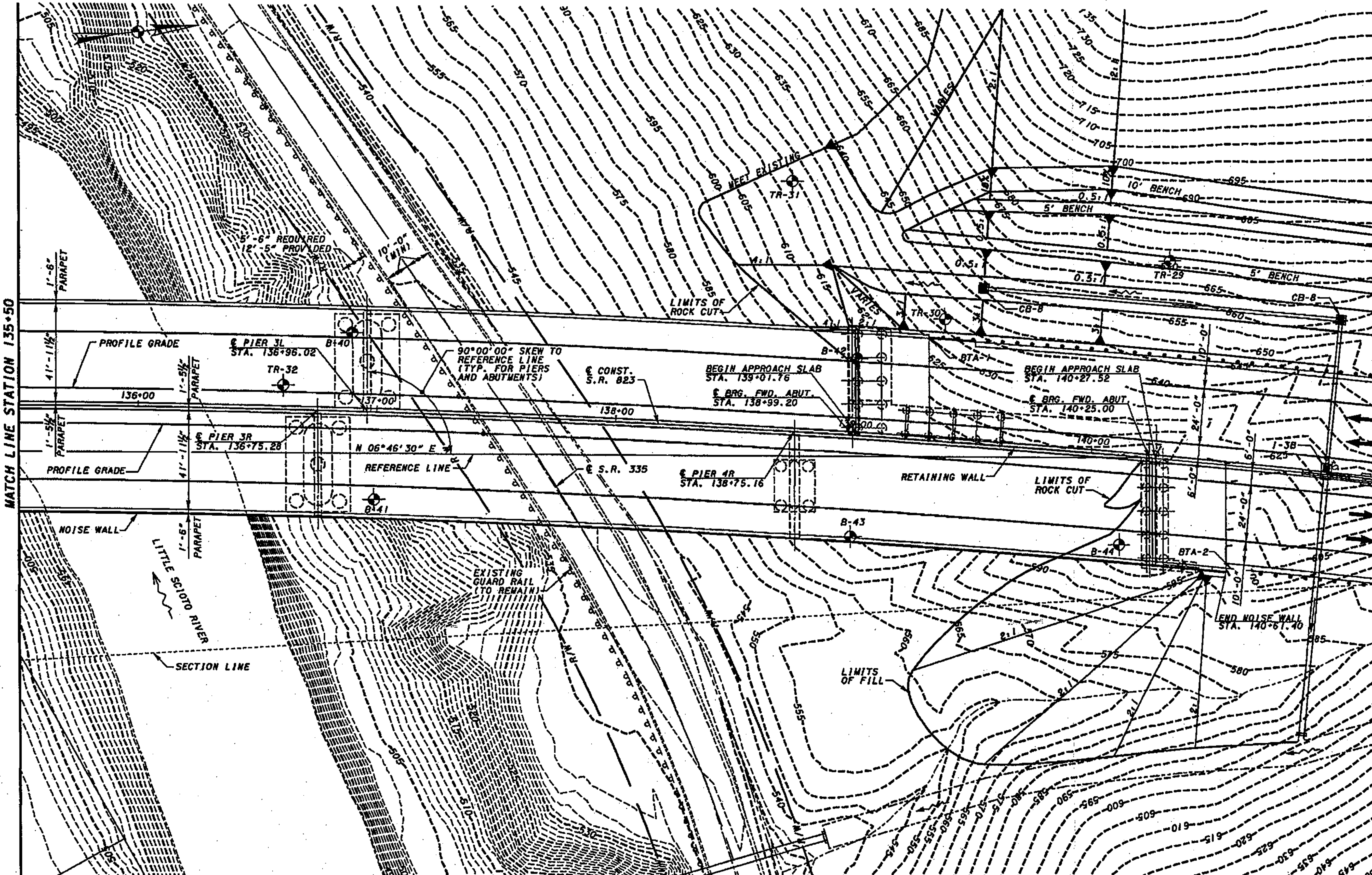
SCI-823-0.00

PID 77366

5/15

839/847

DESIGNED MTH | DRAWN MTH | REVIEWED MSL | DATE 1/11/08 | CHECKED PJP | REVISED | STRUCTURE FILE NUMBER 7306369L, 7306377R



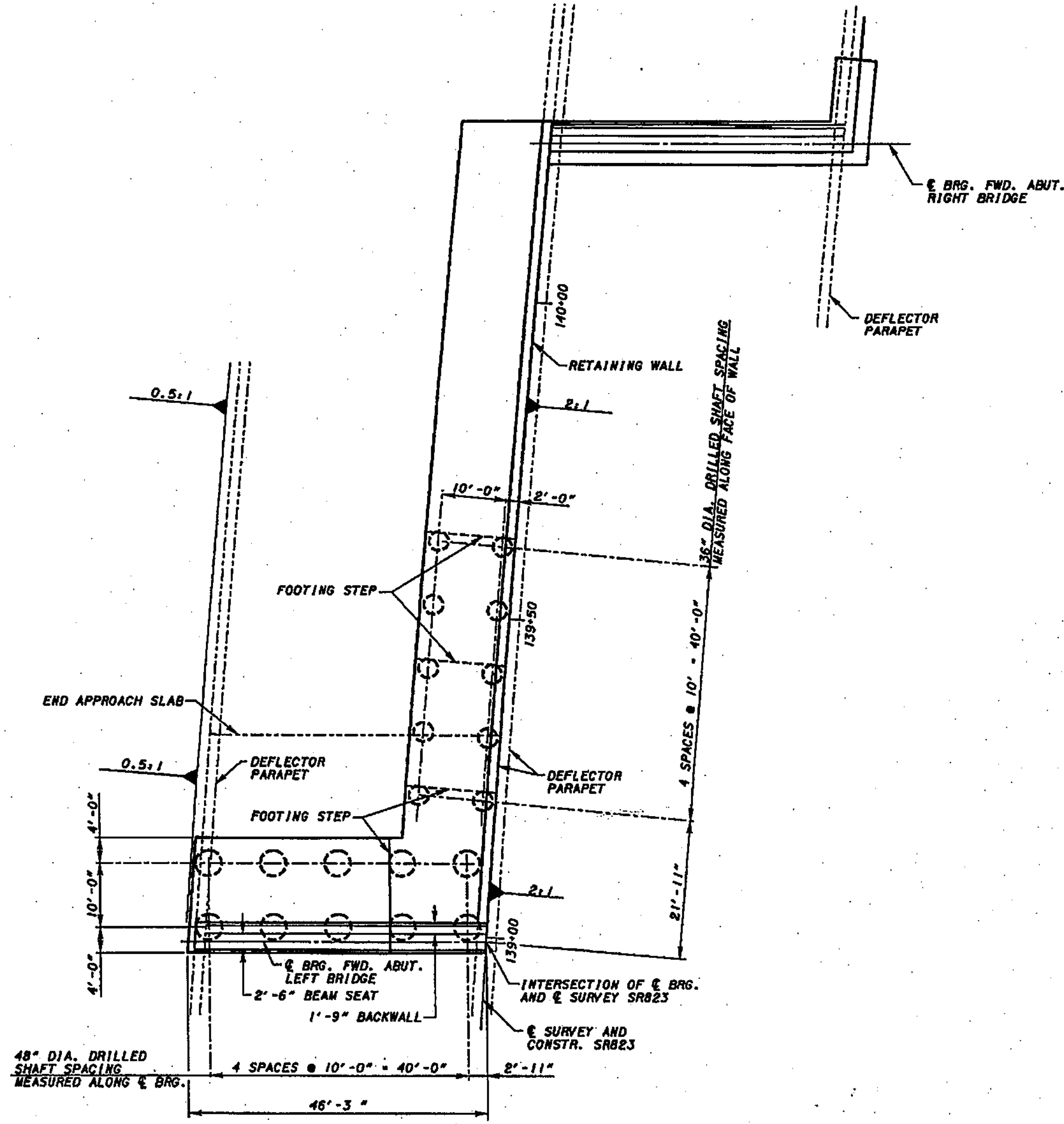
PLAN

DENOTES BORING LOCATION

DESIGNED	DRAWN	REVIEWED	DATE
MTH	MTH	MSL	11/26/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP	MSW	7306368L, 7306377R	

GENERAL PLAN
BRIDGE NO. SCO-823-0248 L&R
S.R. 823 OVER S.R. 335 & LITTLE SCIOTO RIVER

SCI-823-0.00
PID 77366

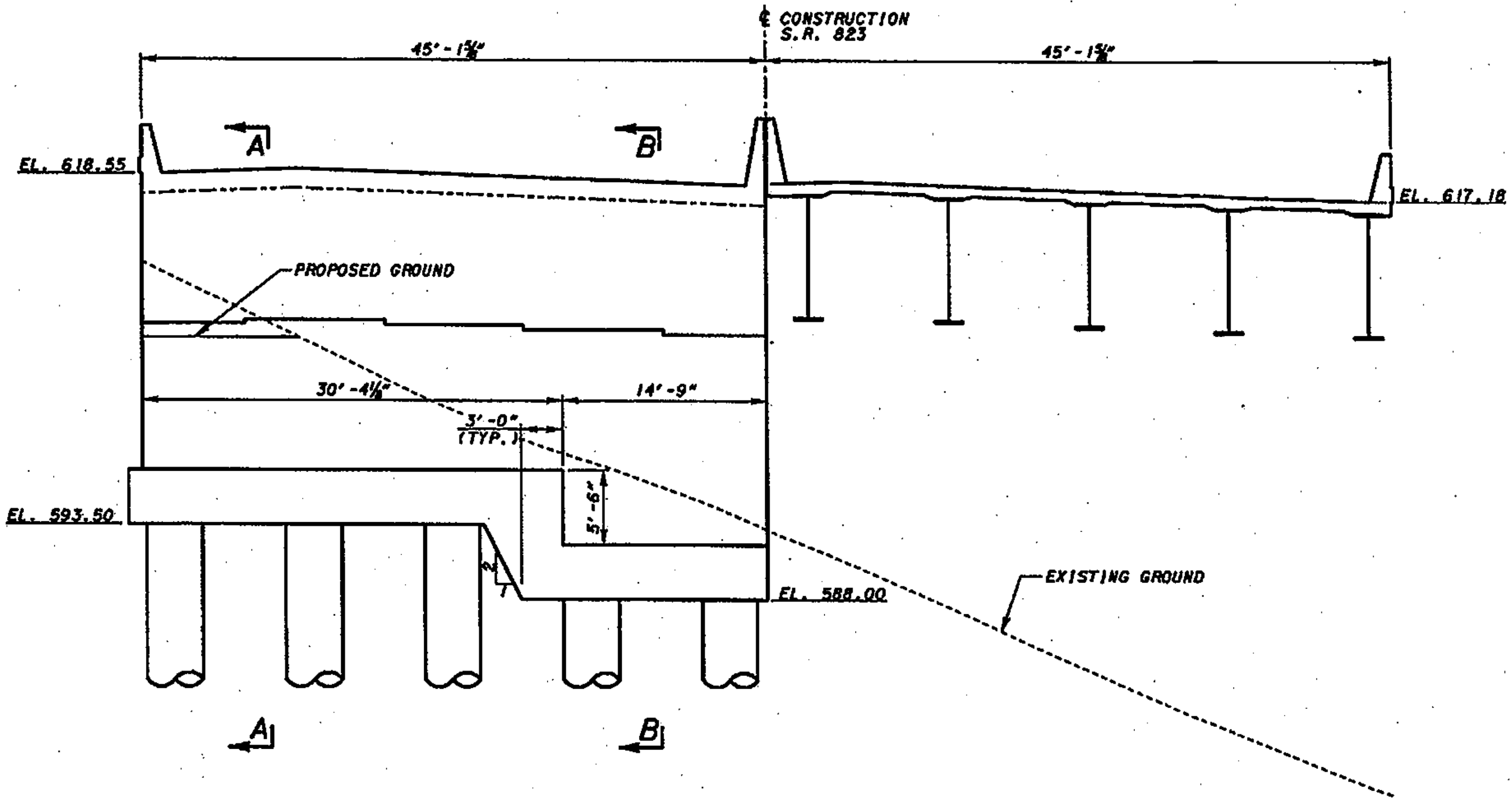


PLAN

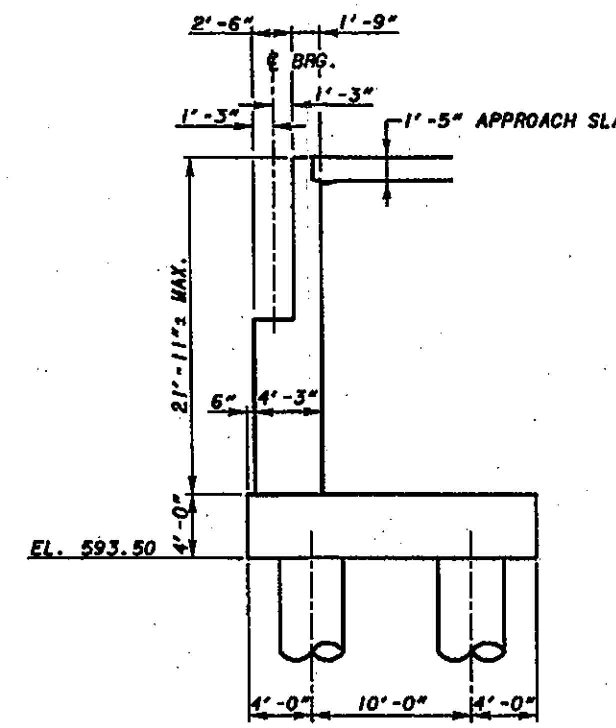
FORWARD ABUTMENT DETAILS - ALTERNATIVE 5
BRIDGE NO. SCI-823-0248
SR823 OVER SR335 & LITTLE SCIOTO RIVER

DESIGNED	DRAWN	REVIEWED	DATE
MTH	JDG	MSL	11/26/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP			

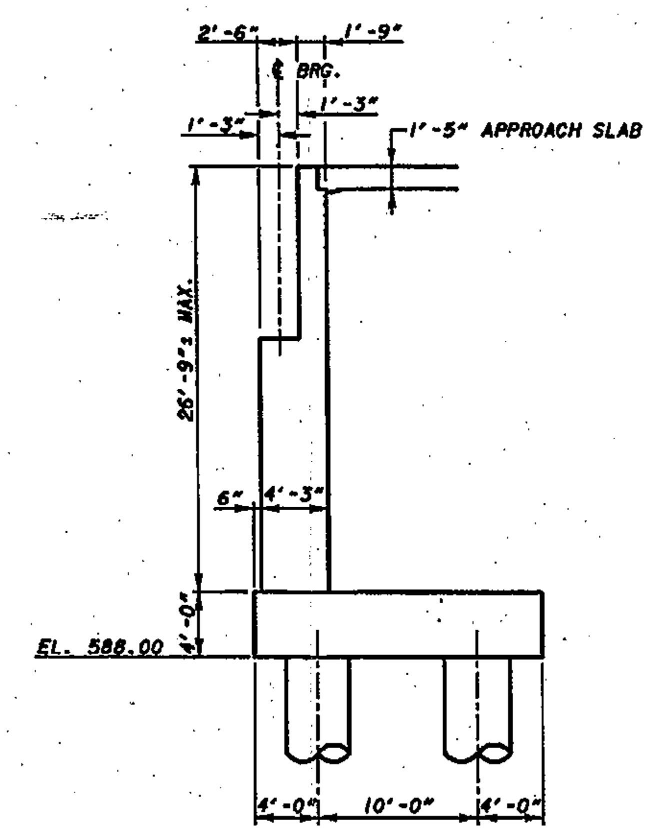
SCI-823-0.00
PID 77366



ELEVATION
(ALONG FACE OF ABUTMENT)



SECTION A-A

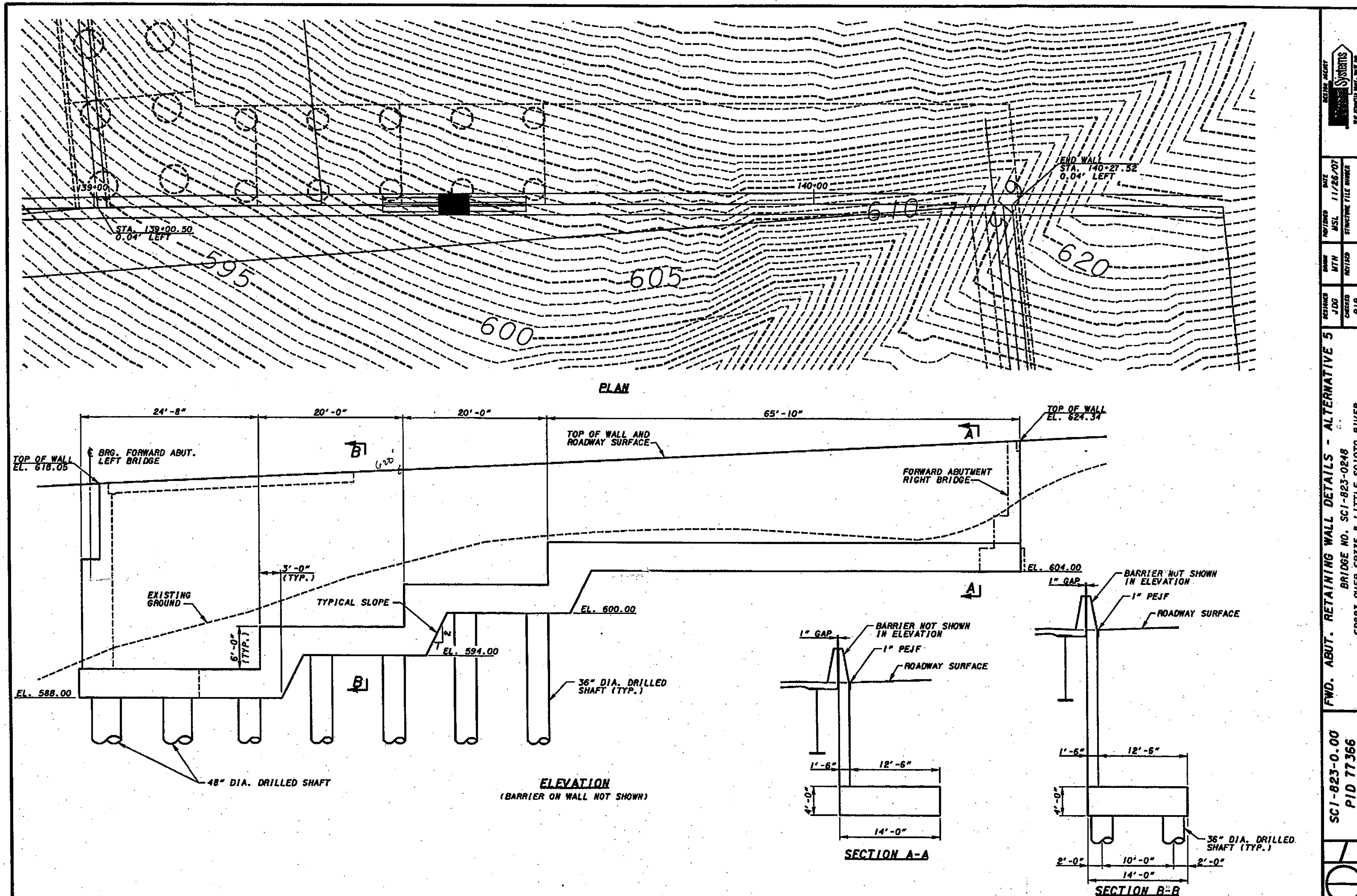


SECTION B-B

DESIGNED MHT | DRAWN JOG | REVIEWED MSL | DATE 11/26/07 | CHECKED PJP

FORWARD ABUTMENT DETAILS - ALTERNATIVE 5
BRIDGE NO. SCI-823-0248
SR823 OVER SR335 & LITTLE SCIOTO RIVER

SCI-823-0.00
PID 77366


139+00
140+00
STA. 139+00.50
0.04' LEFT
END WALL
STA. 140+27.52
0.04' LEFT
595
600
605
610
620
PLAN
24'-8"
20'-0"
20'-0"
65'-10"
TOP OF WALL
EL. 618.05
℄ BRG. FORWARD ABUT.
LEFT BRIDGE
TOP OF WALL AND
ROADWAY SURFACE
TOP OF WALL
EL. 624.34
FORWARD ABUTMENT
RIGHT BRIDGE
EL. 604.00
3'-0"
(TYP.)
EXISTING
GROUND
TYPICAL SLOPE
EL. 600.00
6'-0"
(TYP.)
EL. 594.00
EL. 588.00
36" DIA. DRILLED
SHAFT (TYP.)
48" DIA. DRILLED SHAFT
ELEVATION
(BARRIER ON WALL NOT SHOWN)
1" GAP
BARRIER NOT SHOWN
IN ELEVATION
1" PEJF
ROADWAY SURFACE
1'-6"
12'-6"
4'-0"
14'-0"
SECTION A-A
1" GAP
BARRIER NOT SHOWN
IN ELEVATION
1" PEJF
ROADWAY SURFACE
1'-6"
12'-6"
4'-0"
36" DIA. DRILLED
SHAFT (TYP.)
2'-0"
10'-0"
2'-0"
14'-0"
SECTION B-B
FWD. ABUT. RETAINING WALL DETAILS - ALTERNATIVE 5
BRIDGE NO. SCI-823-0248
SR823 OVER SR335 & LITTLE SCIOTO RIVER
DESIGNED JDG
DRAWN MTH
REVIEWED MSL
DATE 11/26/07
CHECKED PJP
REVISED
STRUCTURE FILE NUMBER
SCI-823-0.00
PID 77366

inter-office communication

to: District 9 - Harry Fry, District Deputy Director date: August 11, 2006
Attn: Tom Barnitz , District Production Administrator

from: Tim Keller, P.E., Administrator, Office of Structural Engineering by: Reza Zandi

subject: SCI-823-XXXX over S.R.335 & Little Scioto River, PID 19415, Structure Type Study (Resubmission)

We have reviewed the information furnished in the Structure Type Study submittal prepared by TranSystems Corporation for the above referenced bridge and offer the following comments:

1. The department requires the bridge plans to be prepared by independent designer, checker and reviewer.

2. Please perform a cost comparison analysis and feasibility study between Alternative 1 and a four span bridge by eliminating Pier No. 4 and the fifth span in Alternative 1 and by locating the forward abutment roughly at Station 139+00.

3. Is the forward abutment footing width (6'-9") shown on page 7 of 9 adequate for spread footing type of abutment?

4. Even though the design of the tall piers always involves looking into slenderness and buckling issues we consider that the following pier types should be further evaluated for applicability.

    a. Braced cap and column

    b. Hollow columns with caps

    c. Narrower stem T-piers

    d. Other viable bridge pier types.

5. Verify if the 4'-0" thick pier footing cap is adequate for the Piers 3 and 4.

6. On the Site Plan, bearings condition (fix or expansion) for each substructure unit shall be designated.

7. Verify surface elevation as shown on Site Plan vs. boring log TR-29 elevation.

8. The gutter spread and inlet capacity calculations do not appear to be correct: We were unable to follow the logic used in the calculations. We have an example of how the spread sheet should be calculated and for some reason the spread sheet prepared by HJS has values that we can not verify. The calculations are probably more complicated than need be though. We have never seen curb opening calculations for scuppers. Most people only account for the water going over

the scupper going down the scupper, and do not account for flow going by the edge since the edge is only six inches long. Please investigate, revise and resubmit calculations as necessary.

9. Provide roadway catch basins as appropriate upstation of the forward approach slab so as to intercept roadway surface drainage prior to the bridge.

10. Provide roadway catch basins as appropriate off the rear approach slab to capture bridge deck drainage coming off the bridge.

11. Please show all the roadway catch basins adjacent to the bridge on the site plan.

12. We question the need for four 8'-0" diameter drilled shaft for Piers 3and 4?

13. An allowable bearing pressure of 20 tsf for hard sandstone with RQD values ranging from 70 to 100 percent is overly conservative. ODOT's general practice would be to use an allowable bearing pressure of 40 to 50 tsf for this rock.

14. The geotechnical report did not include recommended side resistance for the drilled shafts. This should be provided as some of the drilled shafts at the piers may be subjected to uplift loads.

15. The boring log for TR-33 did not include RQD values for the rock core.

After addressing the above review comments, please furnish us a copy of the revised site plan for further review.

Nothing in these comments is to be construed as authorizing extra work for which additional compensation may be claimed. If you believe that these comments require work outside the limits of the Scope of Services for this project, please contact this office before proceeding.

If you should have any questions regarding these comments, please contact our office.

TJK:JS: rz

c: Tim Keller, P.E., Office of Structural Engineering
Jawdat Siddiqi, P.E., Office of Structural Engineering
Dave Norris, P.E., District 9
Larry Wills, P.E. Dist 9
file

inter-office communication

to: Gary Cochenour, District 9 Production Administrator **date:** December 13, 2007

from: Tim Keller, Administrator, Office of Structural Engineering **by:** Jeff Crace, P.E.

subject: SCI-823-0248 over S. R. 335 and Little Scioto River; PID 77366; Structure Type Study

We have reviewed the information furnished in the structure type study submitted by TranSystems Corporation for the above referenced bridge and offer the following comments:

1) We recommend that Alternative 5 be advanced to detail design. We deem it more prudent to utilize our limited funds on a more traditional solution (alt.5) than an alternate which includes a retaining wall that is nearly 50 feet tall (alt. 4b). This is especially true when for all practical purposes the two alternates have the same estimated Total Relative Ownership Cost, which includes Life Cycle Maintenance cost [$31,156,000 (alt. 4b) vs. $31,475,000 (alt. 5)] and the same estimated Total Construction Cost [$21,160,000 (alt. 4b) vs. $20,830,000(alt. 5)].

2) Is it possible to utilize a spread footing for the left bridge forward abutment/retaining wall since the footing will be founded on bedrock?

3) We agree that the abutment/retaining wall for the left structure should be design utilizing active earth pressure. To allow this design methodology considers reducing the shoulder width across the length of thee structure by 1 inch on each side. This will result in a 3 inch separation between the median barriers.

4) Can the spans for the right bridge (5 spans) be adjusted (shortened) while maintaining a maximum abutment/retaining wall height is 30 feet or less?

   Generally the design of the structure should be optimized, in this case we have narrowed this optimization down to the span length versus abutment/retaining wall height/foundation type. This can be finalized by the phase 2 (detail design) consultant.

If there are questions regarding our review comments for this project, please contact our office.

TJK:JS:jc

c: District 9 - Tom Barnitz
District 9 – John Wetzel
District 9 – June Wayland
District 9 - Doug Buskirk
District 9 - Larry Wills
Preliminary Design
file

SECTION 2

DLZ

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Thirteen (13) 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

S:\Dept\Geotechnical\Forms\Borings\Legend ODOT English.doc

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.

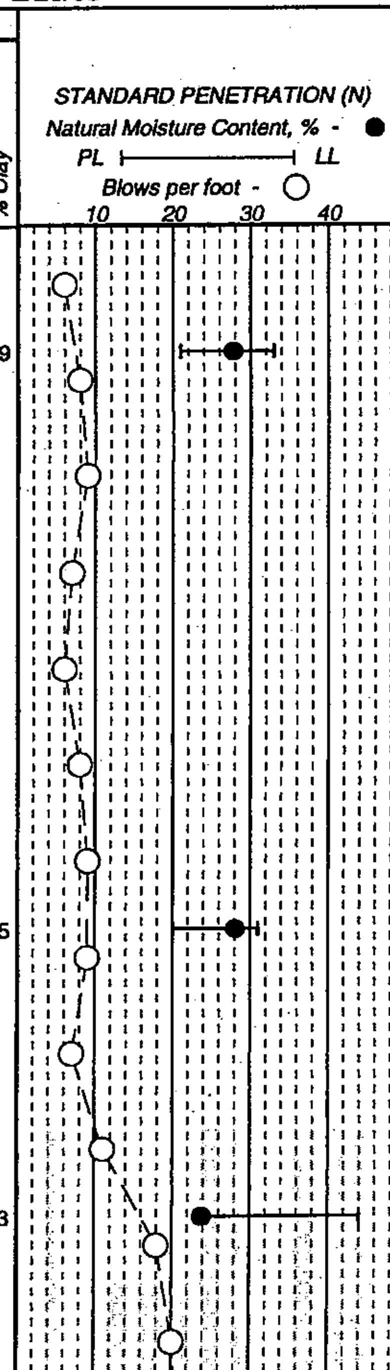
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-35 | Location: Sta. 132+52.2, 31.5 ft. LT of SR 823 CL | Date Drilled: 2/22/05 to 2/23/05

WATER OBSERVATIONS: Water seepage at: 7.0'-22.0', 63.0'-76.5'
Water level at completion: 50.0' (Prior to coring)
7.4' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	552.8												
0.3	552.5	1 2 4	18	1		3.75	Topsoil - 4" / Stiff to very stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand, contains roots; damp to moist.						
		3 3 5	18	2		2.0	@ 3.5', wet.	0	0	–	1	70	29
5		3 5 4	18	3		1.75							
		2 3 4	18	4		0.75	@ 7.5'-9.0', medium stiff.						
10		1 2 4	18	5		1.75							
		2 4 4	18	6		1.0							
15		2 3 6	18	7		3.0							
		1 3 6	18	8		1.25		0	2	–	5	58	35
20		2 3 4	18	9		1.25							
22.0	530.8	2 4 7	18	10		2.75	Very stiff to hard gray CLAY (A-7-6), trace fine sand; varved; damp.						
25		4 8 10	18	11		4.25		0	0	–	1	26	73
		5 8 12	18	12		4.0							
30													



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

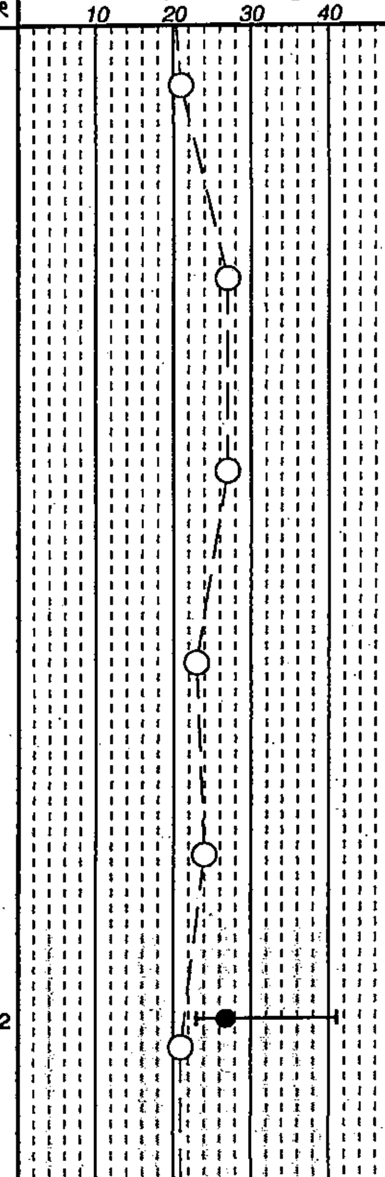
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-35 | Location: Sta. 132+52.2, 31.5 ft. LT of SR 823 CL | Date Drilled: 2/22/05 to 2/23/05

WATER OBSERVATIONS: Water seepage at: 7.0'-22.0', 63.0'-76.5'
Water level at completion: 50.0' (Prior to coring)
7.4' (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
30	522.8	6 8 13	18	13		4.5+	Very stiff to hard gray CLAY (A-7-6), trace fine sand; varved; damp.						
35		7 11 16	18	14		4.5+							
40		4 12 15	18	15		4.5+							
45		7 9 14	18	16		4.5+							
50		7 10 14	18	17		4.5+							
55		5 8 13	18	18		3.75	@ 55.0', slightly organic, contains very thin fine grained sand seams; damp to moist.	0	0	--	1	27	72
60													

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



FILE: 0121-3070-03 [11/20/2007 1:52 PM]

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

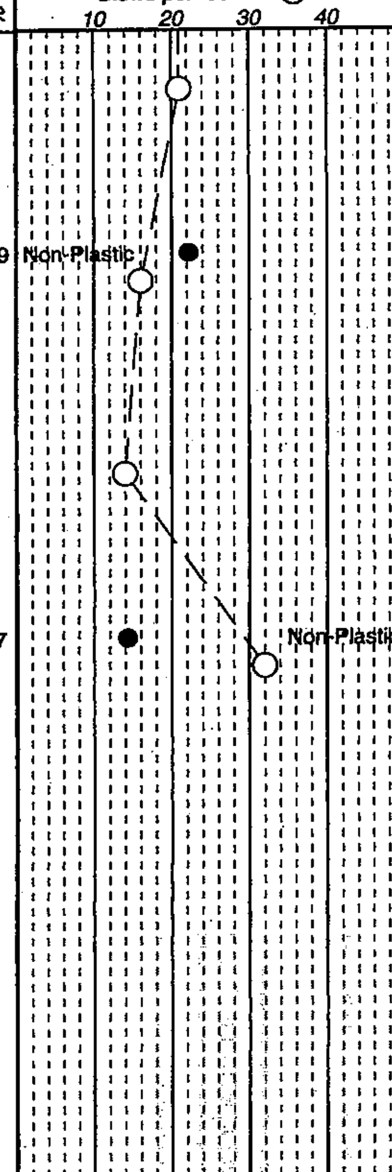
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-35 | Location: Sta. 132+52.2, 31.5 ft. LT of SR 823 CL | Date Drilled: 2/22/05 to 2/23/05

WATER OBSERVATIONS: Water seepage at: 7.0'-22.0', 63.0'-76.5'
Water level at completion: 50.0' (Prior to coring)
7.4' (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
60	492.8	4 9 12	18	19		2.75	Very stiff to hard gray CLAY (A-7-6), trace fine sand; varved; damp to moist.						
63.0	489.8						Medium dense gray SILT (A-4b), little clay, some fine to coarse sand; moist to wet.						
65		5 8 8	18	20				0	1	–	21	59	19
70		0 6 8	18	21									
73.0	479.8						Dense gray SANDY SILT (A-4a), trace gravel, trace clay; moist.						
75		11 16 16	12	22				6	26	–	24	37	7
80.0	472.8	Core 30"	Rec 24"	RQD 80%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded. @ 82.8',84.7',84.8', low angle clay filled fractures. @ 83.8'-83.9', high angle clay filled fracture. @ 83.2', qu = 6,952 psi. @ 83.9'-84.8', few to moderate argillaceous laminations. @ 84.8'-85.0', vuggy zone.						
84.8	468.0	Core 60"	Rec 60"	RQD 90%	R-2		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thinly to thickly bedded.						
		Core 60"	Rec 60"	RQD 100%	R-3								
90													

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



FILE: 0121-3070-03 [11/20/2007 1:52 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-35 | Location: Sta. 132+52.2, 31.5 ft. LT of SR 823 CL | Date Drilled: 2/22/05 to 2/23/05

WATER OBSERVATIONS: Water seepage at: 7.0'-22.0', 63.0'-76.5'
Water level at completion: 50.0' (Prior to coring)
7.4' (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) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
90	462.8						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thickly bedded, slightly fractured.							
95		Core 60"	Rec 60"	RQD 100%	R-4		@ 91.1', low angle clay filled fracture.							
		Core 36"	Rec 36"	RQD 100%	R-5									
100 / 100.5	452.3						Bottom of Boring - 100.5'							
105														
110														
115														
120														

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

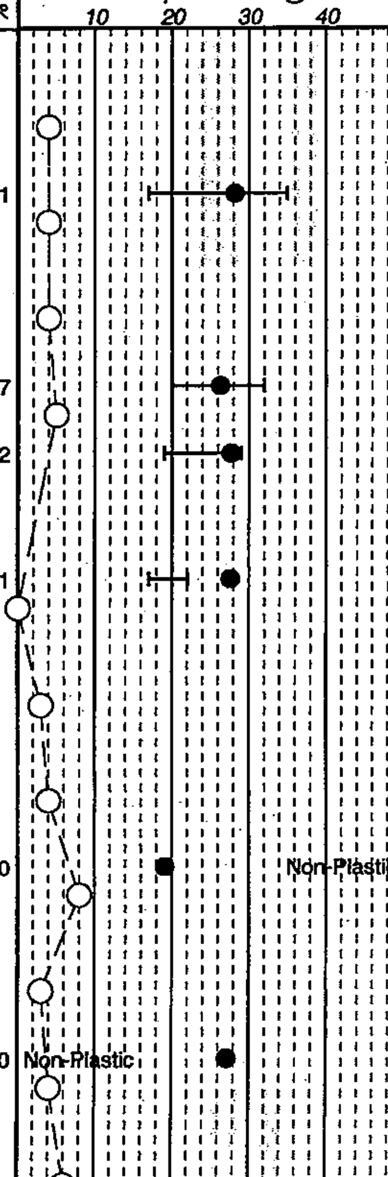
Client: TranSystems, Inc. | **Project:** SCI-823-0.00 | **Job No.** 0121-3070.03

LOG OF: Boring B-39 | **Location:** Sta. 134+39.9, 34.2 ft. RT of SR 823 CL | **Date Drilled:** 5/9/07

WATER OBSERVATIONS: Water seepage at: 16.0', 19.5', 23.5'
Water level at completion: 32.4' (prior to coring)
12.9' (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	508.0												
0.4	507.6						Topsoil - 5"						
		2 2 2	18	1		1.25	Stiff brown SILTY CLAY (A-6b), trace fine sand; moist.						
		2 2 2	18	2		1.25		0	0	--	2	67	31
5.5	502.5	WOH 2 2	18	3		1.0	Medium stiff brown SILT AND CLAY (A-6a), trace fine sand; moist.						
		2 3 2	18	4		0.75		0	0	--	8	55	37
10.0	498.0				ST-1	1.0-1.25	Stiff brown SILT (A-4b), some clay, little fine sand; moist.	0	0	--	11	57	32
13.0	495.0	WOH WOH WOH	18	5		-	Very soft to soft brown SANDY SILT (A-4a), little to some clay; moist to wet.	0	0	--	39	40	21
		WOH WOH 3	18	6		-							
18.0	490.0	2 2 2	18	7A 7B		1.0	Medium stiff to stiff mottled brown and gray SILTY CLAY (A-6b), trace fine sand; moist.						
19.5	488.5						Loose gray SILT (A-4b), little clay, some to "and" fine to coarse sand; moist to wet. @ 19.5'-20.5', brown.						
		1 3 5	18	8				0	2	--	20	58	20
		2 1 2	16	9									
		1 2 2	18	10			@ 26.0', attempted to press Shelby Tube, no recovery.	0	0	--	37	53	10
		2 3 3	14	11			@ 28.5', attempted to press Shelby Tube, no recovery.						
30													

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



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

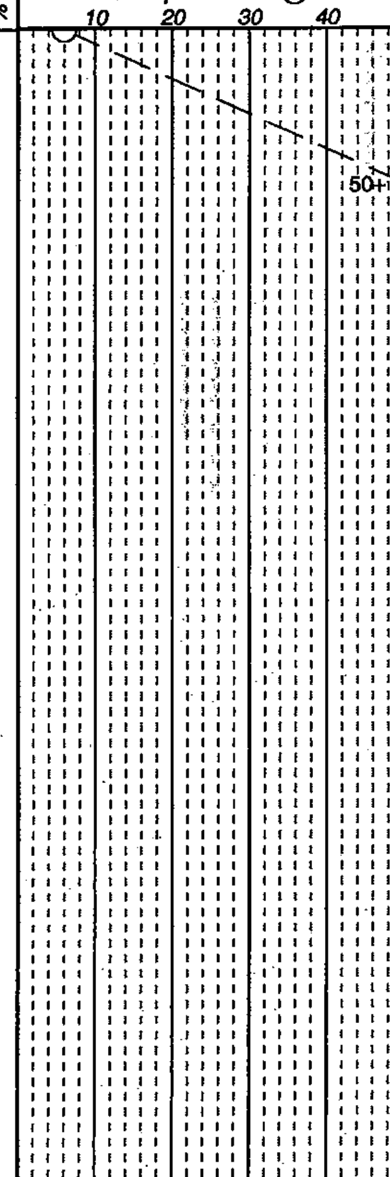
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-39 | Location: Sta. 134+39.9, 34.2 ft. RT of SR 823 CL | Date Drilled: 5/9/07

WATER OBSERVATIONS: Water seepage at: 16.0', 19.5', 23.5'
Water level at completion: 32.4' (prior to coring)
12.9' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
30	478.0						Loose brown SILT (A-4b), some fine to coarse sand, trace to little clay; contains rock fragments; wet.						
33.7	474.3	50/2	1	12		-	Medium hard gray SANDSTONE; fine grained, moderately to highly weathered, arenaceous, highly fractured.						
		Core 67"	Rec 67"	RQD 58%	R-1								
		Core 60"	Rec 59"	RQD 98%	R-2		@ 38.8', slightly weathered, slightly arenaceous, slightly to moderately fractured.						
		Core 120"	Rec 120"	RQD 99%	R-3		@ 48.0'-48.6', 49.6'-50.2', fine to medium grained.						
54.3	453.7						Bottom of Boring - 54.3'						

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

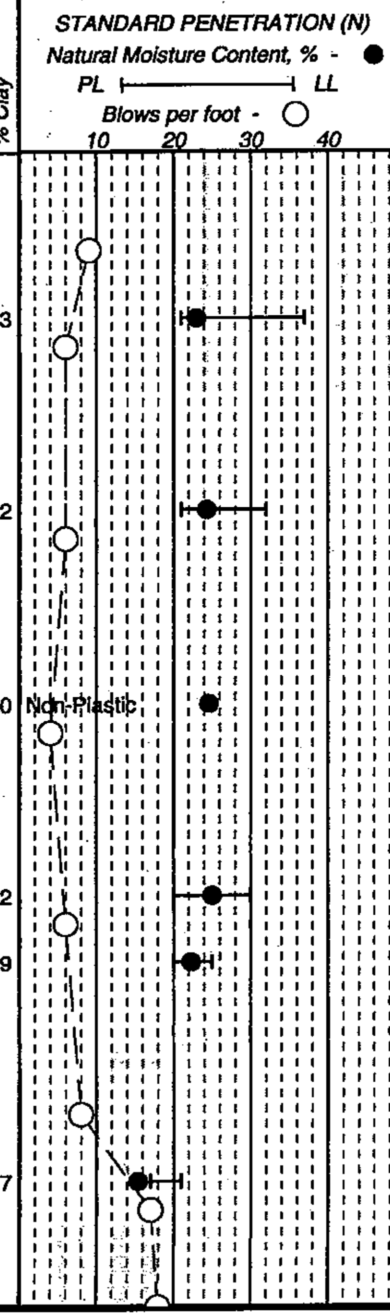


DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring B-40	Location: Sta. 136+89.0, 33.6 ft. LT of SR 823 CL	Date Drilled: 05/11/07 to 05/14/07

WATER OBSERVATIONS: Water seepage at: 34.7'-35.0', 43.5'-45.0'
Water level at completion: 29.8' (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	529.0												
0.4	528.6						Topsoil - 5"						
		2 4 5	16	1		3.25	Very stiff brown SANDY SILT (A-4a), little clay, little gravel; damp to moist.						
3.0	526.0	3 3 3	16	2		3.0	Very stiff brown SILTY CLAY (A-6b), trace fine to coarse sand; moist.	0	1	--	2	54	43
5					ST-1		@ 5.0', 6" recovery in Shelby Tube.						
7.5	521.5	2 2 4	15	3		2.5	Very stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand; moist.	0	1	--	2	65	32
10					ST-2		@ 10.0', 4" recovery in Shelby Tube.						
12.0	517.0	1 2 2	18	4			Very loose to loose brown and gray SILT (A-4b), little to some clay, little fine sand; moist to wet.	0	0	--	16	64	20
15.0	514.0		24	5	ST-3	1.0	Stiff to very stiff gray SILT (A-4b), little to some clay, trace fine sand; moist. @ 15.0', no recovery in Shelby Tube, pushed split spoon.						
		2 3 3	18	6		1.5		0	0	--	8	70	22
20					ST-4	1.0-2.0	@ 20.0', 24" recovery in Shelby Tube. @ 21.0'-30.0', contains rock fragments.	0	1	--	3	77	19
25		3 4 4	10	7		2.75							
		4 8 9	14	8		4.5+	@ 26.0'-27.5', hard, some fine to coarse sand, damp. @ 26.0'-30.0', brown.	2	15	--	11	55	17
30		7 9 9	13	9		2.25							



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

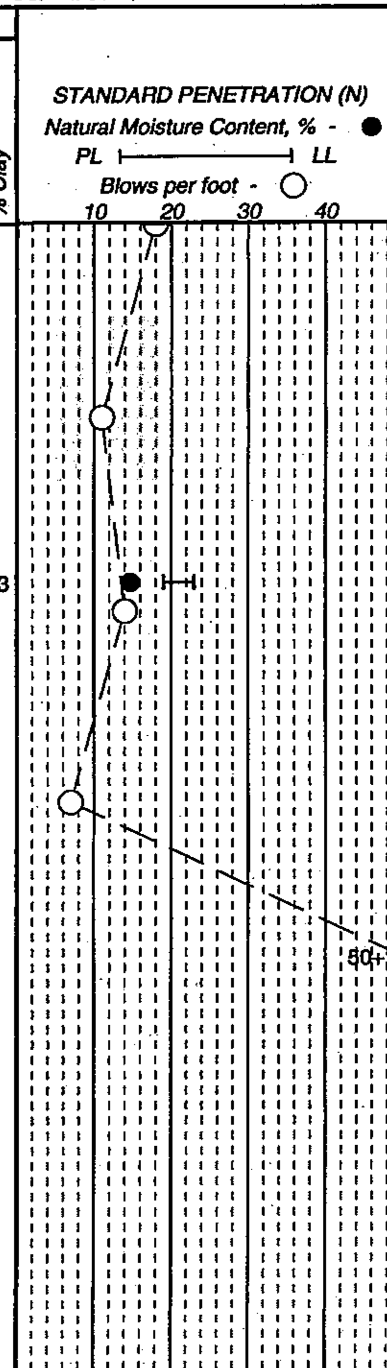
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-40 | Location: Sta. 136+89.0, 33.6 ft. LT of SR 823 CL | Date Drilled: 05/11/07 to 05/14/07

WATER OBSERVATIONS: Water seepage at: 34.7'-35.0', 43.5'-45.0'
Water level at completion: 29.8' (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
30.0	499.0						Hard gray SANDY SILT (A-4a); some fine to coarse sand, little clay, trace to little gravel; contains rock fragments; damp to moist.						
35		3 5 6	18	10		4.5+							
40		5 6 8	18	11		4.5+		7	20	--	12	47	13
42.5	486.5						Loose gray and brown FINE SAND (A-3), little silt; wet.						
45		3 3 4	17	12									
48.5	480.5	50/3	3	13			Severely weathered gray SANDSTONE.						
49.0	480.0	Core 60"	Rec 60"	RQD 86%	R-1		Medium hard light gray SANDSTONE; fine to medium grained, moderately to highly weathered, highly fractured. @ 49.6'-49.7', high angle fracture. @ 53.0'-53.6', 54.8'-56.1', argillaceous, dark gray.						
55		Core 60"	Rec 60"	RQD 91%	R-2								
59.0	470.0						Medium hard to hard light gray SANDSTONE; fine grained.						
60													



FILE: 0121-3070-03 [2/11/2008 10:33 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 B-40 | Location: Sta. 136+89.0, 33.6 ft. LT of SR 823 CL | Date Drilled: 05/11/07 to 05/14/07

WATER OBSERVATIONS: Water seepage at: 34.7'-35.0', 43.5'-45.0'
Water level at completion: 29.8' (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)
60	469.0	Core 120"	Rec 120"	RQD 94%	R-3		Medium hard to hard light gray SANDSTONE; fine grained, slightly weathered, slightly to moderately fractured. @ 61.5'-61.8', argillaceous zone. @ 64.7'-65.2', 65.9'-66.4', dark gray argillaceous zone. @ 65.6'-65.7', high angle fracture.							
69.0	460.0						Bottom of Boring - 69.0'							

Depth scale: 60, 65, 70, 75, 80, 85, 90

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

FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

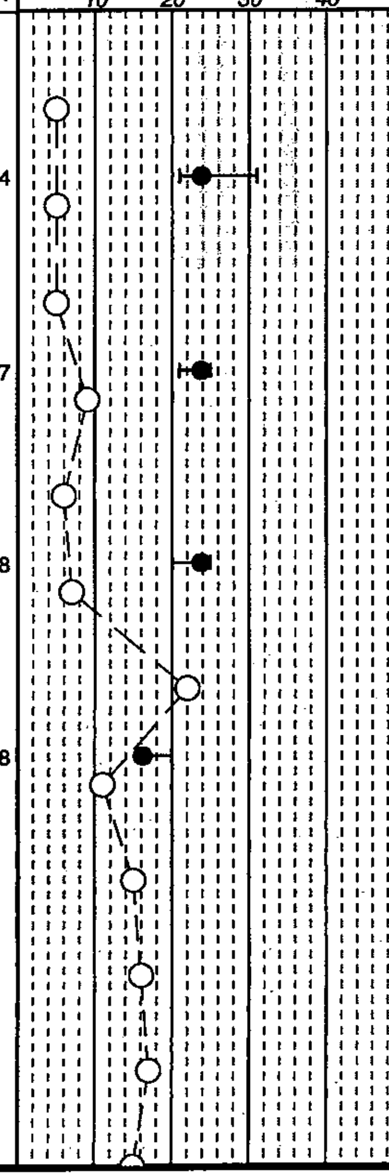
Client: TranSystems, Inc. | **Project:** SCI-823-0.00 | **Job No.** 0121-3070.03

LOG OF: Boring B-41 | **Location:** Sta. 136+99.7, 37.2 ft. RT of SR 823 CL | **Date Drilled:** 5/10/07

WATER OBSERVATIONS: Water seepage at: 28.5'-35.0'
Water level at completion: 29.2' (prior to coring)
20.3' (includes drilling water)
22.1' (after 16 hours)

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	518.9												
0.3	518.6						Topsoil - 4"						
		2 2 3	15	1		1.25	Medium stiff to stiff brown SILT (A-4b), little to some clay, trace to little fine to coarse sand, trace gravel; damp to moist.						
		1 2 3	16	2		1.5		0	0	–	4	62	34
5							@ 6.0'-15.0', mottled brown and gray.						
		2 2 3	18	3		1.0							
		2 4 5	18	4		1.0		0	0	–	15	68	17
10							@ 10.5', moist, trace clay.						
		3 2 4	18	5		0.75							
		1 3 4	18	6		0.75		0	1	–	18	63	18
15													
		6 9 13	18	7		2.5	@ 16.0'-17.5', very stiff. @ 16.0'-16.3',18.5'-19.0', reddish brown. @ 16.3'-17.5', 19.0'-20.0', gray to light gray.						
		5 5 6	17	8		1.0	@ 18.5'-20.0', some fine to coarse sand.	2	10	–	19	51	18
20													
20.5	498.4	7 7 8	11	9			Medium dense brown and gray mottled SANDY SILT (A-4a), some fine to coarse sand, little gravel, trace to little clay; contains rust stains and rock fragments; moist.						
23.5	495.4	5 8 8	18	10		4.5+	Hard gray SILT (A-4b), some fine to coarse sand, little clay; damp.						
25													
26.0	492.9	6 8 9	17	11			Medium dense brown and gray SANDY SILT (A-4a), little gravel, trace clay; contains rock fragments; damp.						
		3 7 8	16	12									
30													

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



FILE: 0121-3070-03 (2/11/2008 10:33 AM)

DLZ OHIO INC. * 6121 HUNTLEY ROAD, CO[illegible]MBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-41 | Location: Sta. 136+99.7, 37.2 ft. RT of SR 823 CL | Date Drilled: 5/10/07

WATER OBSERVATIONS: Water seepage at: 28.5'-35.0'
Water level at completion: 29.2' (prior to coring)
20.3' (includes drilling water)
22.1' (after 16 hours)

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) Blows per foot
30.0	488.9						Medium stiff gray SILT (A-4b), little fine sand, little gravel; contains rock fragments; damp to moist.							
35		WOH WOH 2	20	13		0.75								2
40.0	478.9	9 15 50/2	14	14		--	@ 38.5'-40.0', hard.							50+
42.9	476.0	Core 48"	Rec 48"	RQD 69%	R-1		Medium hard gray SANDSTONE; fine grained, moderately to highly weathered, argillaceous, highly fractured to broken, contains argillaceous low angle fractures.							
46.4	472.5						Soft to medium hard gray SHALE interbedded with SANDSTONE; very fine to fine grained, highly weathered, broken.							
50, 55		Core 120"	Rec 116"	RQD 74%	R-2		Medium hard gray SANDSTONE; fine to medium grained, slightly to moderately weathered, micaceous, slightly to moderately fractured, contains typical low angle fractures.							
60.0	458.9	Core 72"	Rec 72"	RQD 100%	R-3		Bottom of Boring - 60.0'							

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

FILE: 0121-3070-03 [2/11/2008 10:33 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 B-41 | **Location:** Sta. 136+99.7, 37.2 ft. RT of SR 823 CL | **Date Drilled:** 5/10/07

WATER OBSERVATIONS: Water seepage at: 28.5'-35.0'
Water level at completion: 29.2' (prior to coring)
20.3' (includes drilling water)
22.1' (after 16 hours)

Depth (ft)	Elev. (ft) 458.9	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
60														
65														
70														
75														
80														
85														
90														

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-42	Location: Sta. 138+99.0, 32.7 ft. LT of SR 823 CL	Date Drilled: 05/22/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 12.6' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
0	608.1													
0.5	607.6						Topsoil - 6"							
2.0	606.1	4 20 27	18	1A 1B			Dense brown SANDY SILT (A-4a), little gravel, trace sitly clay; damp.	39	11	–	6	34	10	Non-Plastic
		32 50/5	10	2			Severely weathered brown SANDSTONE, very fine to fine grained.							50+
4.5	603.6	Core 36"	Rec 33"	RQD 25%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, highly weathered, argillaceous, slightly micaceous, massive, highly fractured to broken, contains iron stained, argillaceous fractures. @ 5.0'-5.2', high angle fracture. @ 5.7'-5.9', possible lost recovery. @ 8.9'-9.2', 9.5'-9.8', highly weathered to decomposed, broken.							
10		Core 60"	Rec 60"	RQD 70%	R-2		@ 11.7'-11.9', argillaceous, broken zone.							
12.2	595.9													
15		Core 60"	Rec 60"	RQD 71%	R-3		Soft to medium hard brown SANDSTONE; very fine to fine grained, highly weathered, argillaceous, massive, highly fractured to broken, contains iron staining, contains few to moderate argillaceous laminations. @ 14.3'-14.5', 14.7'-15.3', 16.8'-17.0', 17.8'-18.0', 18.8'-19.0', 20.0'-20.3', 20.6'-20.9', iron stained high angle fractures.							
20		Core 60"	Rec 60"	RQD 86%	R-4		@ 21.1', gray.							
25		Core 60"	Rec 56"	RQD 39%	R-5		@ 23.5', high angle fracture brown, decomposed, broken.							
25.6	582.5						Medium hard brownish gray SANDSTONE; very fine to fine grained, moderately to highly weathered, micaceous, argillaceous, moderately to highly fractured, contains few argillaceous laminations. @ 26.4'-26.6', iron stained high angle fracture.							
30		Core 60"	Rec 60"	RQD 78%	R-6									

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

LOG OF: Boring B-42 | Location: Sta. 138+99.0, 32.7 ft. LT of SR 823 CL | Date Drilled: 05/22/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 12.6' (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	578.1						Medium hard gray SANDSTONE; very fine to fine grained, moderately to slightly weathered, micaceous, argillaceous, moderately to slightly fractured. @ 26.6', gray, contains argillaceous low angle fractures.							
35		Core 60"	Rec 60"	RQD 100%	R-7									
40		Core 60"	Rec 60"	RQD 100%	R-8									
		Core 24"	Rec 24"	RQD 100%	R-9									
44.5	563.6						Bottom of Boring - 44.5'							
45														
50														
55														
60														

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

FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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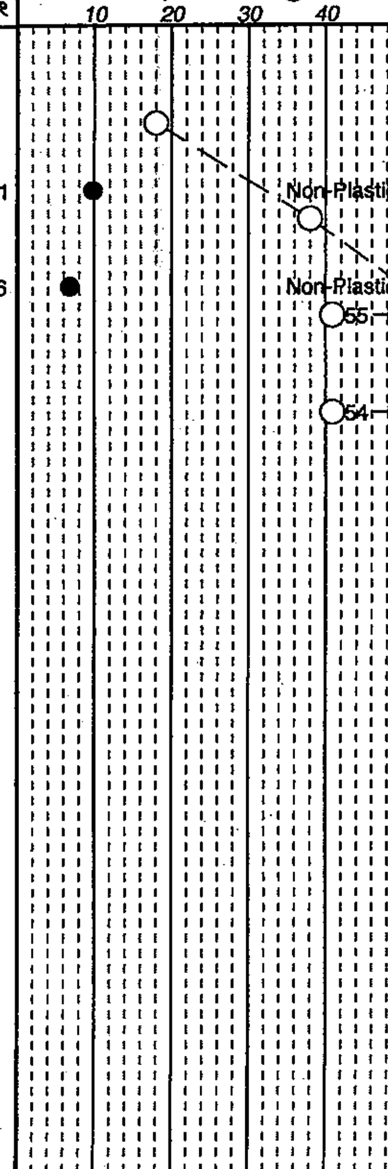
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-43 | Location: Sta. 139+00.9, 43.0 ft. RT of SR 823 CL | Date Drilled: 05/17/07 to 05/18/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 28.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	573.4												
0.4	573.0						Topsoil - 5"						
		7 7 11	18	1			Medium dense to dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace to little clay; contains sandstone fragments; dry to damp.						
5		26 18 20	16	2				54	8	–	5	22	11
		33 24 31	15	3			@ 6.0'-7.5', very dense.	60	8	--	5	21	6
8.0	565.4	10 16 38	15	4			Severely weathered brownish gray SANDSTONE.						
10.5	562.9	Core 18"	Rec 18"	RQD 91%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, highly weathered, micaceous, massive, highly fractured. @ 11.6', thin clay filled fracture. @ 12.0'-13.4', iron stained low angle fractures.						
15		Core 60"	Rec 60"	RQD 96%	R-2		@ 14.5', brownish-gray, moderately to highly fractured.						
20		Core 60"	Rec 60"	RQD 85%	R-3		@ 17.4'-17.5', iron stained high angle fracture. @ 17.6', gray, contains pyritic halos. @ 21.2'-22.6', iron stained zone.						
25		Core 60"	Rec 60"	RQD 96%	R-4		@ 23.4'-23.5', clay filled high angle fracture.						
25.5	547.9						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, fossiliferous, massive, moderately fractured. @ 27.3', slightly to moderately fractured.						
30		Core	Rec	RQD	R-5		@ 29.1', iron stained low angle fracture.						

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



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

LOG OF: Boring B-43 | Location: Sta. 139+00.9, 43.0 ft. RT of SR 823 CL | Date Drilled: 05/17/07 to 05/18/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 28.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% 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	543.4	60"	60"	94%			Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, micaceous, fossiliferous, massive, moderately fractured.							
35		Core 60"	Rec 60"	RQD 93%	R-6		@ 35.8'-35.9', 36.0'-36.1', iron stained high angle fractures.							
		Core 42"	Rec 42"	RQD 95%	R-7		@ 37.2', thin clay filled low angle fracture.							
40 / 40.5	532.9						Bottom of Boring - 40.5'							
45														
50														
55														
60														

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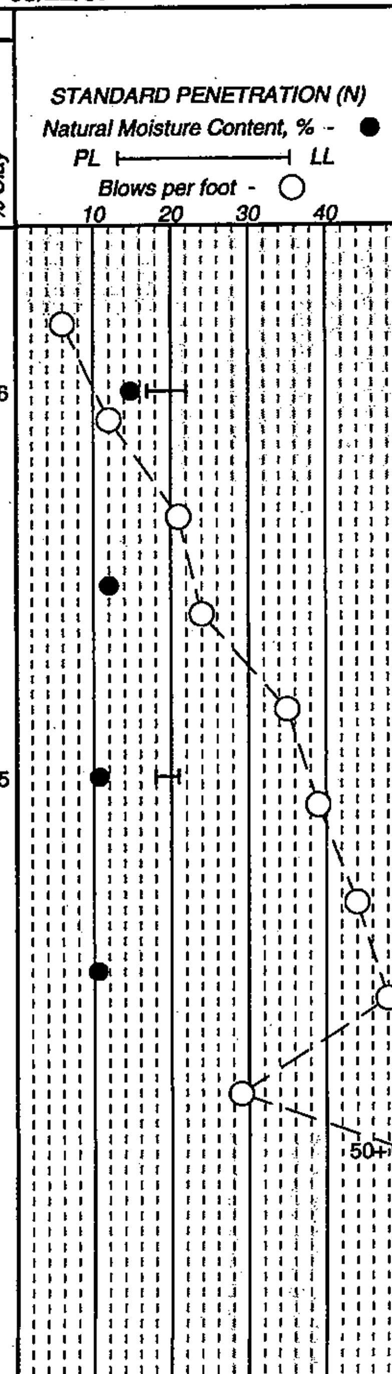
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-44 | Location: Sta. 140+14.9, 37.8 ft RT of SR 823 CL | Date Drilled: 05/21/07 to 05/22/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 10.5' (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	602.0												
0.3	601.7						Topsoil - 4"						
		3 2 4	12	1		--	Very stiff brown SANDY SILT (A-4a), little to some clay, trace gravel; contains sandstone fragments; dry to damp.						
		4 4 8	18	2		--		9	8	--	12	45	26
		5 7 14	15	3		--							
		6 11 13	18	4		--							
		8 13 22	18	5									
13.5	588.5	13 18 21	18	6			Medium dense to dense brown SILT (A-4b), some fine to coarse sand, little clay, trace gravel; damp.	3	19	--	12	51	15
		11 17 27	18	7									
		26 21 27	18	8									
		9 11 18	16	9									
23.5	578.5	50/5	5	10			Severely weathered brown SANDSTONE.						
24.0	578.0	Core 30"	Rec 30"	RQD 43%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, highly weathered, argillaceous, massive, highly fractured to broken, contains iron stained low angle fractures. @ 25.8'-26.1', high angle fracture. @ 26.1', moderately to highly fractured.						
28.0	574.0	Core 60"	Rec 60"	RQD 95%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, micaceous, slightly argillaceous.						

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



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

LOG OF: Boring B-44 **Location:** Sta. 140+14.9, 37.8 ft RT of SR 823 CL **Date Drilled:** 05/21/07 to 05/22/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 10.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	572.0						Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, micaceous, slightly argillaceous, moderately to highly fractured.							
							@ 32.3'-32.6', argillaceous, broken zone.							
							@ 32.7'-33.7', brown, iron stained zone.							
		Core 60"	Rec 60"	RQD 76%	R-3		@ 32.5', slightly to moderately fractured.							
35														
		Core 60"	Rec 59"	RQD 98%	R-4									
40														
		Core 60"	Rec 60"	RQD 92%	R-5		@ 43.7'-44.0', broken.							
45														
		Core 60"	Rec 60"	RQD 95%	R-6									
50														
		Core 60"	Rec 60"	RQD 100%	R-7									
55														
							@ 56.3'-56.8', high angle fracture.							
		Core 60"	Rec 59"	RQD 98%	R-8									
60														

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

LOG OF: Boring B-44 | Location: Sta. 140+14.9, 37.8 ft RT of SR 823 CL | Date Drilled: 05/21/07 to 05/22/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 10.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
60	542.0	Core 30"	Rec 30"	RQD 100%	R-9		Medium hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, micaceous, slightly argillaceous, massive, moderately to highly fractured.							
64.0	538.0						Bottom of Boring - 64.0'							
65														
70														
75														
80														
85														
90														

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

LOG OF: Boring TR-30 | Location: Sta. 139+35.0, 52.3 ft. LT of SR 823 CL | Date Drilled: 3/8/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 12.2' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
0	637.1													
0.4	636.7						Topsoil - 5" / 3.2' soil removed before drilling							
		Core 120"	Rec 120"	RQD 62%	R-1		Soft to medium hard gray and brown SANDSTONE; very fine to fine grained, highly weathered to decomposed, argillaceous, thinly to thickly bedded, moderatly fractured. @ 1.0'-1.3',5.0'-5.1', broken zones. @ 3.6'-3.9', clay filled zone. @ 5.8'-6.4', qu = 5,441 psi. @ 3.9'-4.7', high angle clay filled fracture.							
7.5	629.6						Medium hard gray very fine grained SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded, slightly fractured.							
		Core 120"	Rec 120"	RQD 100%	R-2		@ 11.9',15.9',16.8',18.8' low angle clay filled fractures.							
20.0	617.1						Bottom of Boring - 20.0'							

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

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

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

LOG OF: Boring TR-31 | Location: Sta. 138+68.7, 106.5 ft. LT of SR 823 CL | Date Drilled: 3/8/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 5.3' (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	STANDARD PENETRATION (N)
0	621.4													
0.5	620.9						Topsoil - 6" / 4.0' soil removed before drilling							
0.5–7.9		Core 120"	Rec 110"	RQD 50%	R-1		Soft to medium hard brown SANDSTONE; very fine to fine grained, highly weathered to decomposed, argillaceous, thinly to thickly bedded, highly fractured, with typically low angle clay filled fractures. @ 0.0'-0.9', lost recovery. @ 0.9'-2.0', broken zones. @ 5.1'-5.4',6.8'-7.0',7.7'-7.9' high angle clay filled fractures. @ 6.3'-6.7', qu = 1,254 psi. @ 7.9'-9.8', iron staining.							
7.9	613.5													
7.9–20.0		Core 120"	Rec 116"	RQD 96%	R-2		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded, unfractured to slightly fractured. @ 10.4'-10.5', broken zone. @ 11.0'-11.4',11.9'- 12.1',15.2', rust stained zones. @ 11.2', low angle rust stained fracture. @ 19.6'-20.0', lost recovery.							
20.0	601.4						Bottom of Boring - 20.0'							

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

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

FILE: 0121-3070-03 [2/11/2008 10:33 AM]

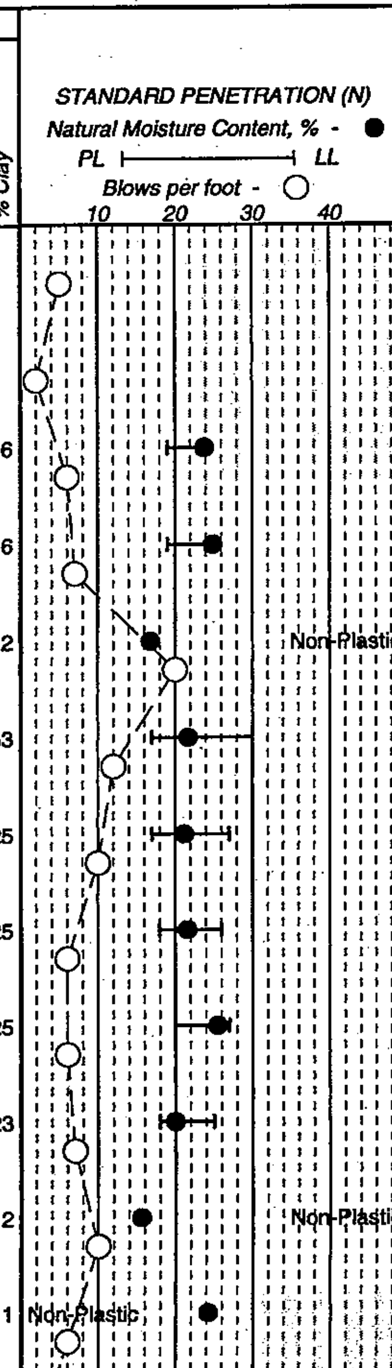
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-32 **Location:** Sta. 136+60.6, 10.4 ft. LT of SR 823 CL **Date Drilled:** 3/10/05

WATER OBSERVATIONS: Water seepage at: 4.0'-11.0', 26.5'-38.0'
Water level at completion: 7.0' (Prior to coring)
3.0' (Including 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 / 0.3	515.1 / 514.8	1 2 3	18	1		–	Topsoil - 4" / 2.6' soil removed before drilling Soft to medium stiff brown SILT (A-4b), little to some fine sand; moist to wet. @ 0.0'-1.5', organic odor.						
		1 1 1	16	2		–							
5		2 2 4	15	3		–		0	0	–	26	58	16
		3 3 4	17	4		–		0	0	–	15	69	16
10		4 12 8	18	5			@ 10.0'-12.5', medium dense, trace coarse sand.	1	5	–	24	58	12
12.5	502.6	4 5 7	16	6		1.5	Stiff gray SILT AND CLAY (A-6a), little fine to coarse sand; moist.	0	3	–	15	49	33
15.0	500.1	5 5 5	16	7		–	Medium stiff to stiff gray SILT (A-4b), trace to little fine to coarse sand, trace gravel; moist to wet.	0	1	–	8	66	25
		3 3 3	17	8		–		0	1	–	11	63	25
20		WOH 3 3	15	9		–		0	0	–	8	67	25
		3 4 3	18	10		–	@ 22.5'-29.0', gray and brown, little to some fine to coarse sand.	2	2	–	18	55	23
25		1 4 6	18	11			@ 25.0'-30.0', loose.	7	9	–	14	58	12
		2 2 4	18	12				0	1	–	33	55	11
30													



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

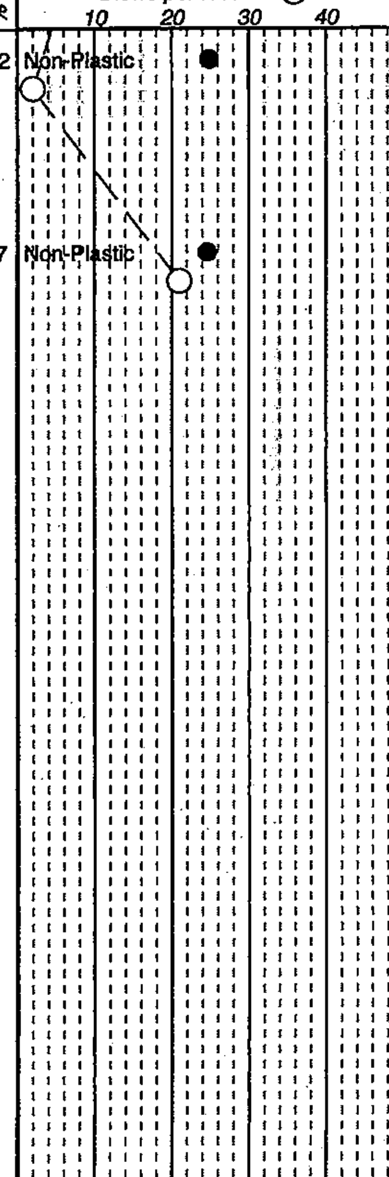
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-32 | Location: Sta. 136+60.6, 10.4 ft. LT of SR 823 CL | Date Drilled: 3/10/05

WATER OBSERVATIONS: Water seepage at: 4.0'-11.0', 26.5'-38.0'
Water level at completion: 7.0' (Prior to coring)
3.0' (Including 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
30	485.1	1 1 1	17	13			Loose gray SILT (A-4b), some fine to coarse sand; moist to wet.	0	1	–	31	56	12
33.0	482.1						Medium dense gray SANDY SILT (A-4a), trace gravel; wet.						
35		2 8 13	18	14				3	3	--	51	36	7
39.0	476.1	Core 120"	Rec 120"	RQD 100%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded, slightly fractured. @ 39.0'-40.5', highly weathered to decomposed. @ 43.2', qu = 9,344 psi. @ 39.6',42.0',43.1', low angle fractures.						
49		Core 120"	Rec 120"	RQD 100%	R-2								
59.0	456.1						Bottom of Boring - 59.0'						

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



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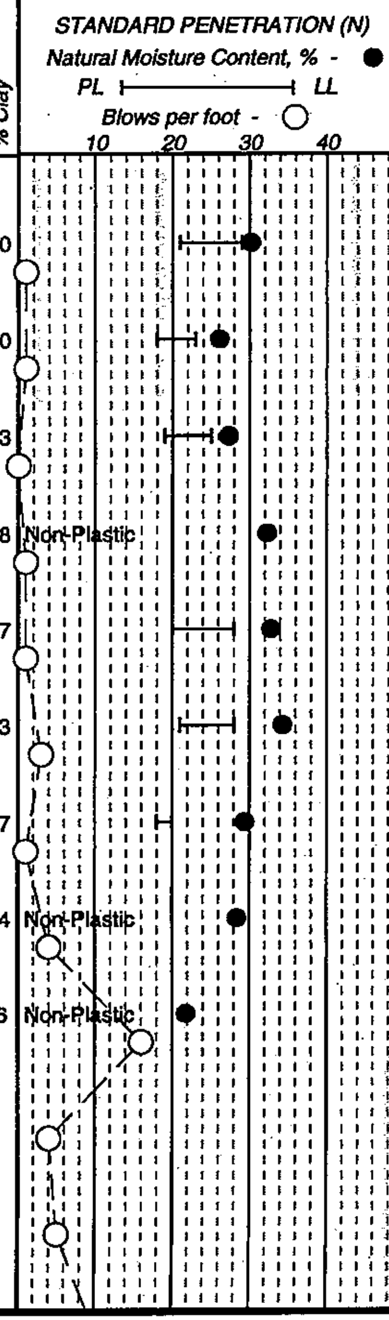
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-33 | Location: Sta. 134+67.6, 60.6 ft. LT of SR 823 CL | Date Drilled: 2/23/05 to 2/24/05

WATER OBSERVATIONS: Water seepage at: 5.5'-34.0'
Water level at completion: 15.0' (Prior to coring)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	502.7												
0.4	502.3						Topsoil - 5"						
							Very soft to soft brown SILT (A-4b), little fine sand; wet.						
		WOH WOH 1	12	1		0.25		0	0	–	18	62	20
3.0	499.7						Very soft to soft brown SANDY SILT (A-4a), little to some clay; wet.						
5		1 WOH 1	16	2		0.25		0	1	–	32	47	20
5.5	497.2						Very soft to soft brown SILT (A-4b), trace to little fine sand; wet.						
		WOH WOH WOH	3	3		0.25		0	0	–	17	60	23
							@ 8.5'-10.0', very loose.						
10		WOH WOH 1	18	4				0	0	–	16	66	18
		WOH WOH 1	18	5		0.25		0	0	–	14	59	27
15		WOH 1 2	18	6		0.5		0	0	–	4	73	23
							@ 16.0'-20.0', some fine sand.						
		WOH WOH 1	18	7		0.25		0	0	–	32	51	17
							@ 18.5'-20.0', very loose to loose.						
20		WOH 2 2	18	8				0	0	–	36	50	14
21.0	481.7						Medium dense gray COARSE AND FINE SAND (A-3a), some silt, trace gravel, trace clay; wet.						
		2 9 7	18	9				8	10	–	54	22	6
23.5	479.2						Loose gray FINE SAND (A-3), little to some silt; wet.						
25		2 2 2	18	10									
		1 2 3	18	11									
28.5	474.2						Loose gray SANDY SILT (A-4a); moist to wet.						
30		2 4		12									



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

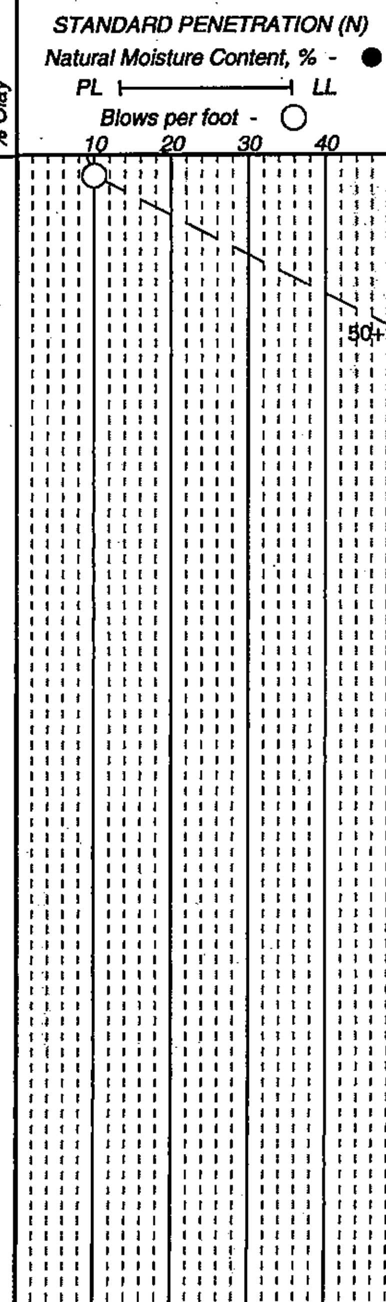
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-33 | Location: Sta. 134+67.6, 60.6 ft. LT of SR 823 CL | Date Drilled: 2/23/05 to 2/24/05

WATER OBSERVATIONS: Water seepage at: 5.5'-34.0'
Water level at completion: 15.0' (Prior to coring)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
30	472.7	6					Loose gray SANDY SILT (A-4a); moist to wet.						
34.1	468.6	50/1	0	13			Medium hard to hard gray SANDSTONE; very fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded. @ 34.6', high angle fracture. @ 34.7'-36.1', contains moderate argillaceous laminations.						
35		Core 42"	Rec 42"	RQD 100%	R-1								
36.1	466.6						Hard gray SANDSTONE; very fine grained, slighty weathered, argillaceous, micaceous, thinly to thickly bedded. @ 38.3', qu = 11,676 psi.						
40		Core 60"	Rec 60"	RQD 93%	R-2		@ 41.6'-42.4', contains few to moderate argillaceous laminations. @ 41.9', clay seam. @ 42.4', low angle fracture.						
45		Core 60"	Rec 60"	RQD 100%	R-3								
50		Core 60"	Rec 60"	RQD 100%	R-4								
		Core 18"	Rec 18"	RQD 100%	R-5		@ 53.3', iron staining.						
54.1	448.6						Bottom of Boring - 54.1'						
55													
60													



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

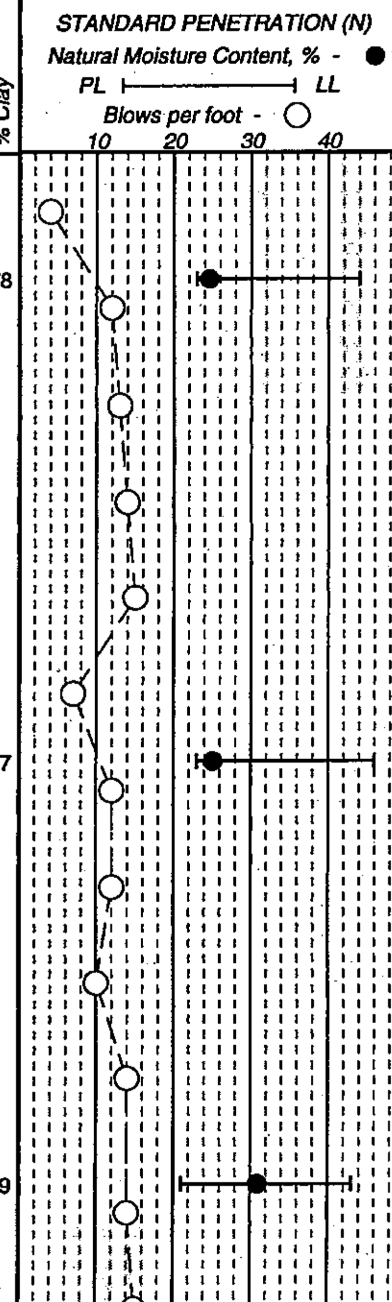
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-34 | *Location:* Sta. 133+61.1, 2.0 ft. LT of SR 823 CL | *Date Drilled:* 2/24/05 to 3/2/05

WATER OBSERVATIONS: Water seepage at: 30.0'-38.0'
Water level at completion: 20.0' (Prior to coring)
6.0' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	516.4												
0.6	515.8	1 2 2	16	1		2.5	Topsoil - 7" / 2-3' soil removed before drilling Very stiff brown SILT AND CLAY (A-6a); damp.						
2.0	514.4						Very stiff to hard brown CLAY (A-7-6), trace fine to coarse sand; varved; damp to moist.						
		4 5 7	18	2		4.0		0	0	--	1	21	78
5													
		4 5 8	18	3		4.5	@ 6.0', contains occasional very thin gray seams with desiccation cracking.						
		3 6 8	18	4		4.5+							
10							@ 10.0', gray.						
		3 6 9	18	5		4.5+							
		2 3 4	18	6		2.25							
15													
		2 5 7	18	7		3.25		0	0	--	1	22	77
		2 5 7	18	8		2.25	@ 17.5'-19.0', slightly organic.						
20													
		2 4 6	0	9		--							
		3 6 8	18	10		3.75							
25							@ 25.0', grayish brown; moist.						
		4 6 8	18	11		2.25		0	1	--	4	56	39
		3 6 9	0	12		--							
30													



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

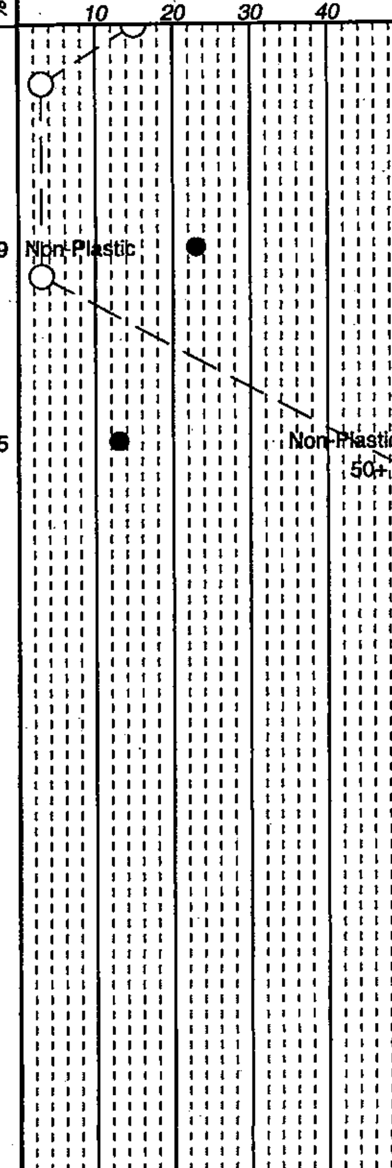
Client: TranSystems, Inc. **Project:** SCI-823-0.00 **Job No.** 0121-3070.03

LOG OF: Boring TR-34 **Location:** Sta. 133+61.1, 2.0 ft. LT of SR 823 CL **Date Drilled:** 2/24/05 to 3/2/05

WATER OBSERVATIONS: Water seepage at: 30.0'-38.0'
Water level at completion: 20.0' (Prior to coring)
6.0' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
30.0	486.4	2 1 2	0	13			Very loose gray SILT (A-4b), some fine sand, trace clay; wet.						
35		0 1 2	18	14				0	0	–	31	60	9
38.0	478.4						Medium dense gray GRAVEL WITH SAND AND SILT (A-2-4), trace clay; moist.						
40		10 12 50/4	14	15				23	26	–	19	27	5
42.0	474.4	Core 12"	Rec 12"	RQD 75%	R-1		Soft to medium hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly to thickly bedded, moderately to highly fractured. @ 43.3', qu = 9,993 psi. @ 44.5'-45.4', 46.6'-48.0', very fine grained, fissile after desiccation. @ 42.2',43.6',44.7',47.1',47.2' 47.6', low angle clay filled fractures. @ 44.2'-44.4',45.0'- 45.1',46.7', high angle clay filled fractures.						
45		Core 60"	Rec 60"	RQD 70%	R-2								
48.0	468.4						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, massive, unfractured to slightly fractured.						
50		Core 60"	Rec 60"	RQD 100%	R-3		@ 53.4'-54.3', very fine grained.						
55		Core 60"	Rec 60"	RQD 97%	R-4		@ 53.5', low angle clay filled fractures.						
60		Core 48"	Rec 48"	RQD 100%	R-5		@ 59.1'-59.5', red iron staining.						

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



FILE: 0121-3070-03 [2/11/2008 10:33 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-34 | Location: Sta. 133+61.1, 2.0 ft. LT of SR 823 CL | Date Drilled: 2/24/05 to 3/2/05

WATER OBSERVATIONS: Water seepage at: 30.0'-38.0'
Water level at completion: 20.0' (Prior to coring)
6.0' (Includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
60	456.4						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, massive, unfractured.							
62.0	454.4						Bottom of Boring - 62.0'							
65														
70														
75														
80														
85														
90														

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

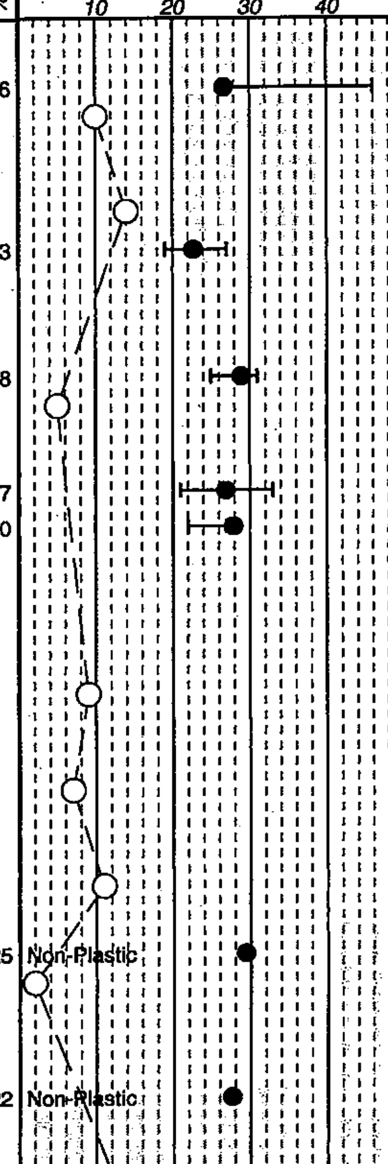
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-35A | Location: Sta. 131+29.4, 14.1 ft. RT of SR 823 CL | Date Drilled: 1/12/06

WATER OBSERVATIONS: Water seepage at: 24.5'-25.0', 68.5'-79.0'
Water level at completion: 67.8' (prior to coring)
13.4' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	554.6												
0.3	554.3						Limestone gravel (driveway) - 3"						
		3 4 6	12	1		3.0	Very stiff yellowish brown CLAY (A-7-6), some silt, trace fine sand; damp to moist.	0	0	--	2	32	66
		6 6 8	13	2		2.25							
5.0	549.6				P-1 P-1	1.0	Stiff brown SILT (A-4b), little fine sand, little to some clay; contains thin clay seams; moist.	0	0	--	4	63	33
		2 2 3	18	3		1.5		0	0	--	1	71	28
10													
11.0	543.6				P-2A	0.75	Medium stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand; moist.	0	1	--	1	51	47
12.4	542.2				P-2B	0.75	Medium stiff to stiff brown SILT (A-4b), some clay, trace fine to coarse sand; moist to wet.	0	2	--	2	66	30
15													
		2 4 5	18	4		1.25							
		12 3 4	18	5		1.00							
20													
20.5	534.1	3 5 6	18	6		1.5	Stiff gray CLAY (A-7-6), little to some silt, trace fine sand; damp.						
23.5	531.1	WOH 1 1	18	7			Very loose gray SILT (A-4b); moist to wet.	0	0	--	0	75	25
25													
					P-3			0	0	--	1	77	22
30													

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



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

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

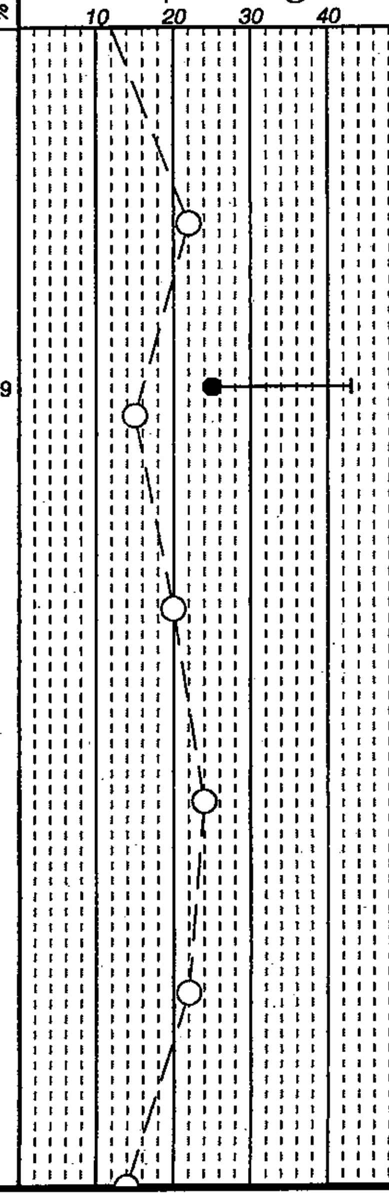
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-35A | *Location:* Sta. 131+29.4, 14.1 ft. RT of SR 823 CL | *Date Drilled:* 1/12/06

WATER OBSERVATIONS: Water seepage at: 24.5'-25.0', 68.5'-79.0'
Water level at completion: 67.8' (prior to coring)
13.4' (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
30	524.6						Loose gray SILT (A-4b); moist to wet.						
32.0	522.6						Very stiff to hard grayish brown CLAY (A-7-6), trace fine sand; damp to moist.						
35		5 9 13	18	8		4.5+							
40		4 6 9	18	9		4.5+		0	0	--	1	20	79
45		5 9 11	18	10		3.0							
50		5 10 14	18	11		4.5+							
55		5 9 13	18	12		4.5+							
60		4 7 7	18	13		2.75							

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



FILE: 0121-3070-03 [2/11/2008 10:33 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, C[illegible]MBUS, OHIO 43229 * (614)888-0040

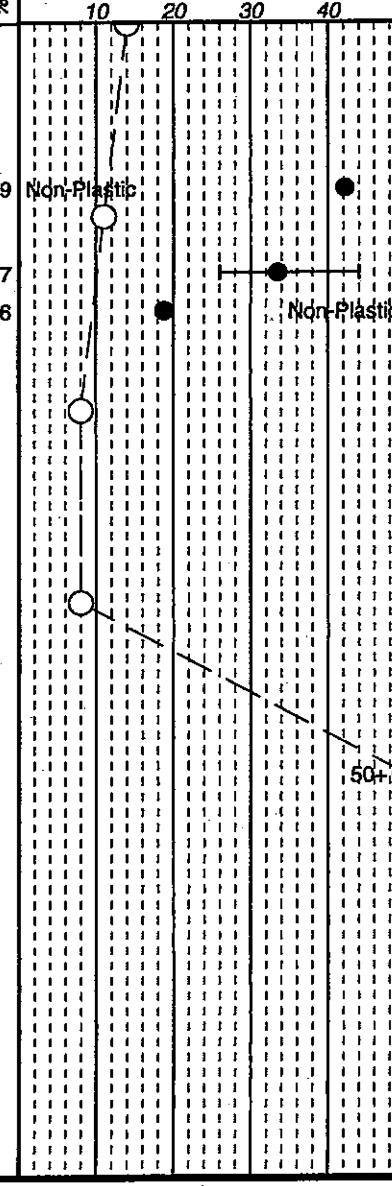
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-35A | Location: Sta. 131+29.4, 14.1 ft. RT of SR 823 CL | Date Drilled: 1/12/06

WATER OBSERVATIONS: Water seepage at: 24.5'-25.0', 68.5'-79.0'
Water level at completion: 67.8' (prior to coring)
13.4' (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
60	494.6						Very stiff to hard grayish brown CLAY (A-7-6), trace fine sand; damp to moist.						
62.0	492.6	4 5 6	18	14		1.5	Stiff mottled dark brown and black ORGANIC SILT (A-4b), trace fine to coarse sand, trace gravel; moist.	1	2	--	2	66	29
65.5	489.1				P-4A	1.0	Medium stiff to stiff dark brown CLAY (A-7-6), trace fine to coarse sand; moist.	0	6	--	3	64	27
66.9	487.7	3 4 4	18	15	P-4B	1.75	Stiff gray SILT (A-4b), little to some fine to coarse sand; damp to moist.	0	1	--	20	63	16
72.0	482.6	WOH 3 5	18	16			Loose gray FINE SAND (A-3), trace silty clay; moist to wet.						
77.0	477.6	5 50/5	10	17a 17b			Loose to medium dense brown COARSE AND FINE SAND (A-3a), trace to little silty clay, trace gravel; moist. @ 79.5', some silt, trace clay.						
81.0	473.6	Core 60"	Rec 60"	RQD 71%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered, argillaceous, micaceous, medium to thickly bedded, moderately fractured. @ 81.7', 82.6', 83.7', low angle fractures. @ 84.1'-84.7',84.3'-84.5', high angle fractures. @ 85.1', qu = 341 psi.						
86.0	468.6						Bottom of Boring - 86.0'						

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



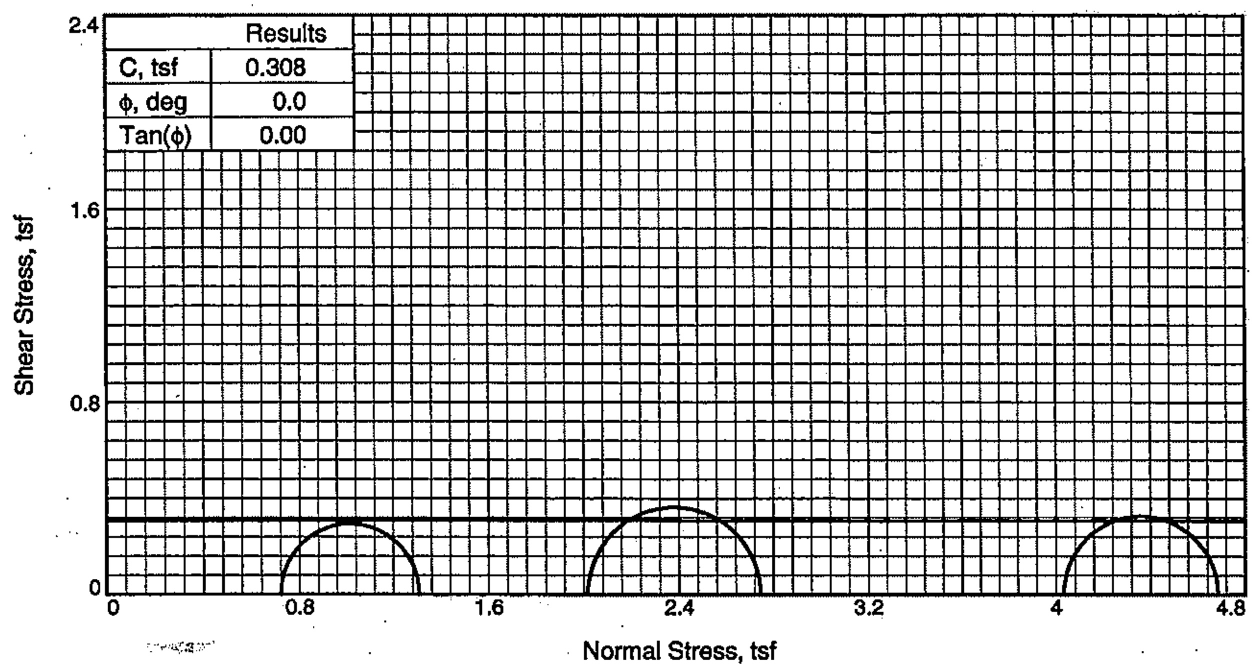
FILE: 0121-3070-03 [2/11/2008 10:33 AM]

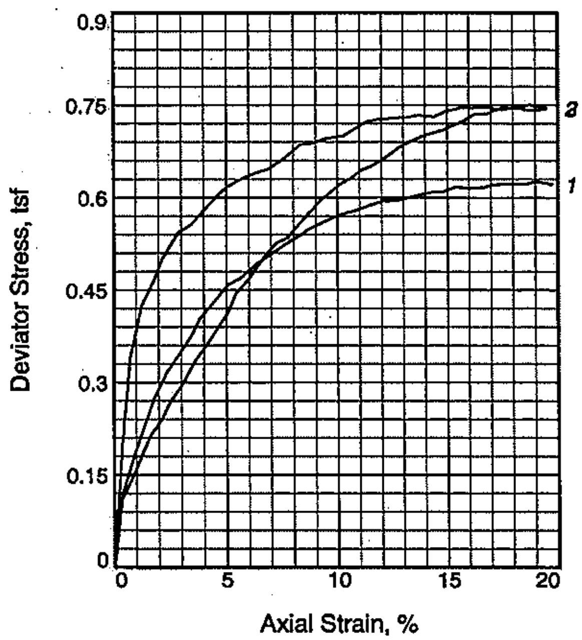
SECTION 3

DLZ

APPENDIX III

Laboratory Test Results





Sample No.		1	2	3
Initial	Water Content,	28.0	28.2	27.1
	Dry Density, pcf	95.5	94.2	97.1
	Saturation,	96.5	94.5	96.8
	Void Ratio	0.7980	0.8215	0.7687
	Diameter, in.	2.81	2.81	2.79
	Height, in.	5.52	5.55	5.54
At Test	Water Content,	28.3	27.8	27.5
	Dry Density, pcf	95.5	94.2	97.1
	Saturation,	97.4	93.2	98.5
	Void Ratio	0.7980	0.8215	0.7687
	Diameter, in.	2.81	2.81	2.79
	Height, in.	5.52	5.55	5.54
Strain rate, in./min.		0.06	0.06	0.06
Back Pressure, tsf		0.00	0.00	0.00
Cell Pressure, tsf		0.72	2.02	4.03
Fail. Stress, tsf		0.59	0.73	0.65
Ult. Stress, tsf		0.59	0.73	0.65
σ_1 Failure, tsf		1.31	2.74	4.68
σ_3 Failure, tsf		0.72	2.02	4.03

Type of Test:
Unconsolidated Undrained

Sample Type: 3" press tube

Description: Lean clay

LL= 29 **PL=** 19 **PI=** 10

Assumed Specific Gravity= 2.75

Remarks:

Figure ________

Client: TranSystems, Inc.

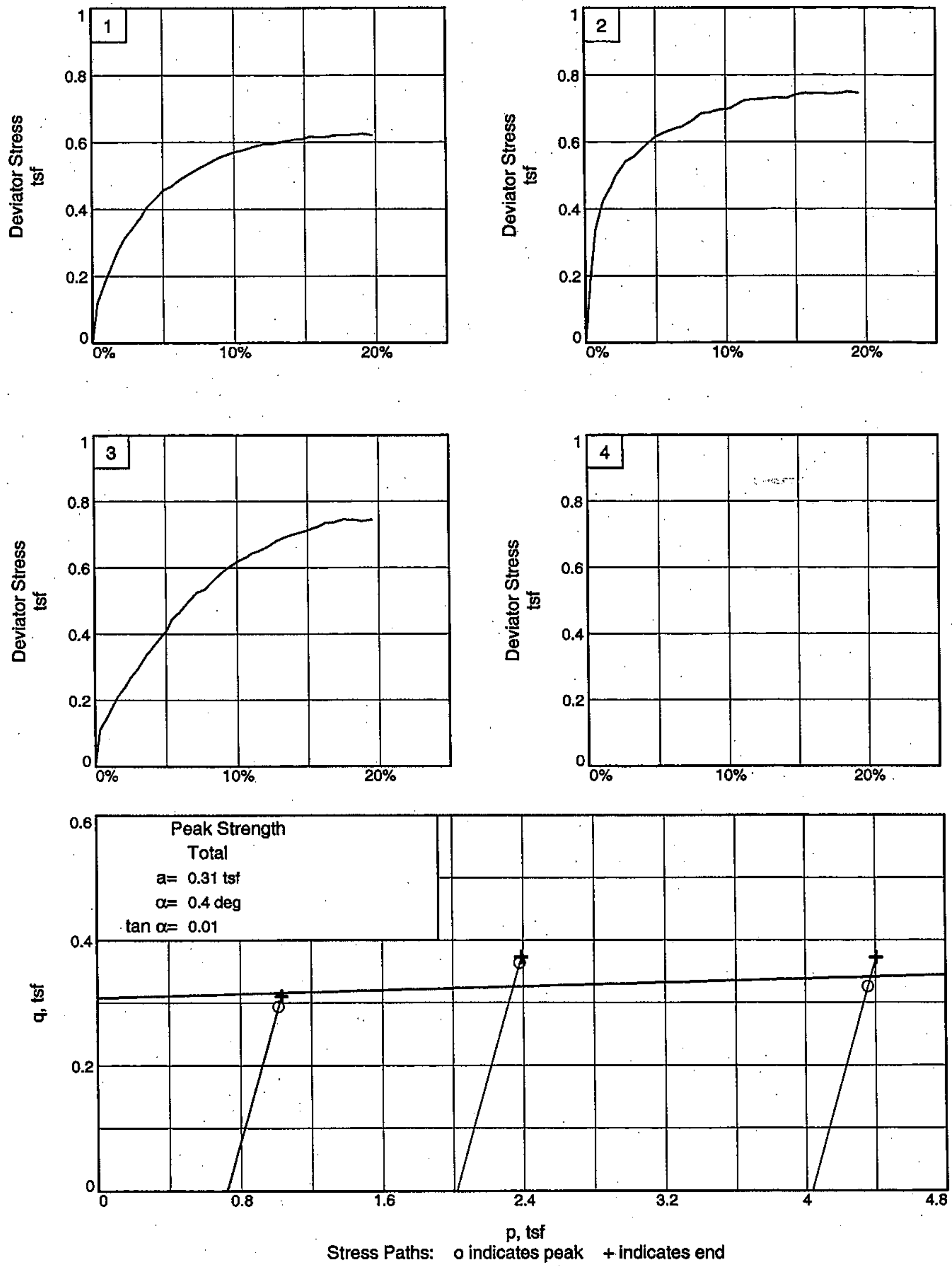
Project: SCI-823-0.00

Source of Sample: B-39 **Depth:** 10.0

Sample Number: ST-1

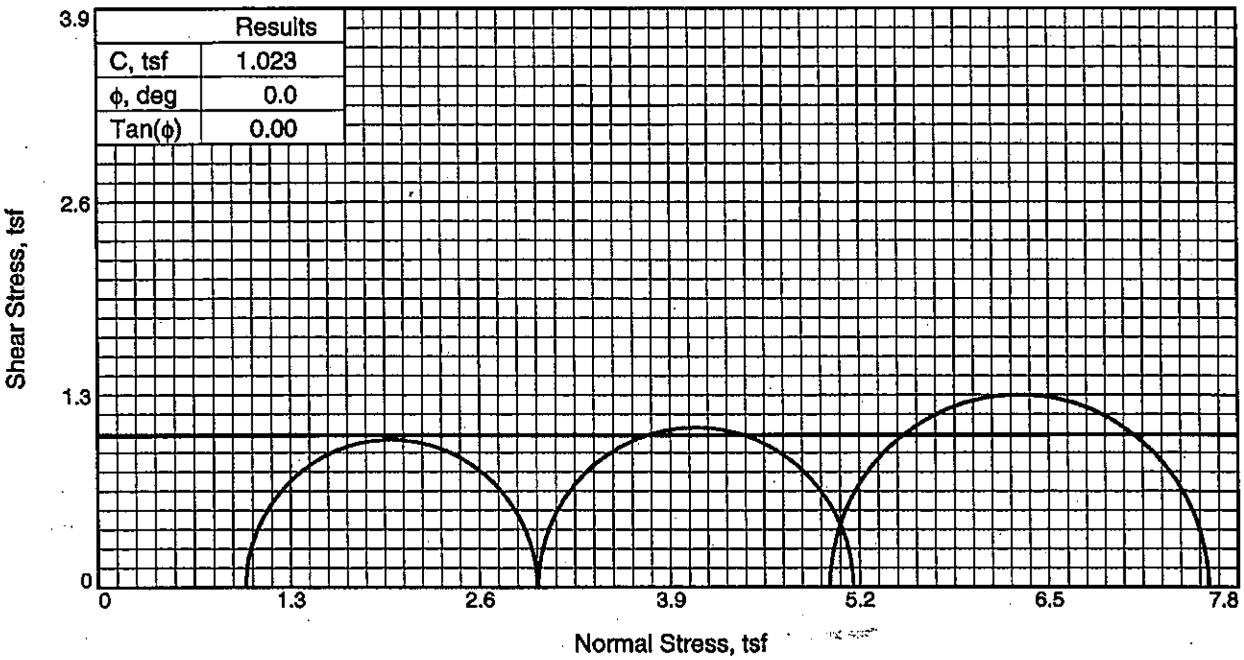
Proj. No.: 0121-3070.03 **Date:** 6/7/07

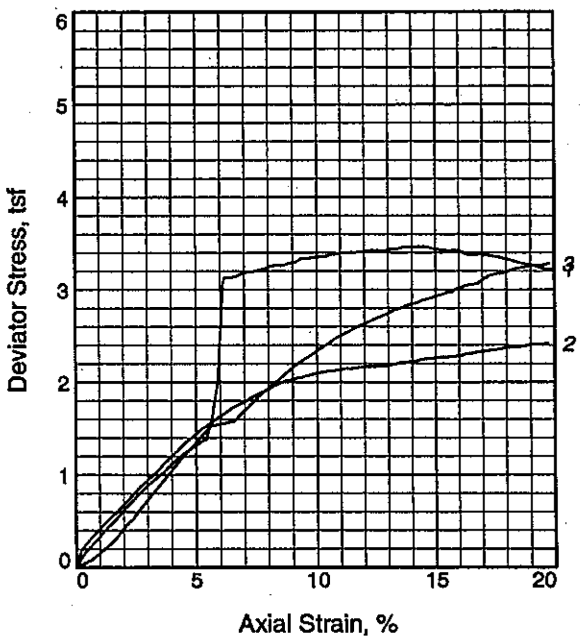
DLZ



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-39 **Depth:** 10.0 **Sample Number:** ST-1
Project No.: 0121-3070.03 **Figure** ________

DLZ, INC.





Sample No.		1	2	3
Initial	Water Content,	23.0	23.0	23.0
	Dry Density, pcf	107.8	105.3	106.9
	Saturation,	109.9	103.1	107.4
	Void Ratio	0.5641	0.6012	0.5774
	Diameter, in.	2.79	2.81	2.81
	Height, in.	5.43	5.41	5.48
At Test	Water Content,	23.7	25.6	23.0
	Dry Density, pcf	107.8	105.3	106.9
	Saturation,	113.6	115.2	107.5
	Void Ratio	0.5641	0.6012	0.5774
	Diameter, in.	2.79	2.81	2.81
	Height, in.	5.43	5.41	5.48
Strain rate, in./min.		0.06	0.06	0.06
Back Pressure, tsf		0.00	0.00	0.00
Cell Pressure, tsf		0.99	3.00	5.00
Fail. Stress, tsf		2.00	2.16	2.60
Ult. Stress, tsf		2.00	2.16	2.60
σ_1 Failure, tsf		2.99	5.16	7.60
σ_3 Failure, tsf		0.99	3.00	5.00

Type of Test:
Unconsolidated Undrained

Sample Type: 3" press tube

Description: Silty clay

LL= 25 **PL=** 20 **PI=** 5

Assumed Specific Gravity= 2.7

Remarks:

Figure ______

Client: TranSystems, Inc.

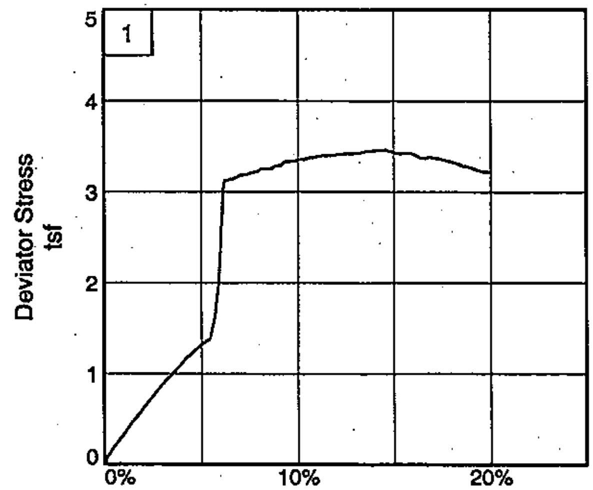
Project: SCI-823-0.00

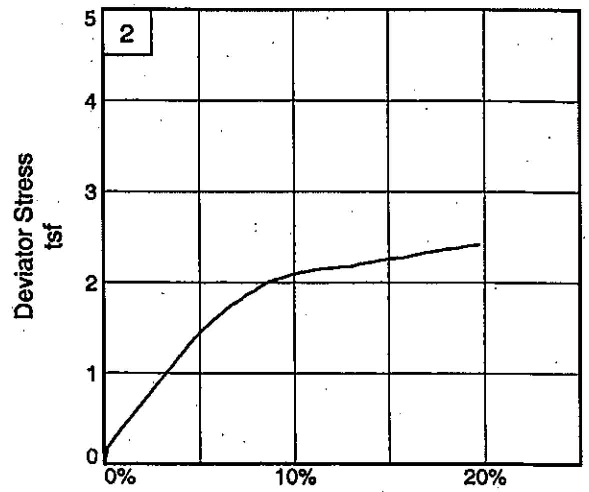
Source of Sample: B-40 **Depth:** 20.0

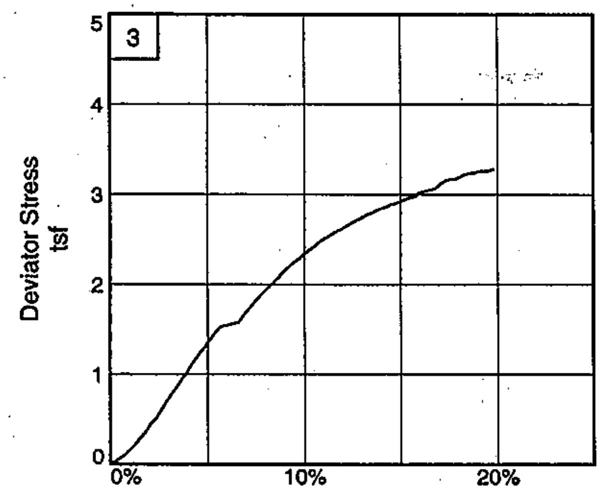
Sample Number: ST-4

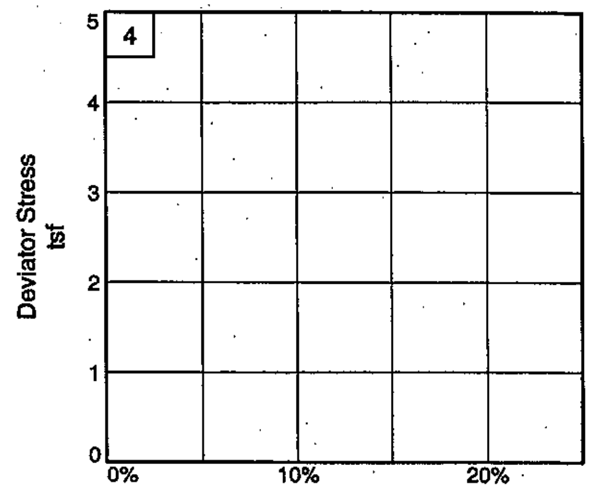
Proj. No.: 0121-3070.03 **Date:** 6/12/07

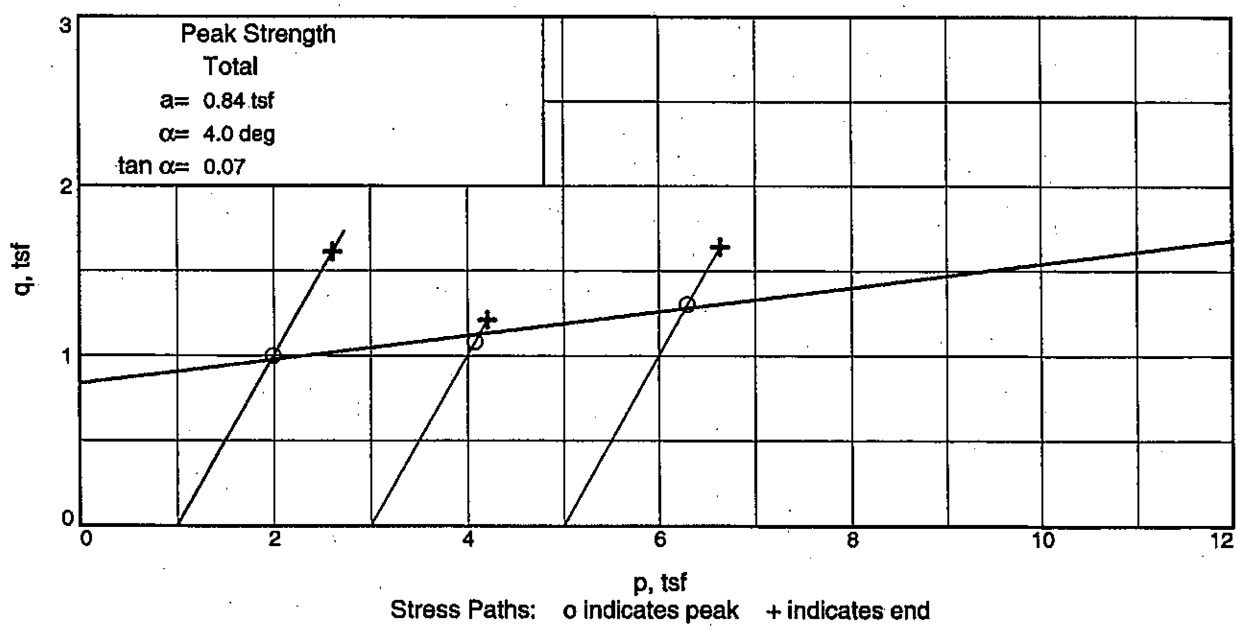
DLZ











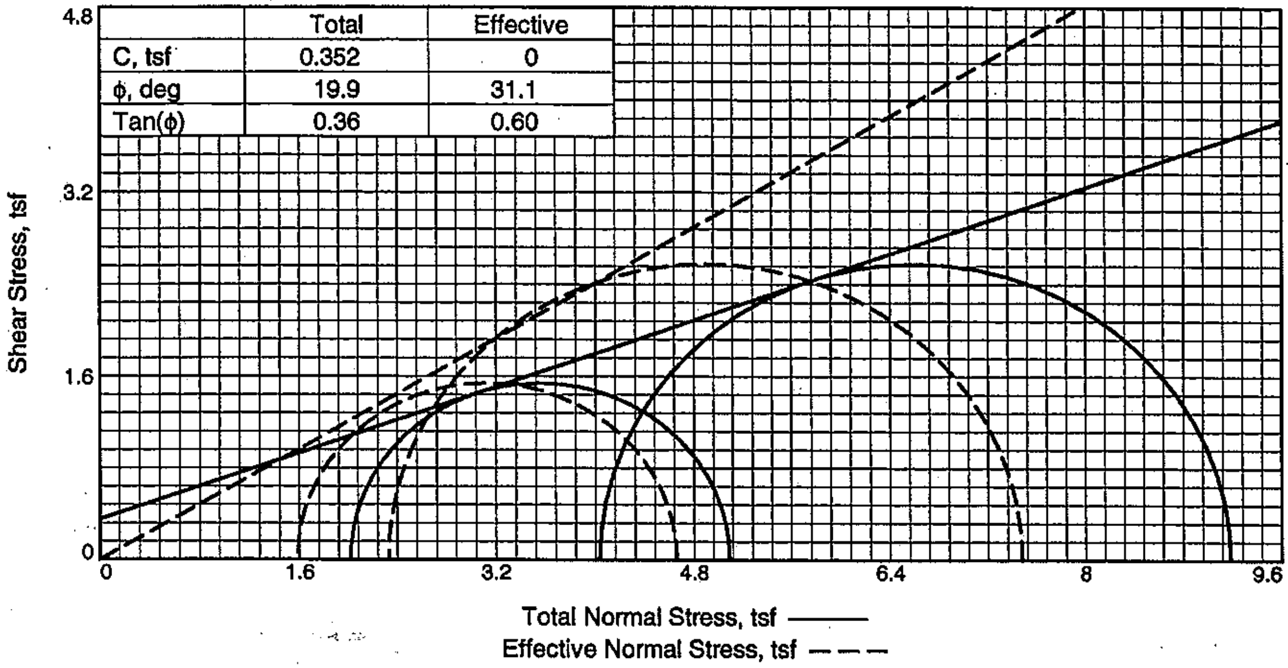
Client: TranSystems, Inc.

Project: SCI-823-0.00

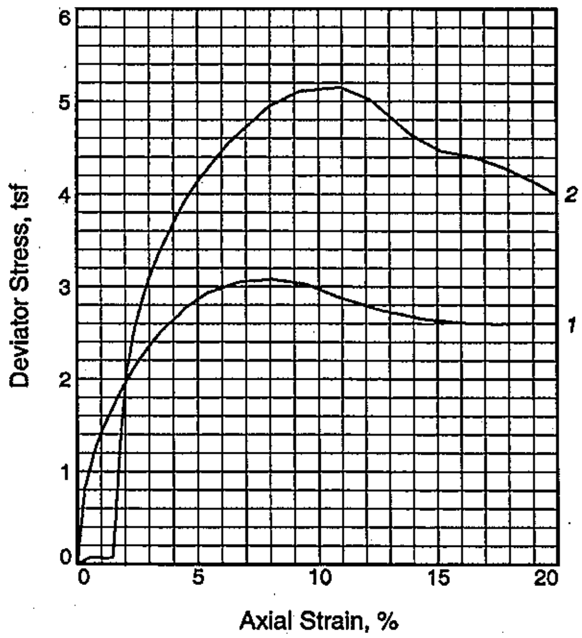
Source of Sample: B-40 **Depth:** 20.0 **Sample Number:** ST-4

Project No.: 0121-3070.03 **Figure** _______

DLZ, INC.



	Total	Effective
C, tsf	0.352	0
ϕ, deg	19.9	31.1
Tan(ϕ)	0.36	0.60



Sample No.		1	2
Initial	Water Content,	22.6	22.9
	Dry Density, pcf	101.1	99.8
	Saturation,	91.2	89.8
	Void Ratio	0.6679	0.6885
	Diameter, in.	2.85	2.87
	Height, in.	5.21	5.57
At Test	Water Content,	26.7	24.9
	Dry Density, pcf	97.9	100.8
	Saturation,	100.0	100.0
	Void Ratio	0.7220	0.6729
	Diameter, in.	2.89	2.85
	Height, in.	5.21	5.57
Strain rate, in./min.		0.01	0.01
Back Pressure, tsf		4.03	4.03
Cell Pressure, tsf		6.05	8.06
Fail. Stress, tsf		3.08	5.15
Total Pore Pr., tsf		4.47	5.74
Ult. Stress, tsf			
Total Pore Pr., tsf			
$\bar{\sigma}_1$ Failure, tsf		4.66	7.48
$\bar{\sigma}_3$ Failure, tsf		1.58	2.33

Type of Test:
CU with Pore Pressures

Sample Type: 3" Press Tube

Description:

Specific Gravity= 2.7

Remarks:

Figure ________

Client: TranSystems, Inc.

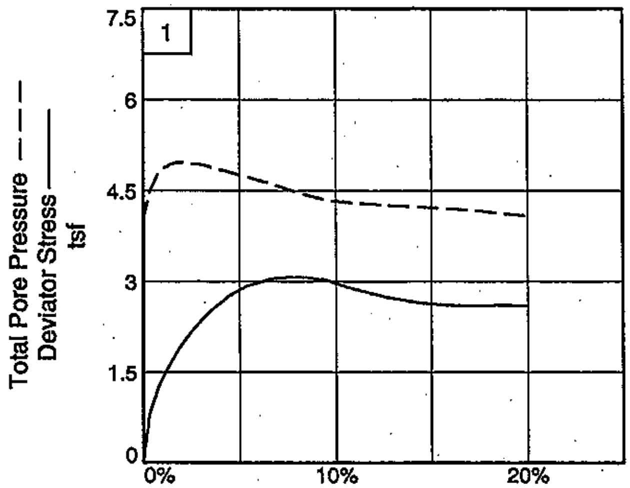
Project: SCI-823-0.00

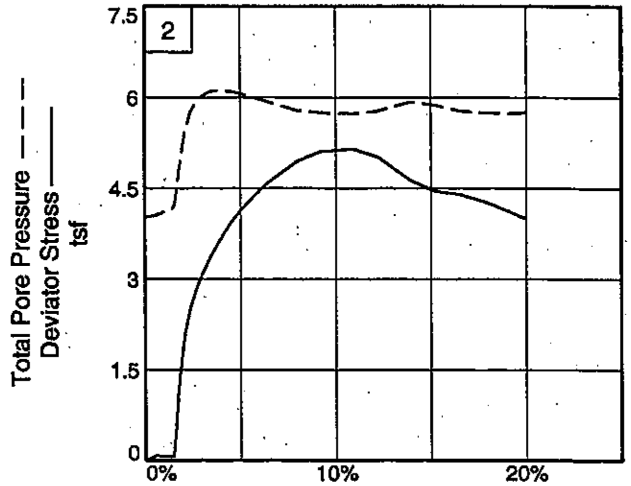
Source of Sample: TR-35A **Depth:** 5.0

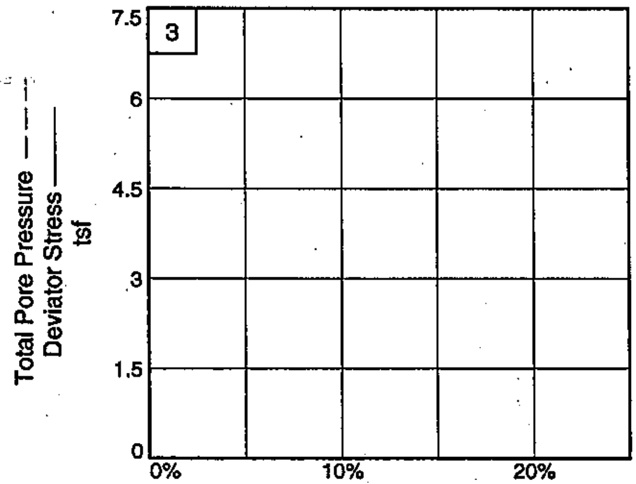
Sample Number: P-1

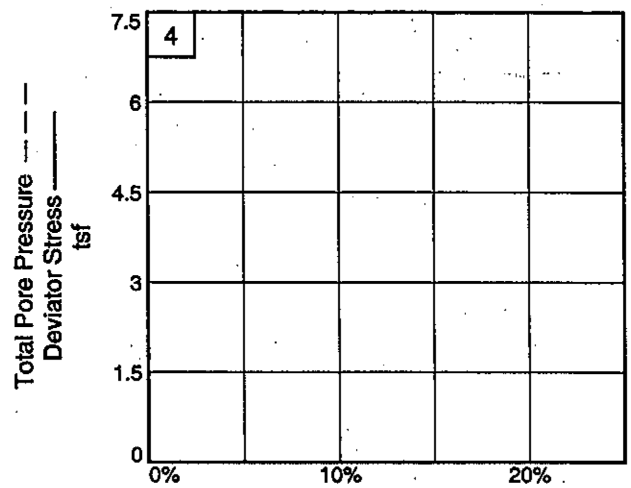
Proj. No.: 0121-3070.03 **Date:**

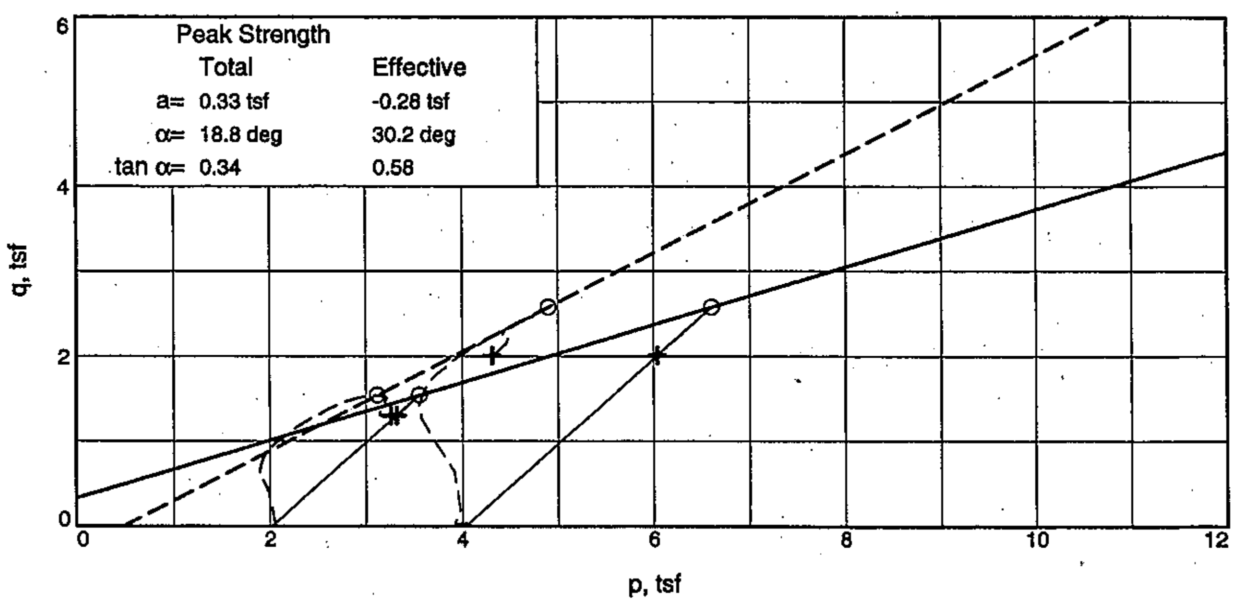
DLZ







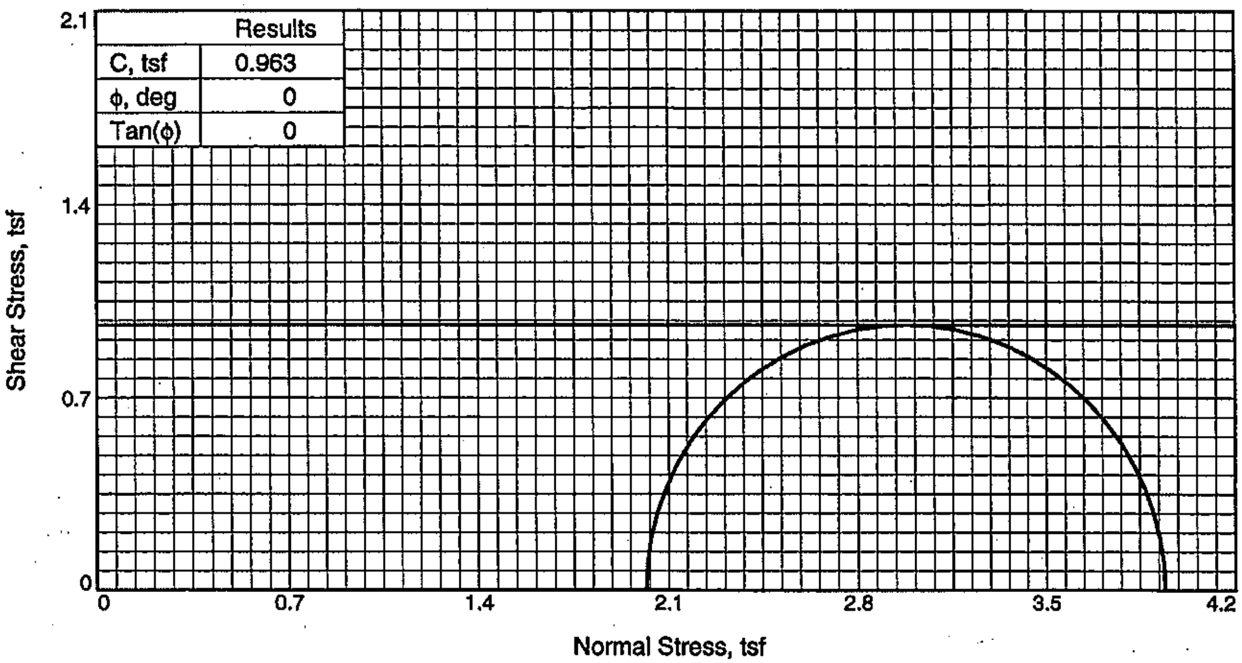


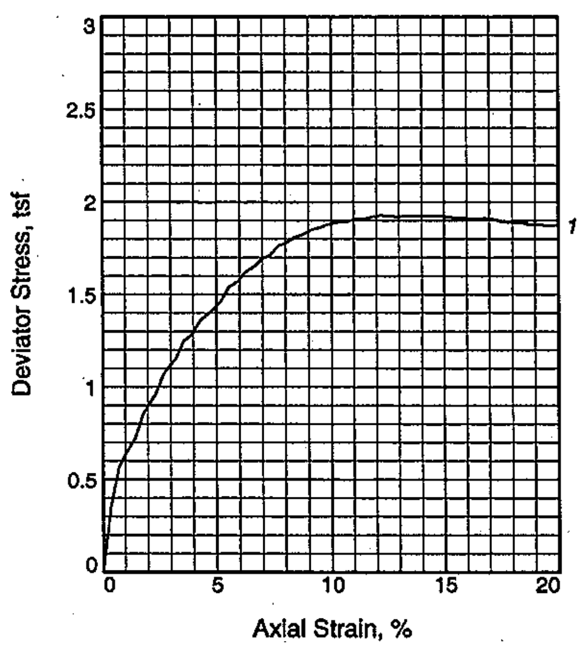


Stress Paths: Total —— Effective – – –

Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: TR-35A **Depth:** 5.0 **Sample Number:** P-1
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.





Sample No.		1
Initial	Water Content,	25.6
	Dry Density, pcf	97.5
	Saturation,	92.6
	Void Ratio	0.7611
	Diameter, in.	2.85
	Height, in.	5.59
At Test	Water Content,	27.7
	Dry Density, pcf	97.5
	Saturation,	100.0
	Void Ratio	0.7611
	Diameter, in.	2.85
	Height, in.	5.59
Strain rate, in./min.		0.06
Back Pressure, tsf		0.00
Cell Pressure, tsf		2.02
Fail. Stress, tsf		1.93
Ult. Stress, tsf		
σ_1 Failure, tsf		3.94
σ_3 Failure, tsf		2.02

Type of Test:
Unconsolidated Undrained
Sample Type: 3" Press Tube
Description: Lean clay

LL= 27 **PL=** 19 **PI=** 8
Assumed Specific Gravity= 2.75
Remarks:

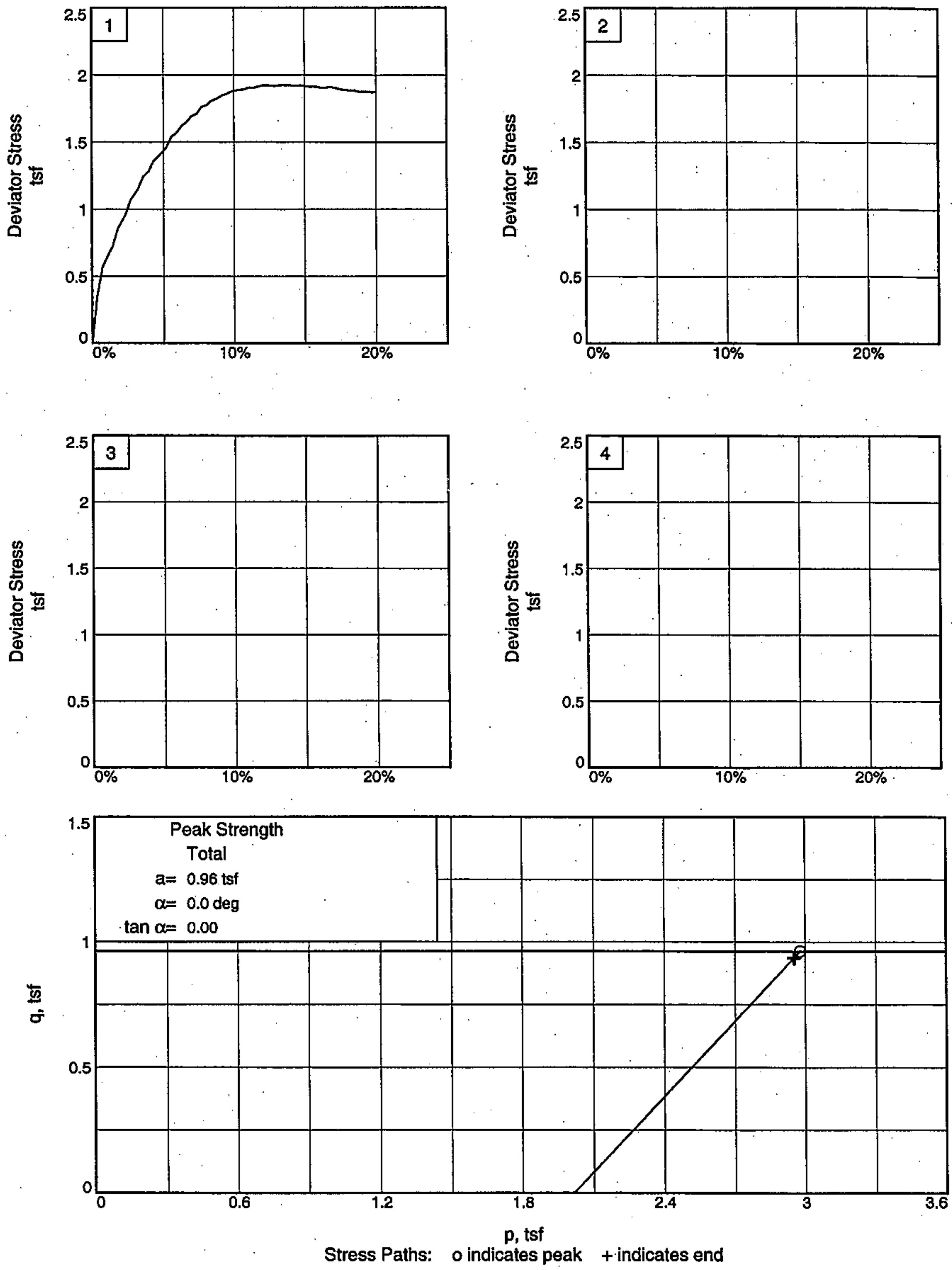
Figure ________

Client: TranSystems, Inc.

Project: SCI-823-0.00

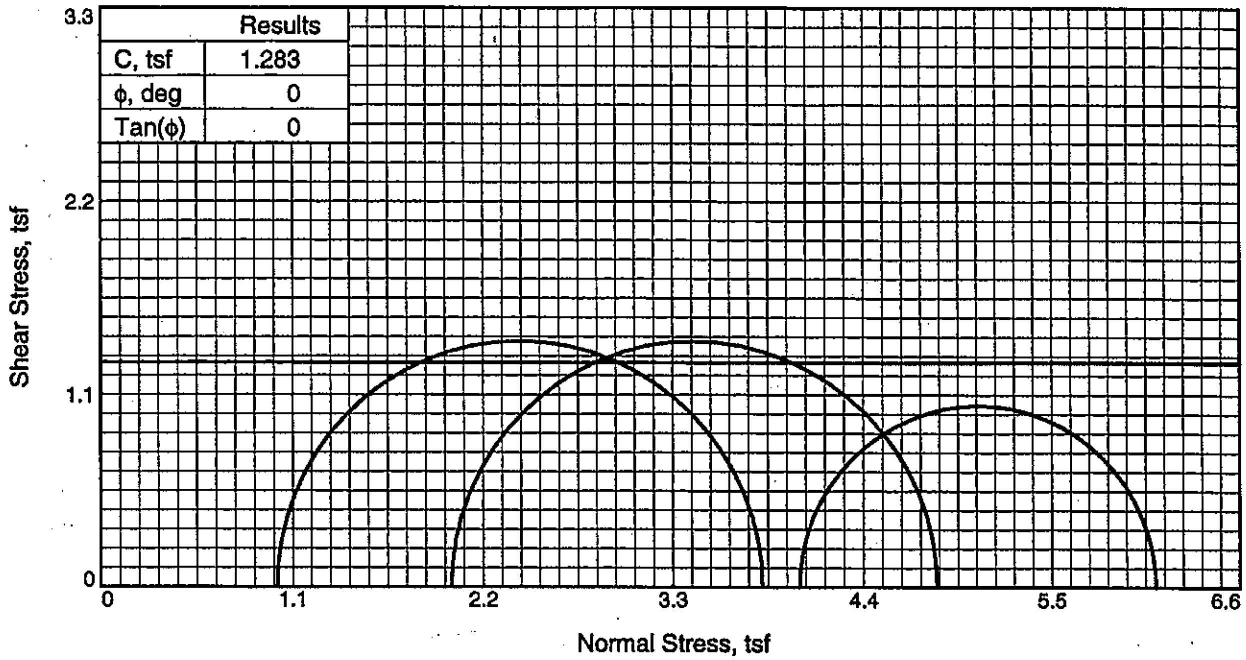
Source of Sample: TR-35A **Depth:** 5.0
Sample Number: P-1
Proj. No.: 0121-3070.03 **Date:** 2/6/06

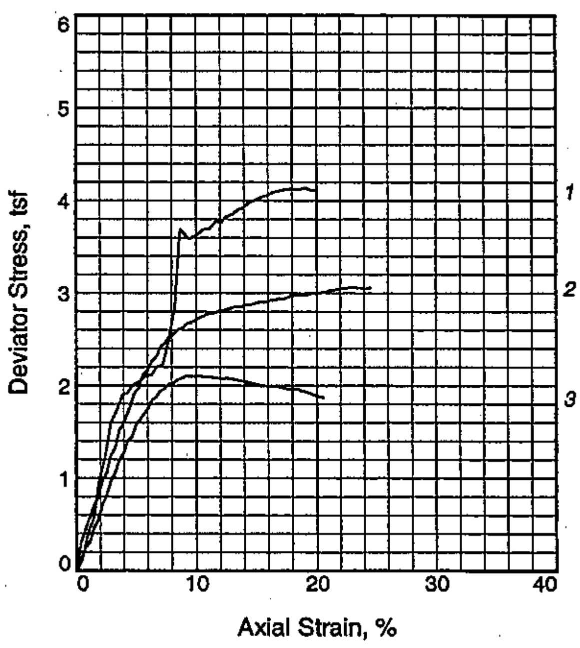
DLZ



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: TR-35A **Depth:** 5.0 **Sample Number:** P-1
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.





Sample No.		1	2	3
Initial	Water Content,	28.6	27.9	30.9
	Dry Density, pcf	97.4	96.2	92.4
	Saturation,	105.8	100.1	101.2
	Void Ratio	0.7300	0.7522	0.8243
	Diameter, in.	2.83	2.86	2.85
	Height, in.	4.14	5.51	5.55
At Test	Water Content,	27.0	27.9	30.5
	Dry Density, pcf	97.4	96.2	92.4
	Saturation,	100.0	100.0	100.0
	Void Ratio	0.7300	0.7522	0.8243
	Diameter, in.	2.83	2.86	2.85
	Height, in.	4.14	5.51	5.55
Strain rate, in./min.		0.06	0.06	0.06
Back Pressure, tsf		0.00	0.00	0.00
Cell Pressure, tsf		1.01	2.02	4.03
Fail. Stress, tsf		2.81	2.81	2.08
Ult. Stress, tsf		2.81	2.81	2.08
σ_1 Failure, tsf		3.82	4.83	6.11
σ_3 Failure, tsf		1.01	2.02	4.03

Type of Test:
Unconsolidated Undrained

Sample Type: 3" Press Tube

Description: Silt

LL= 27 **PL=** 22 **PI=** 5

Assumed Specific Gravity= 2.7

Remarks:

Figure ______

Client: TranSystems, Inc.

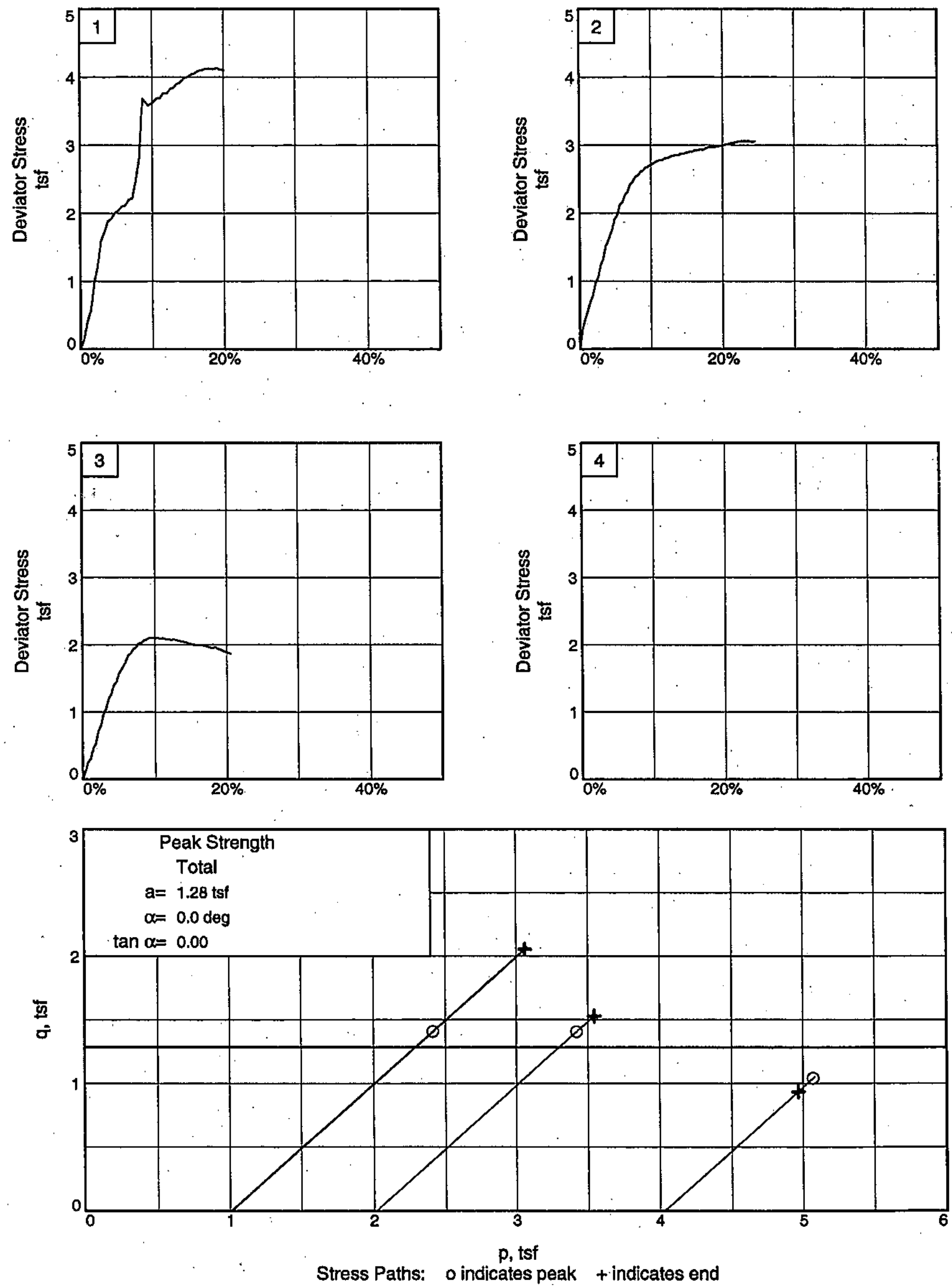
Project: SCI-823-0.00

Source of Sample: TR-35A **Depth:** 12.4

Sample Number: P-2B

Proj. No.: 0121-3070.03 **Date:** 2/6/06

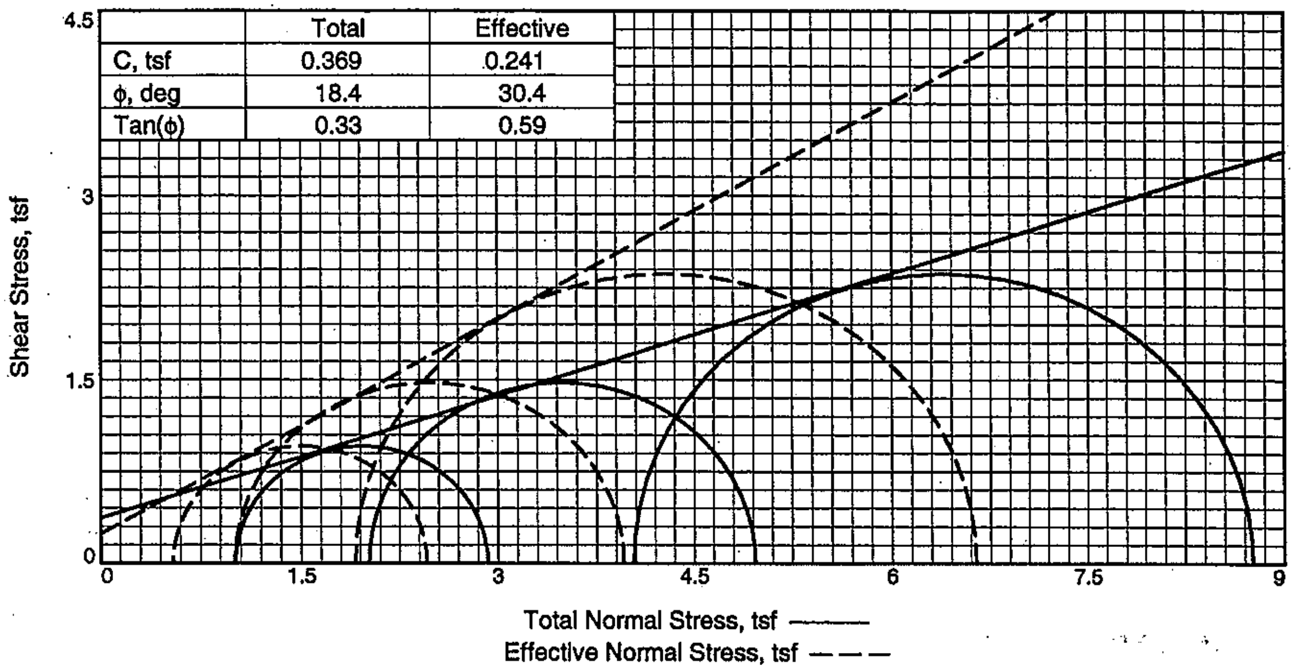
DLZ

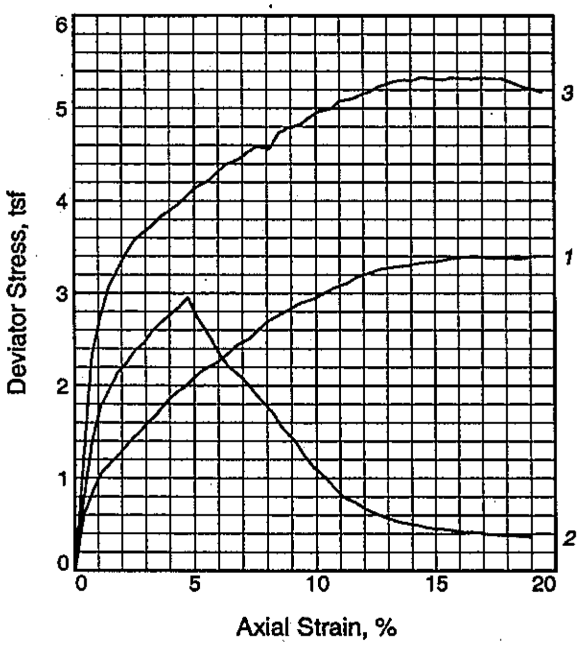


Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: TR-35A **Depth:** 12.4 **Sample Number:** P-2B
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

	Total	Effective
C, tsf	0.369	0.241
ϕ, deg	18.4	30.4
Tan(ϕ)	0.33	0.59





	Sample No.	1	2	3
Initial	Water Content,	27.6	27.6	27.6
	Dry Density, pcf	97.1	96.2	95.1
	Saturation,	98.8	96.7	94.3
	Void Ratio	0.7685	0.7852	0.8048
	Diameter, in.	2.85	2.83	2.83
	Height, in.	5.21	5.47	5.54
At Test	Water Content,	27.9	28.6	29.3
	Dry Density, pcf	97.1	96.2	95.1
	Saturation,	100.0	100.0	100.0
	Void Ratio	0.7685	0.7852	0.8048
	Diameter, in.	2.85	2.83	2.83
	Height, in.	5.21	5.47	5.54
	Strain rate, in./min.	0.06	0.06	0.06
	Back Pressure, tsf	4.03	4.03	4.03
	Cell Pressure, tsf	5.04	6.05	8.06
	Fail. Stress, tsf	1.91	2.95	4.73
	Total Pore Pr., tsf	4.50	5.04	6.15
	Ult. Stress, tsf	1.91	2.95	4.73
	Total Pore Pr., tsf	4.50	5.04	6.15
	$\bar{\sigma}_1$ Failure, tsf	2.46	3.95	6.64
	$\bar{\sigma}_3$ Failure, tsf	0.54	1.00	1.91

Type of Test:
CU with Pore Pressures

Sample Type: 3" Press Tube

Description:

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

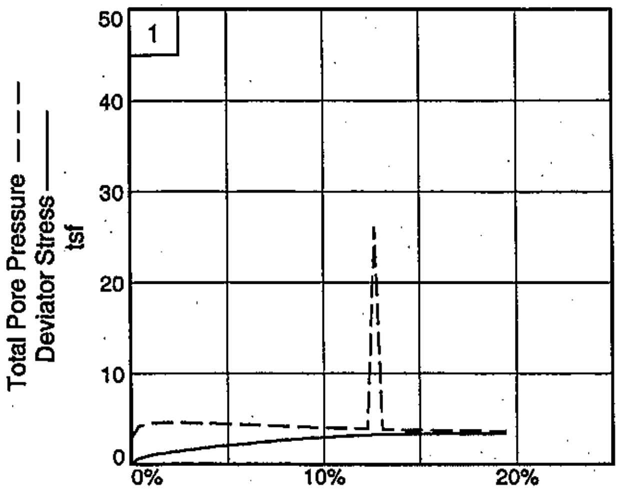
Project: SCI-823-0.00

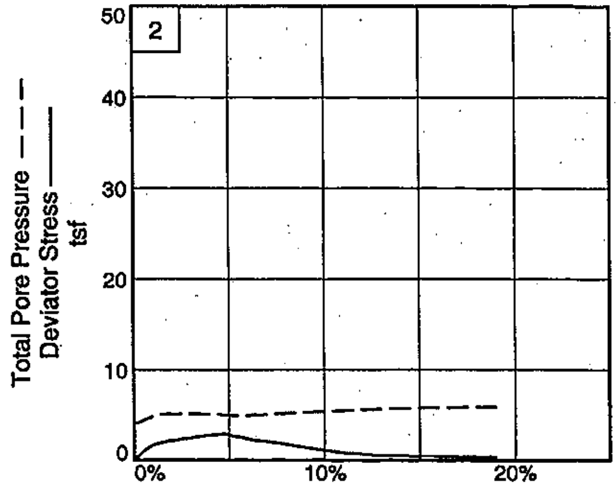
Source of Sample: TR-35A **Depth:** 27.0

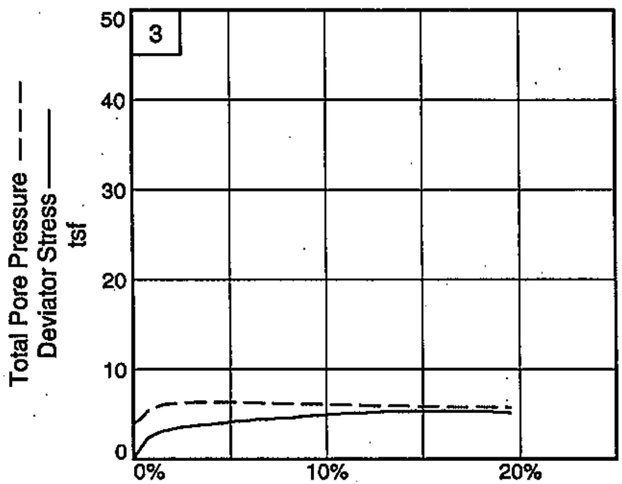
Sample Number: P-3

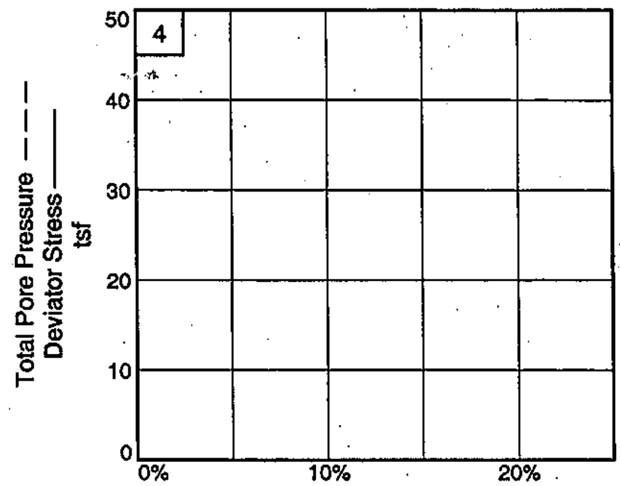
Proj. No.: 0121-3070.03 **Date:**

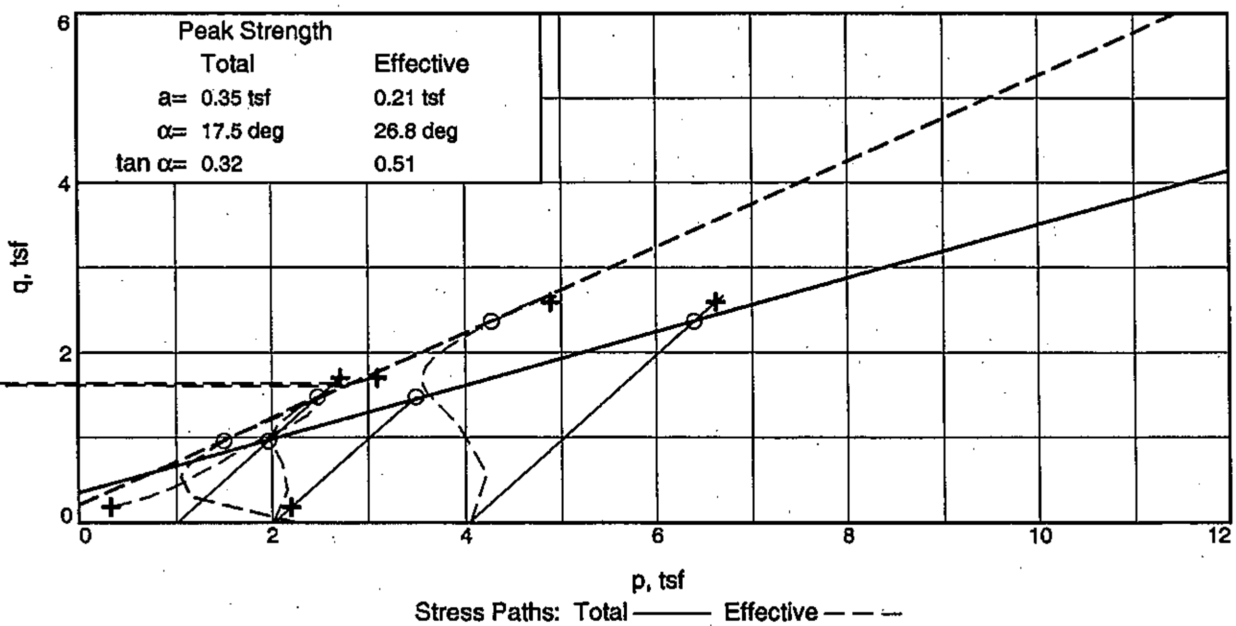
DLZ







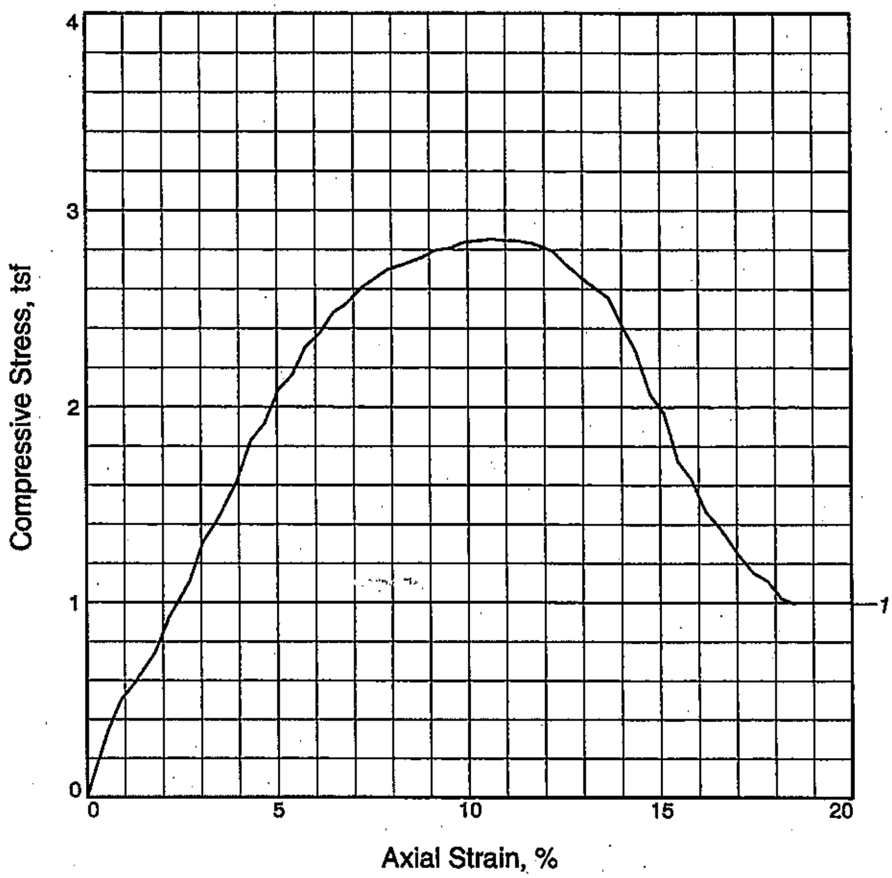




Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: TR-35A **Depth:** 27.0 **Sample Number:** P-3
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.854			
Undrained shear strength, tsf	1.427			
Failure strain,	10.6			
Strain rate, in./min.	0.06			
Water content, %	25.5			
Wet density, pcf	118.2			
Dry density, pcf	94.2			
Saturation, %	87.3			
Void ratio	0.7894			
Specimen diameter, in.	2.85			
Specimen height, in.	5.56			
Height/diameter ratio	1.95			

Description: Lean clay

LL = 44	**PL** = 26	**PI** = 18	**Assumed GS**= 2.7	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 2/6/06

Remarks:

Figure ________

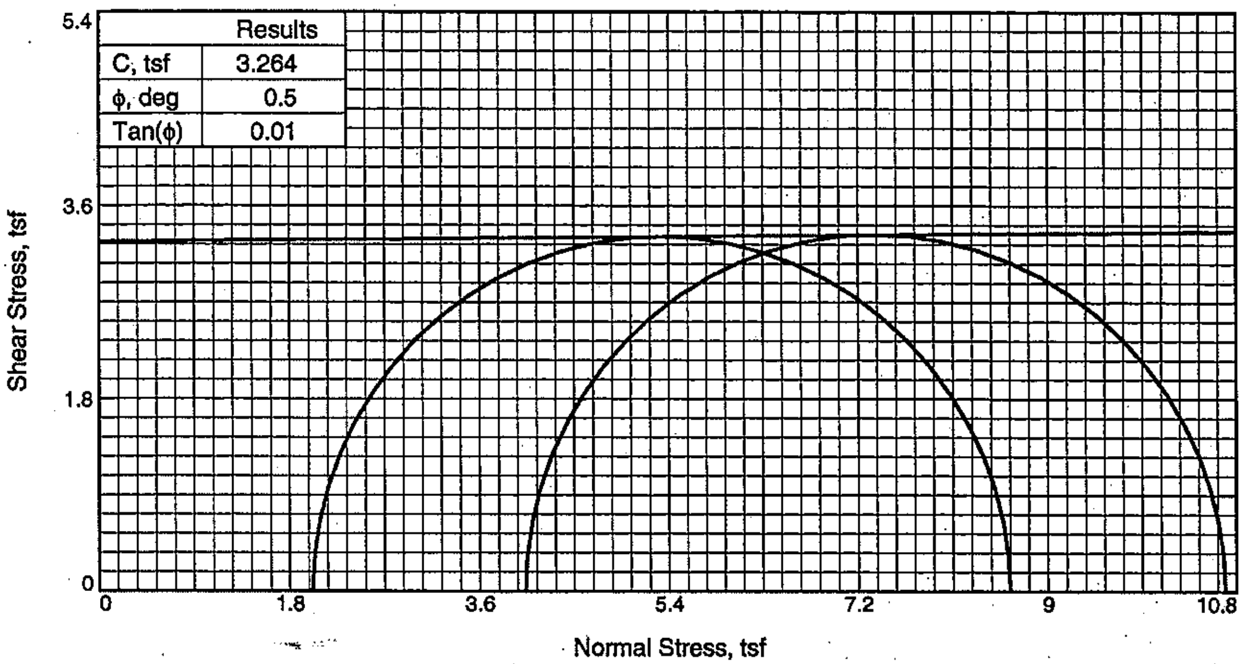
Client: TranSystems, Inc.

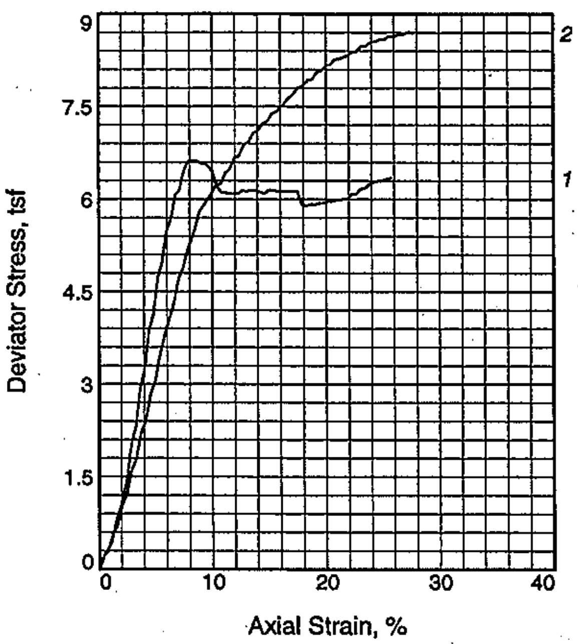
Project: SCI-823-0.00

Source of Sample: TR-35A **Depth:** 66.0

Sample Number: P-4A

DLZ





Sample No.		1	2
Initial	Water Content,	18.8	18.8
	Dry Density, pcf	112.6	113.5
	Saturation,	101.9	104.4
	Void Ratio	0.4971	0.4855
	Diameter, in.	2.83	2.80
	Height, in.	5.21	4.85
At Test	Water Content,	18.4	18.0
	Dry Density, pcf	112.6	113.5
	Saturation,	100.0	100.0
	Void Ratio	0.4971	0.4855
	Diameter, in.	2.83	2.80
	Height, in.	5.21	4.85
Strain rate, in./min.		0.06	0.06
Back Pressure, tsf		0.0	0.0
Cell Pressure, tsf		2.0	4.0
Fail. Stress, tsf		6.6	6.7
Ult. Stress, tsf		6.6	6.7
σ_1 Failure, tsf		8.6	10.7
σ_3 Failure, tsf		2.0	4.0

Type of Test:
Unconsolidated Undrained

Sample Type: 3" Press Tube

Description: Silt with sand

LL= NP **PI=** NP

Assumed Specific Gravity= 2.7

Remarks: Sample S3 - 66.0' - 66.9'

Figure ________

Client: TranSystems, Inc.

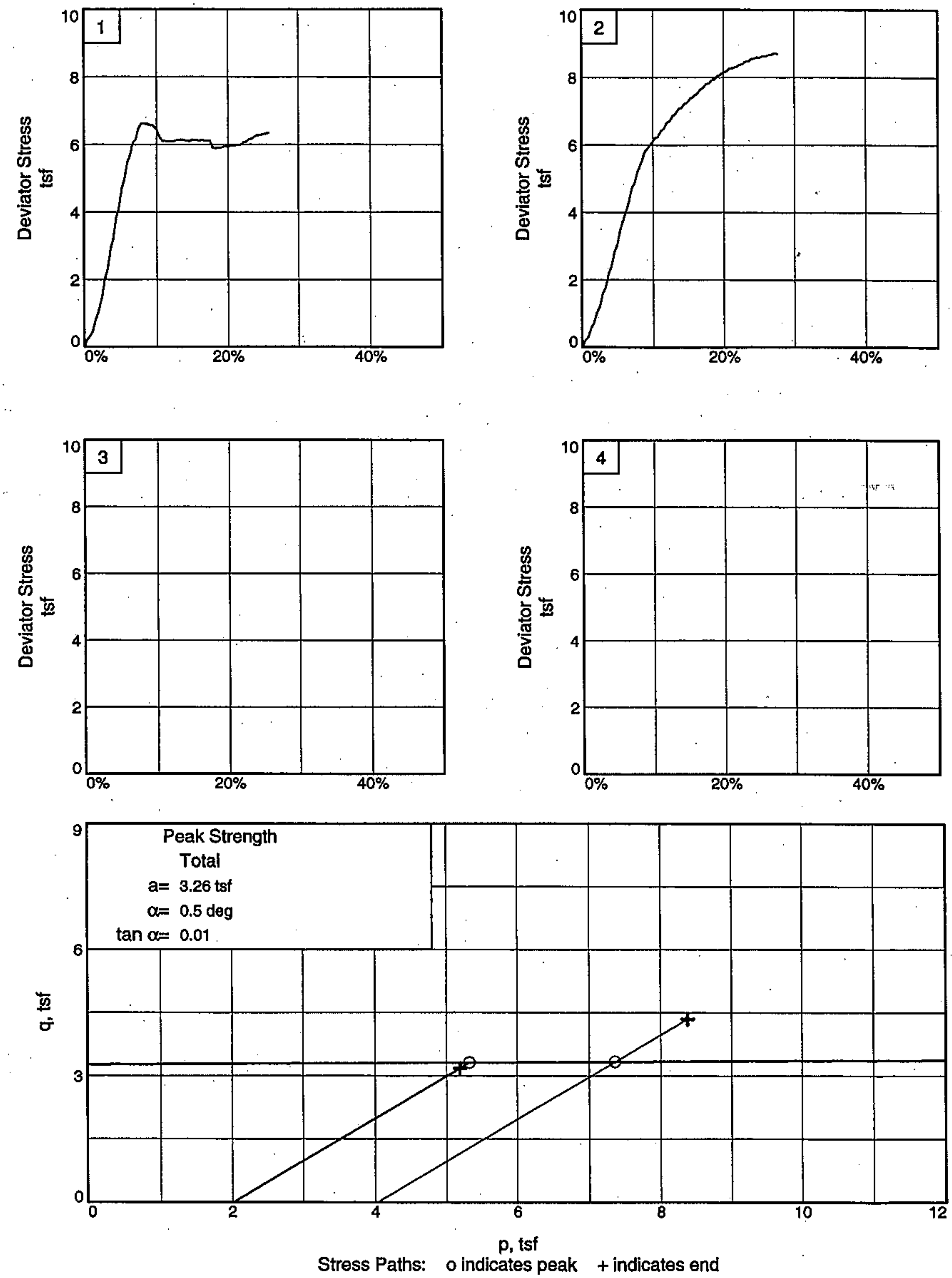
Project: SCI-823-0.00

Source of Sample: TR-35A **Depth:** 66.9

Sample Number: P-4B

Proj. No.: 0121-3070.03 **Date:** 2/6/06

DLZ



Client: TranSystems, Inc.

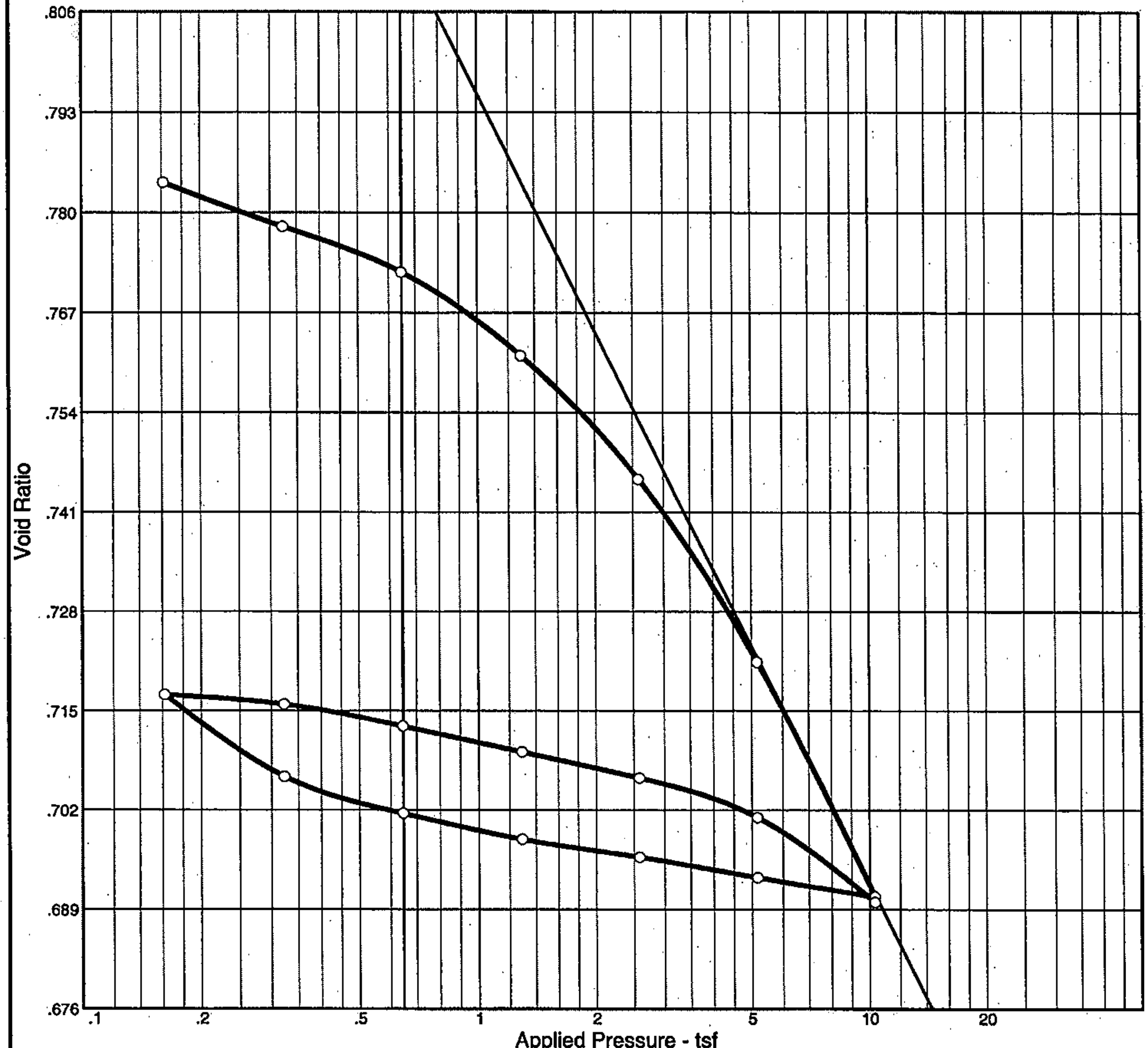
Project: SCI-823-0.00

Source of Sample: TR-35A **Depth:** 66.9 **Sample Number:** P-4B

Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
94.9 %	27.0 %	96.5	27	5	2.76	ML	A-4(4)	0.786

MATERIAL DESCRIPTION

Silt

Project No. 0121- **Client:** TranSystems, Inc.

Project: SCI-823-0.00

Source: TR-35A **Sample No.:** P-2B **Elev./Depth:** 12.4

DLZ

Remarks:

Figure

CONSOLIDATION TEST REPORT

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
91.8 %	30.9 %	89.4	NP	NP	2.76	ML	A-4(0)	0.928

MATERIAL DESCRIPTION

Silt

Project No. 0121- **Client:** TranSystems, Inc.

Project: SCI-823-0.00

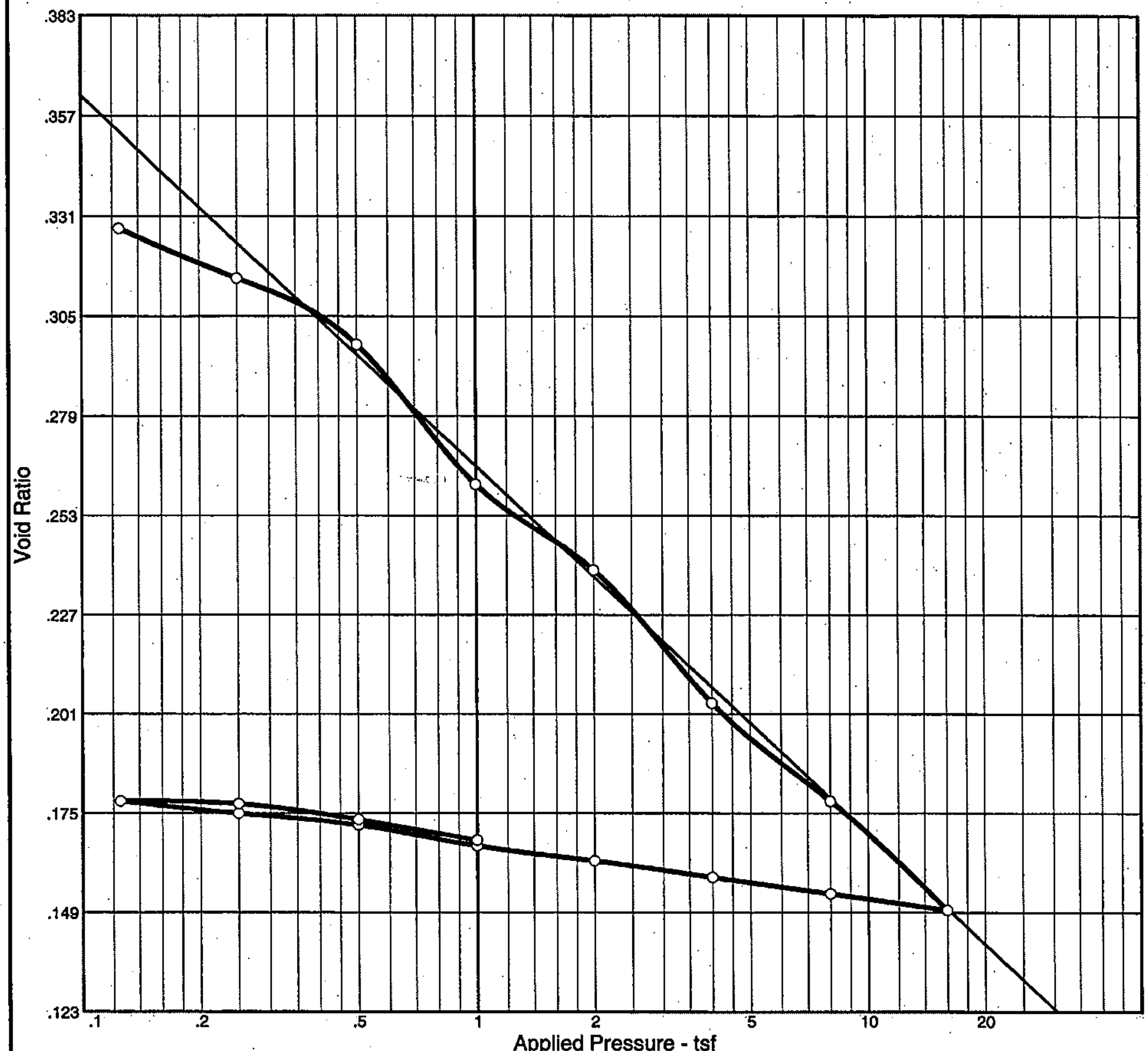
Source: TR-35A **Sample No.:** P-3 **Elev./Depth:** 27.0

DLZ

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
108.9 %	13.8 %	127.4	NP	NP	2.75	ML	A-4(0)	0.347

MATERIAL DESCRIPTION

Silt with sand

Project No. 0121- **Client:** TranSystems, Inc.

Project: SCI-823-0.00

Source: TR-35A **Sample No.:** P-4B **Elev./Depth:** 66.9

DLZ

Remarks:

Figure

Unconfined Compression of Rock Core Specimens
(ASTM D-2938)

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

Project Name: SCI-823-0.00 | **Date: 8/3/07**

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-39	2	39.9'-40.3'	1.985	1.987	1.984	1.986	4.424	4.424	4.411	4.420	2.226	0.007918	516.70	143.9	32,550	**10,509**
			1.984	1.991	1.984											
B-39	3	44.7'-45.3'	1.986	1.985	1.987	1.986	4.440	4.448	4.453	4.447	2.240	0.0079657	520.56	144.1	31,060	**10,030**
			1.985	1.985	1.986											
B-40	2	54.0'-54.6'	1.982	1.979	1.982	1.982	4.630	4.607	4.604	4.614	2.328	0.0082323	560.51	150.1	36,660	**11,884**
			1.986	1.983	1.979											
B-40	3	63.0'-63.4'	1.985	1.986	1.987	1.986	4.256	4.240	4.258	4.251	2.141	0.0076177	516.33	149.4	33,450	**10,798**
			1.987	1.985	1.986											
B-41	1	43.9'-44.3'	1.981	1.974	1.975	1.979	4.630	4.624	4.640	4.631	2.341	0.0082361	567.69	152.0	30,160	**9,810**
			1.982	1.984	1.975											
B-41	2	53.5'-53.9'	1.989	1.986	1.985	1.986	4.441	4.434	4.426	4.434	2.232	0.0079471	539.89	149.8	38,170	**12,318**
			1.985	1.988	1.985											
B-42	2	10.7'-11.2'	1.974	1.971	1.965	1.970	4.576	4.597	4.582	4.585	2.328	0.0080797	541.20	147.7	8,030	**2,636**
			1.969	1.973	1.965											

DLZ

Engineers * Architects * Scientists

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

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

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

Project Name: SCI-823-0.00 | **Date: 8/3/07**

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-42	4	18.6'-19.0'	1.969	1.972	1.965	1.969	4.060	4.091	4.062	4.071	2.068	0.0071691	486.55	149.6	4,840	**1,590**
			1.962	1.975	1.970											
B-42	6	30.6'-31.0'	1.983	1.983	1.984	1.984	4.531	4.522	4.527	4.527	2.282	0.0080907	512.36	139.6	34,700	**11,230**
			1.983	1.983	1.985											
B-42	7	36.0'-36.5'	1.973	1.971	1.968	1.971	4.670	4.674	4.665	4.670	2.369	0.0082414	527.66	141.2	33,250	**10,898**
			1.971	1.972	1.971											
B-43	2	13.4'-13.9'	1.967	1.966	1.967	1.968	4.530	4.520	4.530	4.527	2.301	0.007962	500.70	138.6	23,980	**7,886**
			1.970	1.969	1.967											
B-43	3	18.1'-18.7'	1.971	1.972	1.973	1.972	4.650	4.647	4.639	4.645	2.356	0.0082068	565.57	151.9	34,220	**11,204**
			1.976	1.968	1.972											
B-43	4	25.0'-25.5'	1.974	1.970	1.973	1.973	4.705	4.710	4.708	4.708	2.386	0.0083254	565.59	149.8	32,330	**10,575**
			1.972	1.978	1.971											
B-43	5	31.4'-31.8'	1.972	1.972	1.973	1.971	4.381	4.381	4.383	4.382	2.223	0.0077357	532.26	151.7	38,370	**12,571**
			1.971	1.972	1.968											

DLZ

Engineers * Architects * Scientists

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

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

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

Project Name: SCI-823-0.00 — **Date:** 8/3/07

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-44	2	39.5'-40.0'	1.973	1.965	1.969	1.969	4.632	4.615	4.613	4.620	2.346	0.0081372	498.46	135.0	31,230	**10,256**
			1.967	1.975	1.965											
B-44	3	34.5'-35.0'	1.963	1.965	1.964	1.964	4.560	4.521	4.538	4.540	2.311	0.0079552	521.25	144.5	29,430	**9,714**
			1.966	1.960	1.966											

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Engineers * Architects * Scientists

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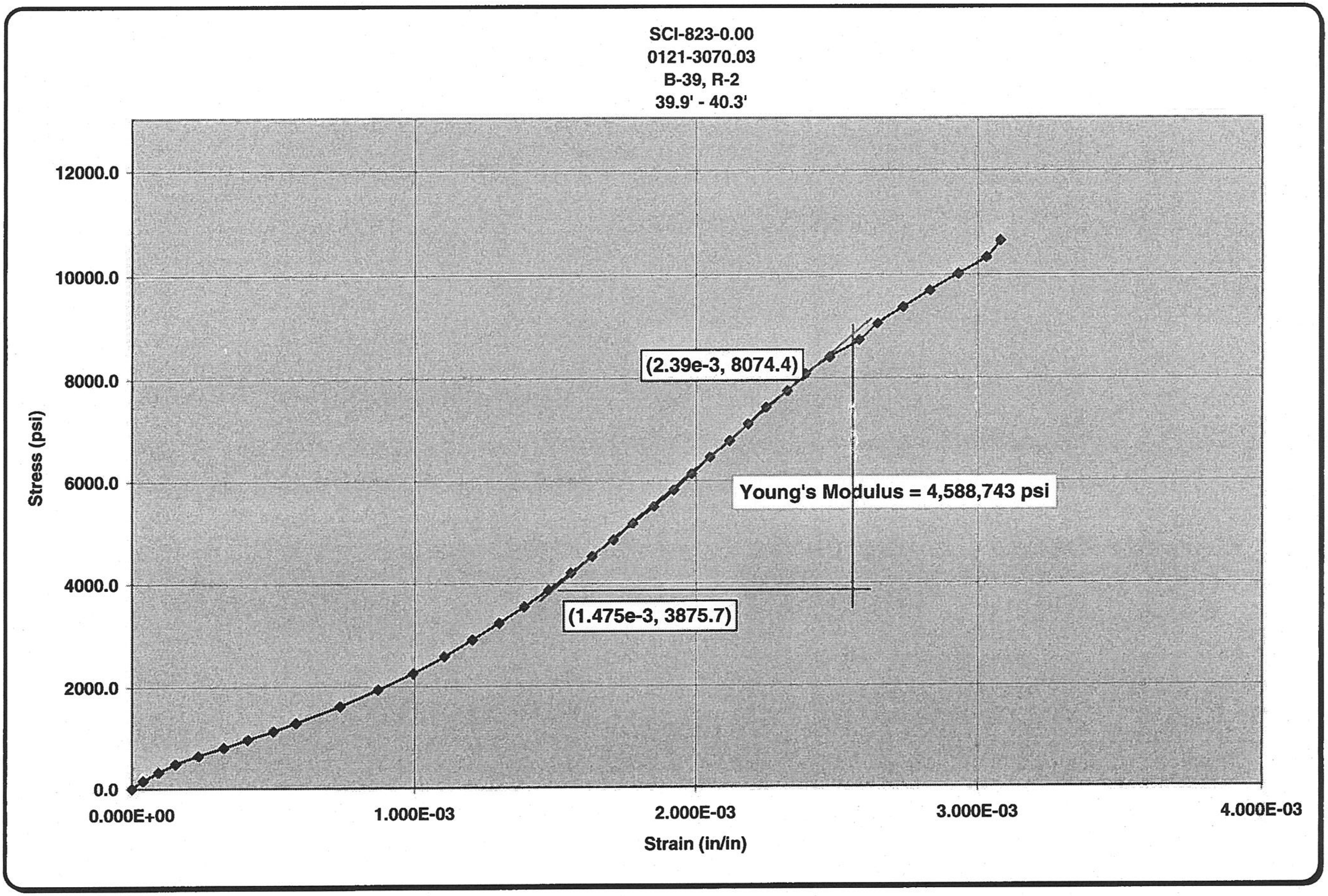
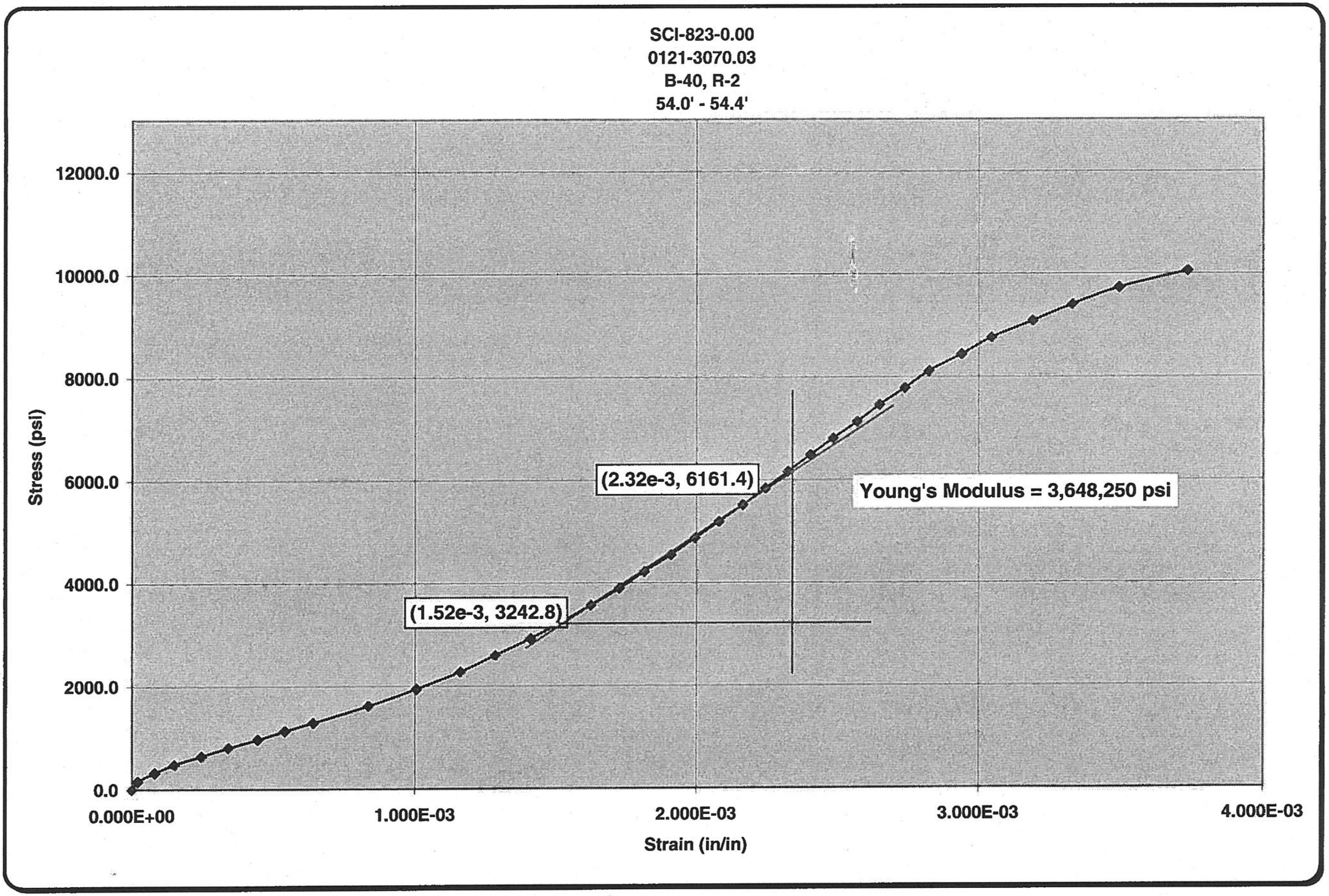
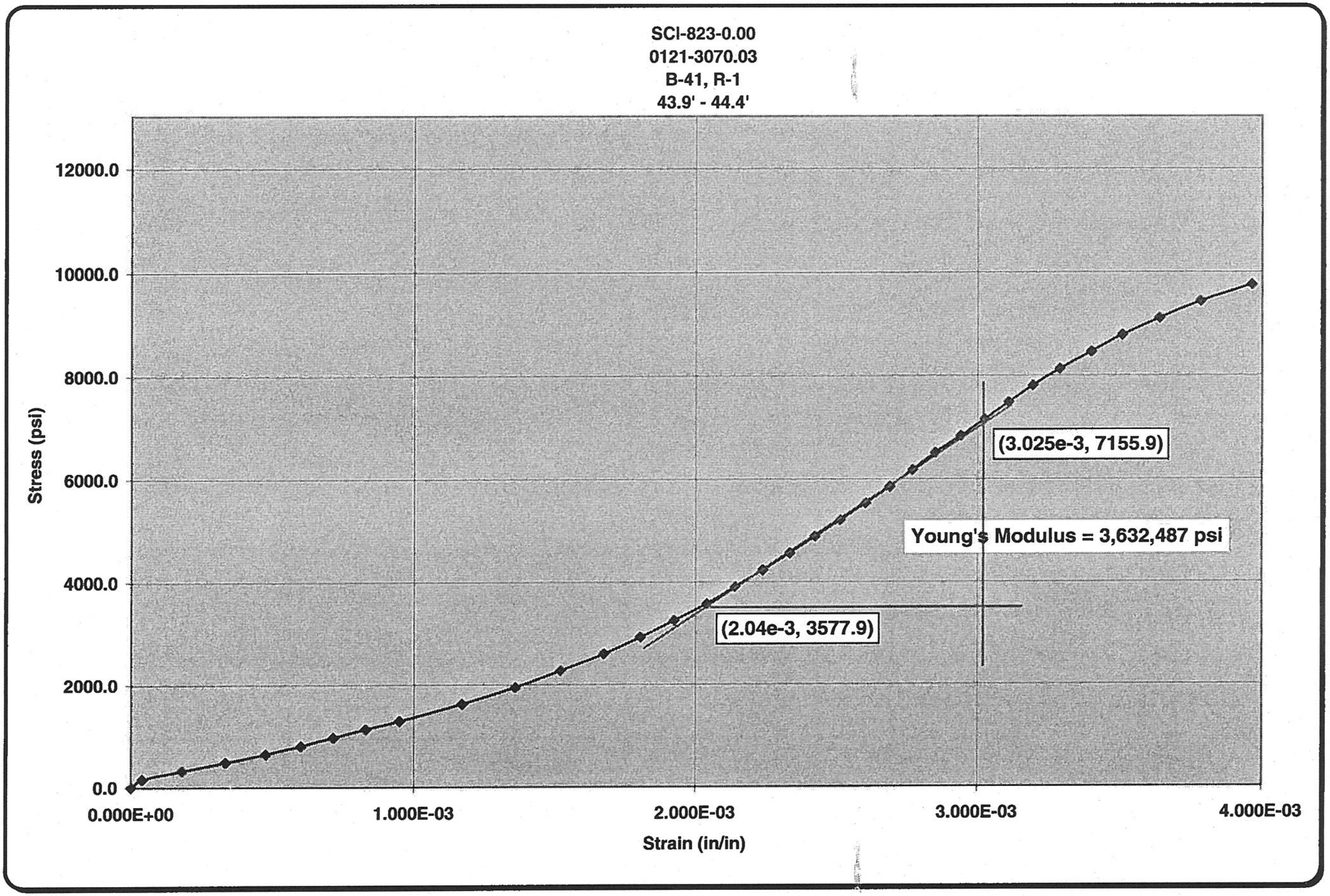


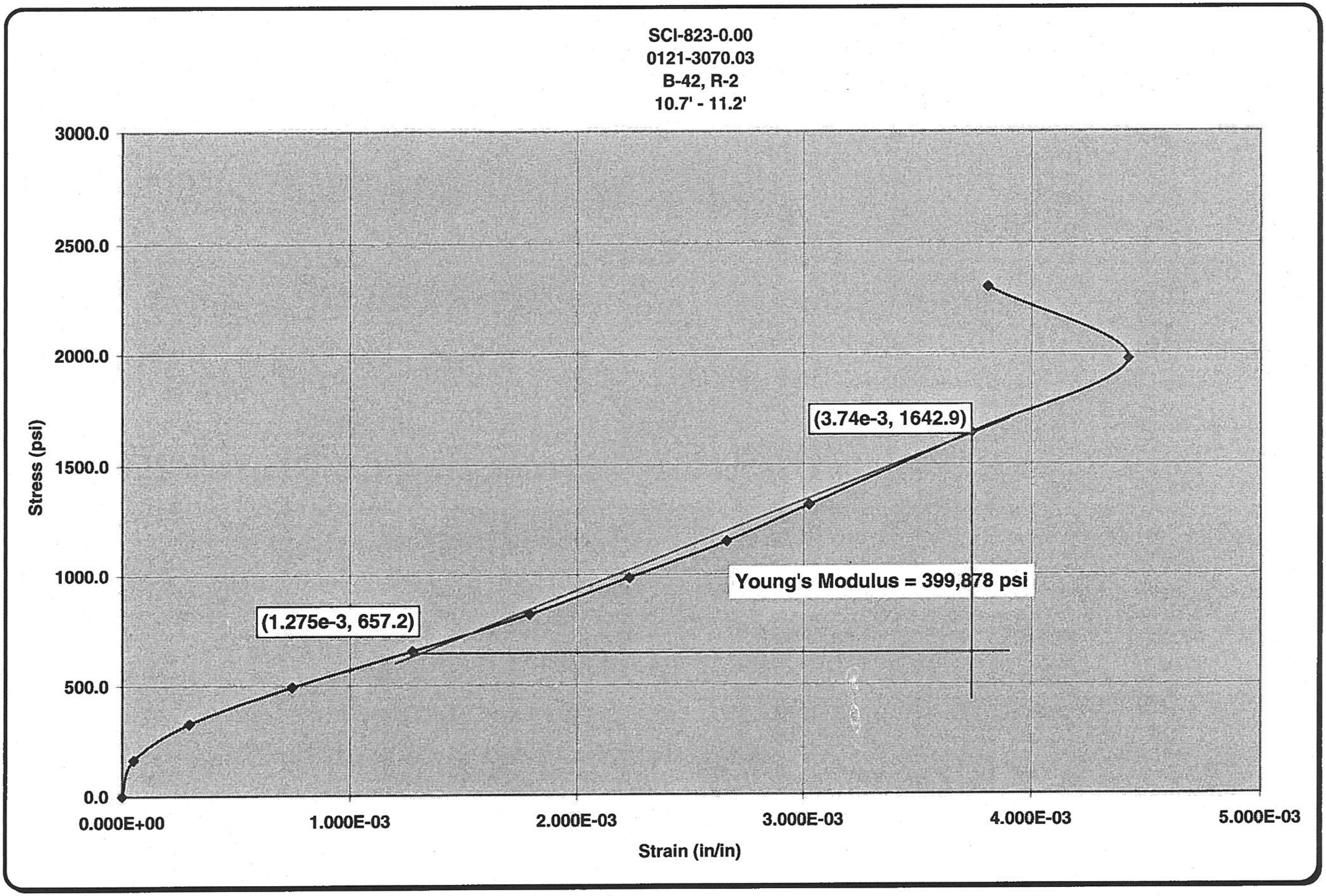
B39 r2 Chart 1



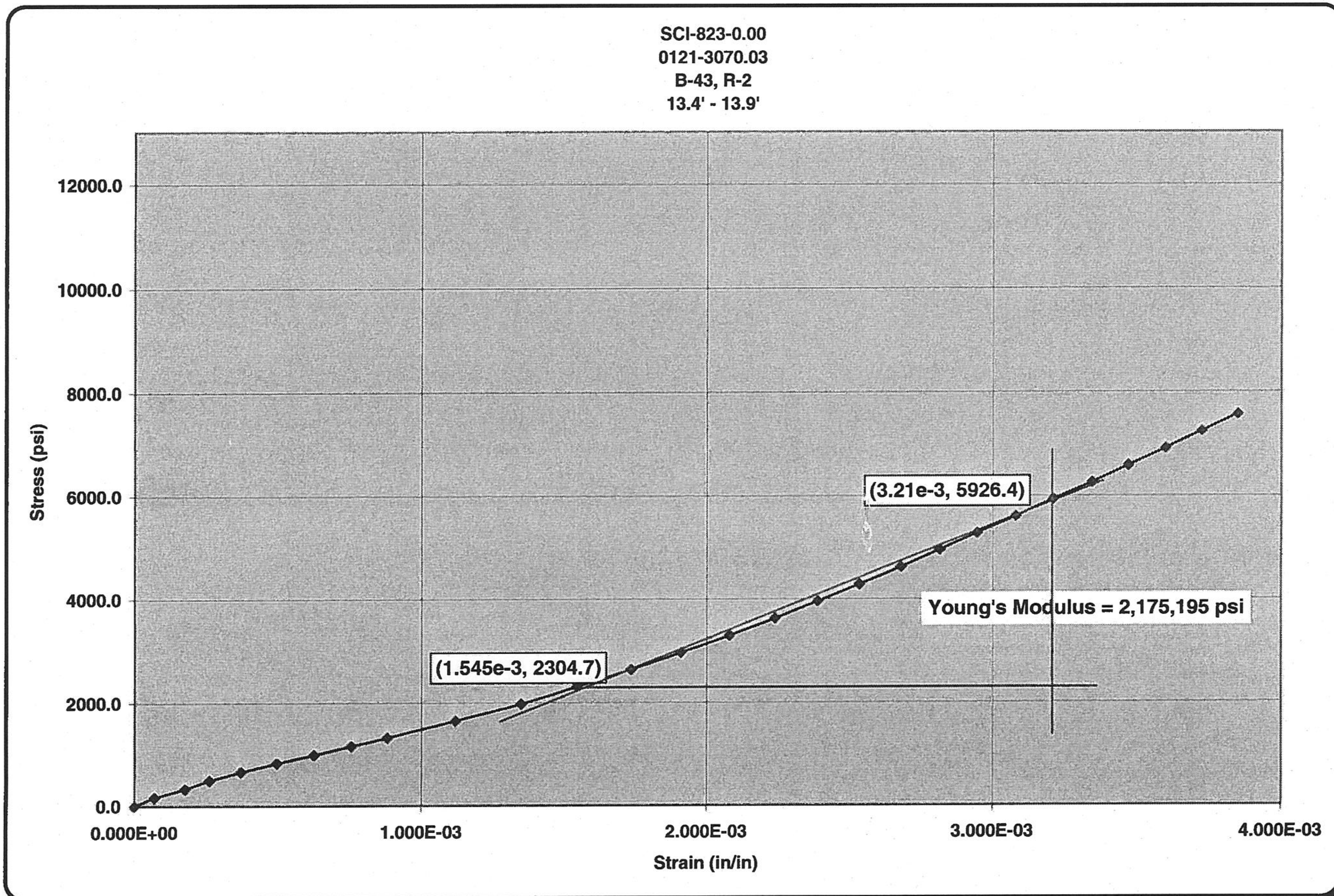
b40 r2 Chart 1



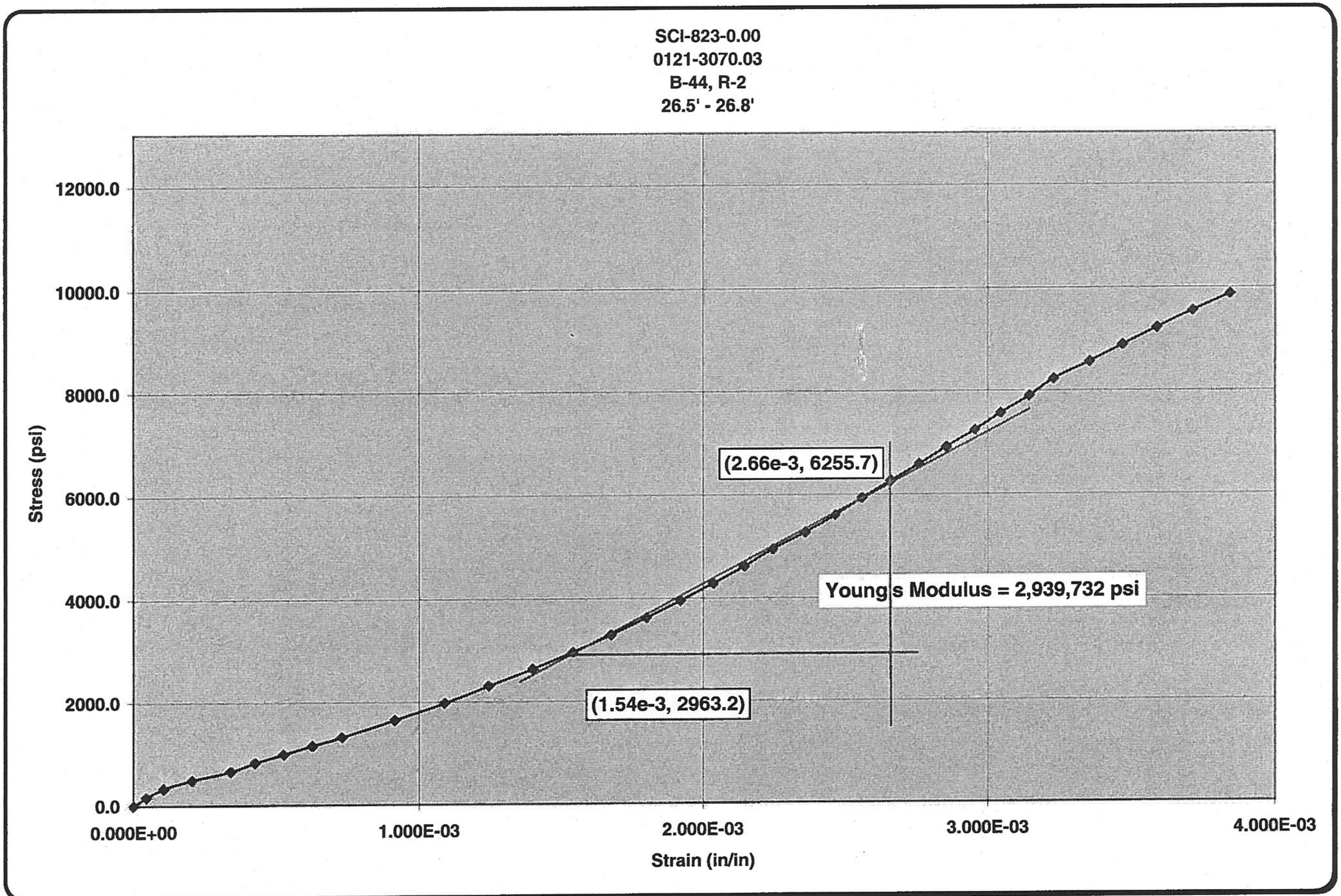
B41 R1 Chart 1



B42 R2 Chart 1

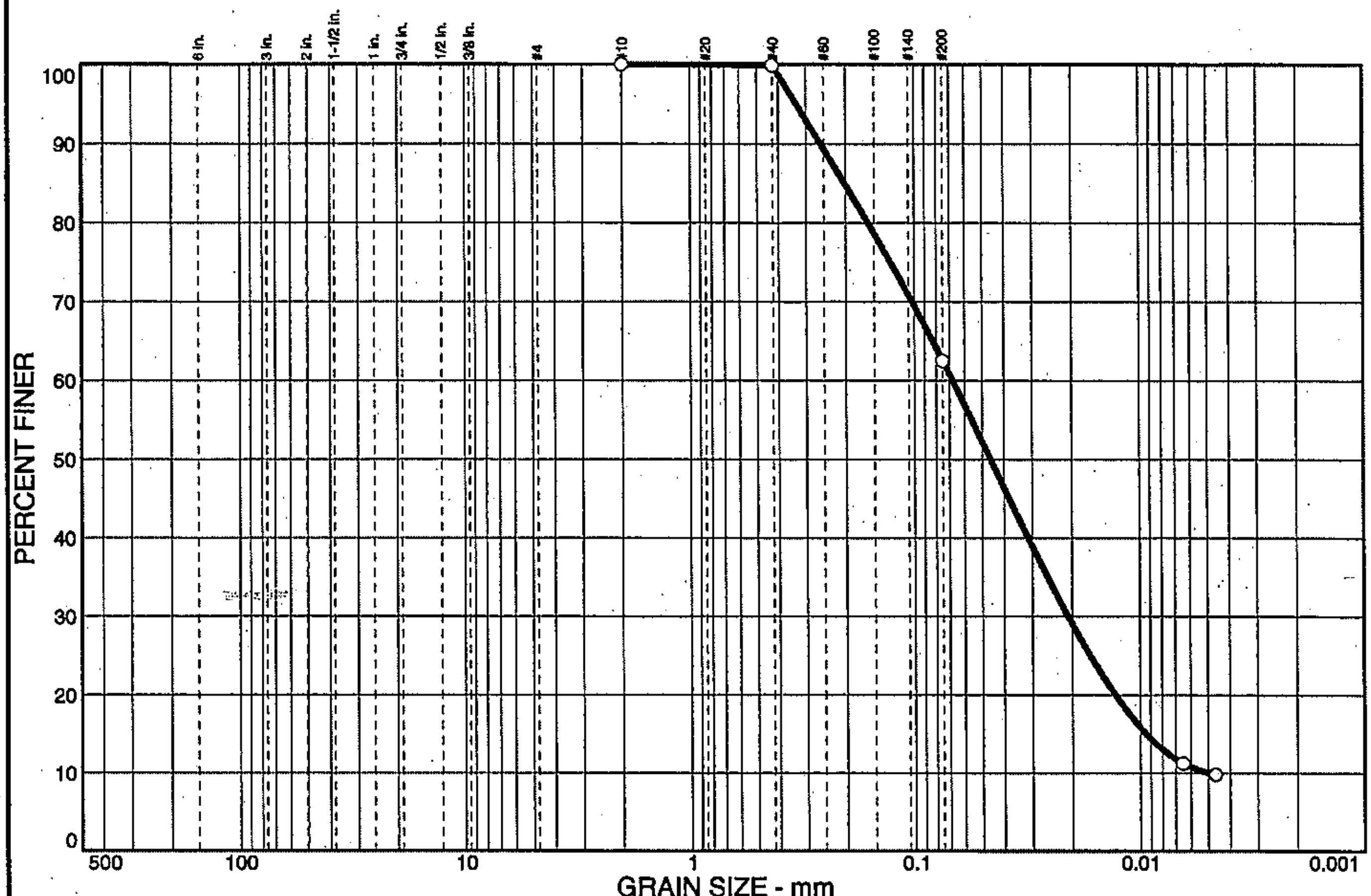


B43 R2 Chart 1



B44 R2 Chart 1

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.1	37.4	52.5	10.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	62.5		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.204 D_{60}= 0.0679 D_{50}= 0.0461
D_{30}= 0.0209 D_{15}= 0.0095 D_{10}= 0.0049
C_u= 13.74 C_c= 1.30

Classification

USCS= ML AASHTO= A-4(0)

Remarks

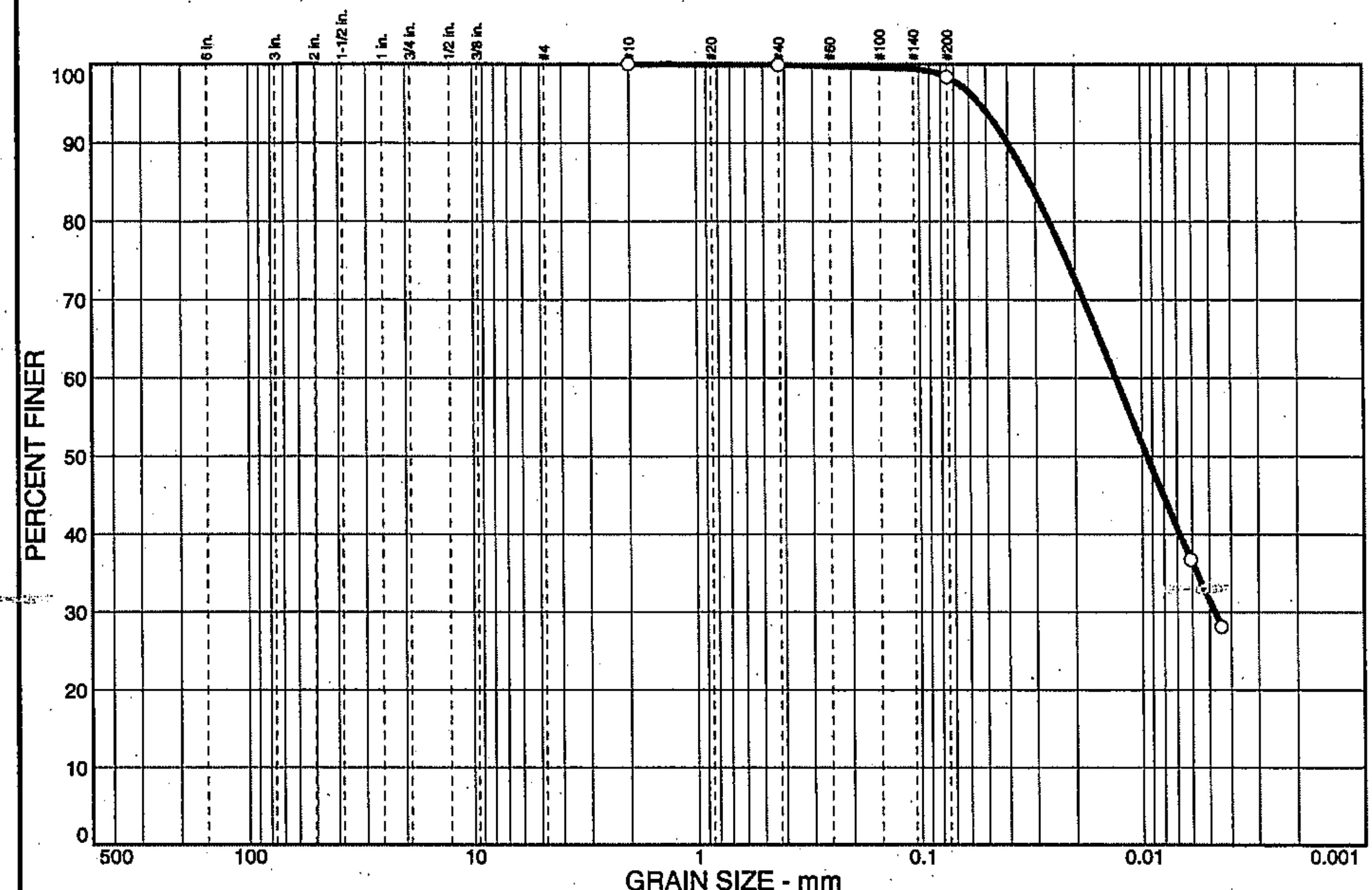
Moisture Content= 27.1%

Sample No.: 10 **Source of Sample:** B-39 **Date:** 6/4/07
Location: **Elev./Depth:** 26.0

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	0.1	1.5	67.0	31.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	98.4		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 17 LL= 35 PI= 18

Coefficients

D_{85}= 0.0320 D_{60}= 0.0132 D_{50}= 0.0095
D_{30}= 0.0047 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(18)

Remarks

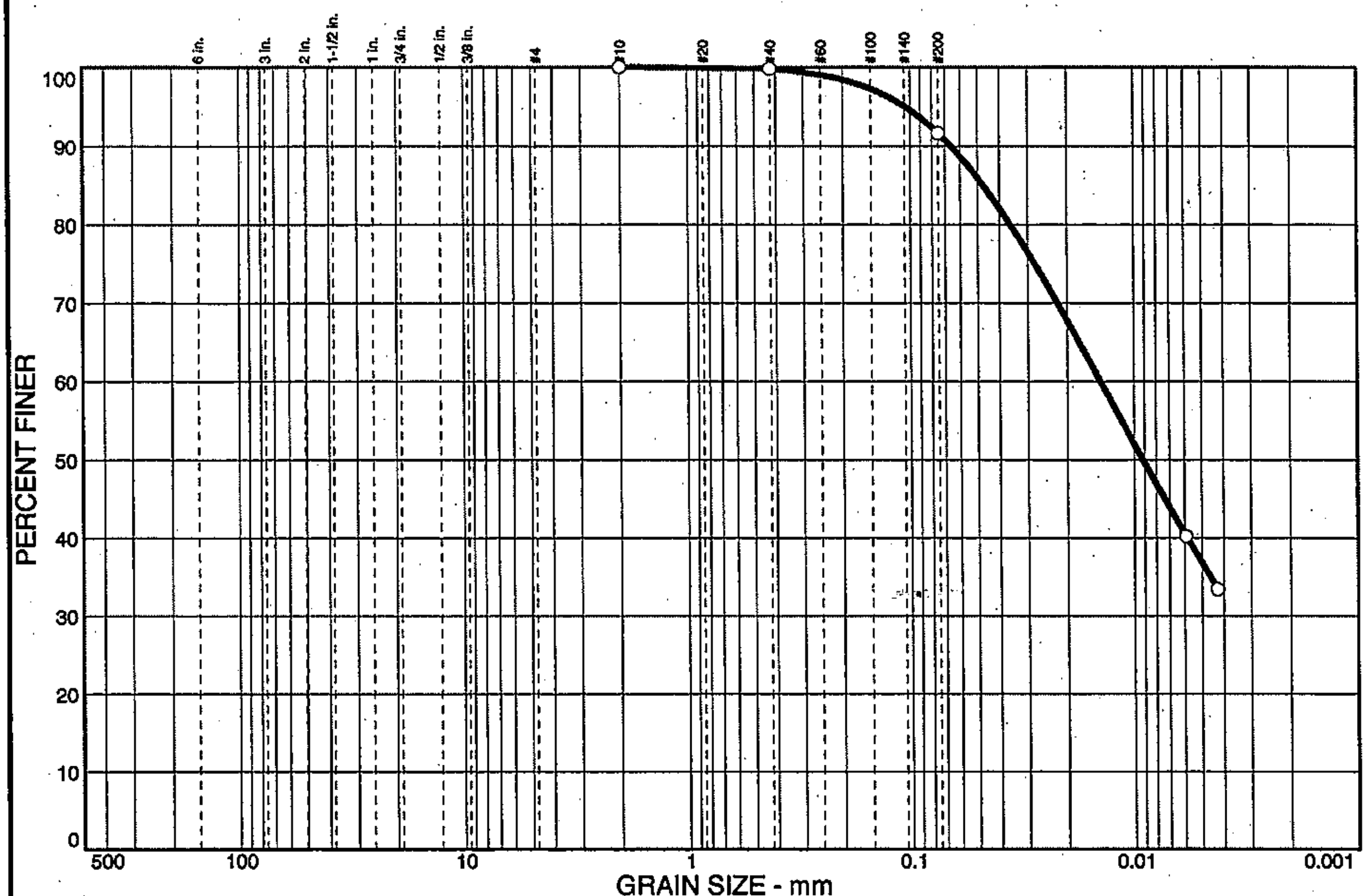
Moisture Content= 28.2%

Sample No.: 2 **Source of Sample:** B-39 **Date:** 6/4/07
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	0.0	0.2	8.2	54.9	36.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.8		
#200	91.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 32 PI= 12

Coefficients

D_{85}= 0.0475 D_{60}= 0.0142 D_{50}= 0.0092
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

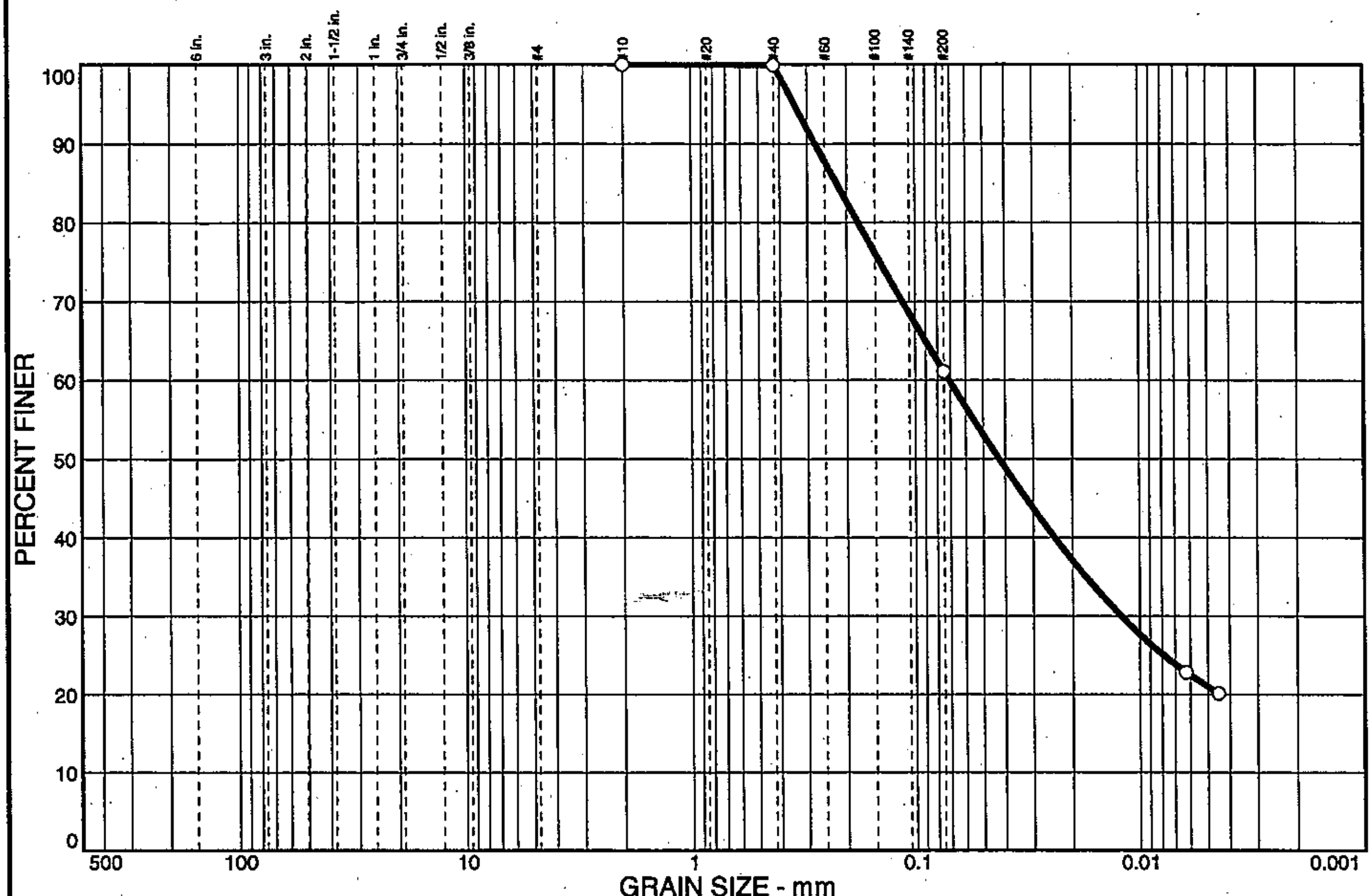
Moisture Content= 26.3%

Sample No.: 4 **Source of Sample:** B-39 **Date:** 6/4/07
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	0.1	38.8	40.2	20.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	61.1		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 17 LL= 22 PI= 5

Coefficients

D_{85}= 0.223 D_{60}= 0.0711 D_{50}= 0.0427
D_{30}= 0.0122 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 27.6%

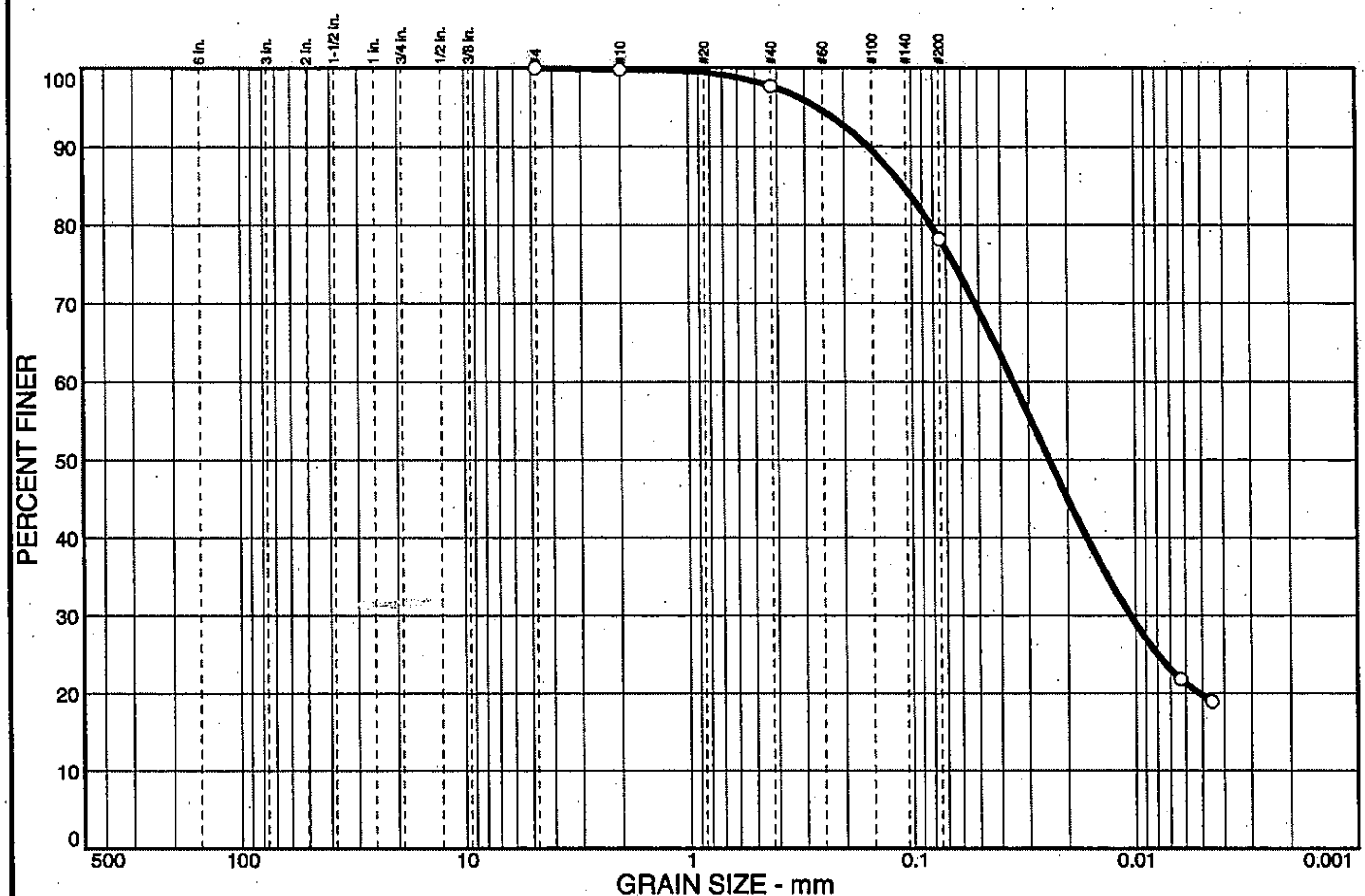
Sample No.: 5 **Source of Sample:** B-39 **Date:** 6/4/07
Location: **Elev./Depth:** 13.5

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	0.2	2.1	19.5	58.5	19.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#40	97.7		
#200	78.2		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.109 D_{60}= 0.0353 D_{50}= 0.0243
D_{30}= 0.0106 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content= 19.1%

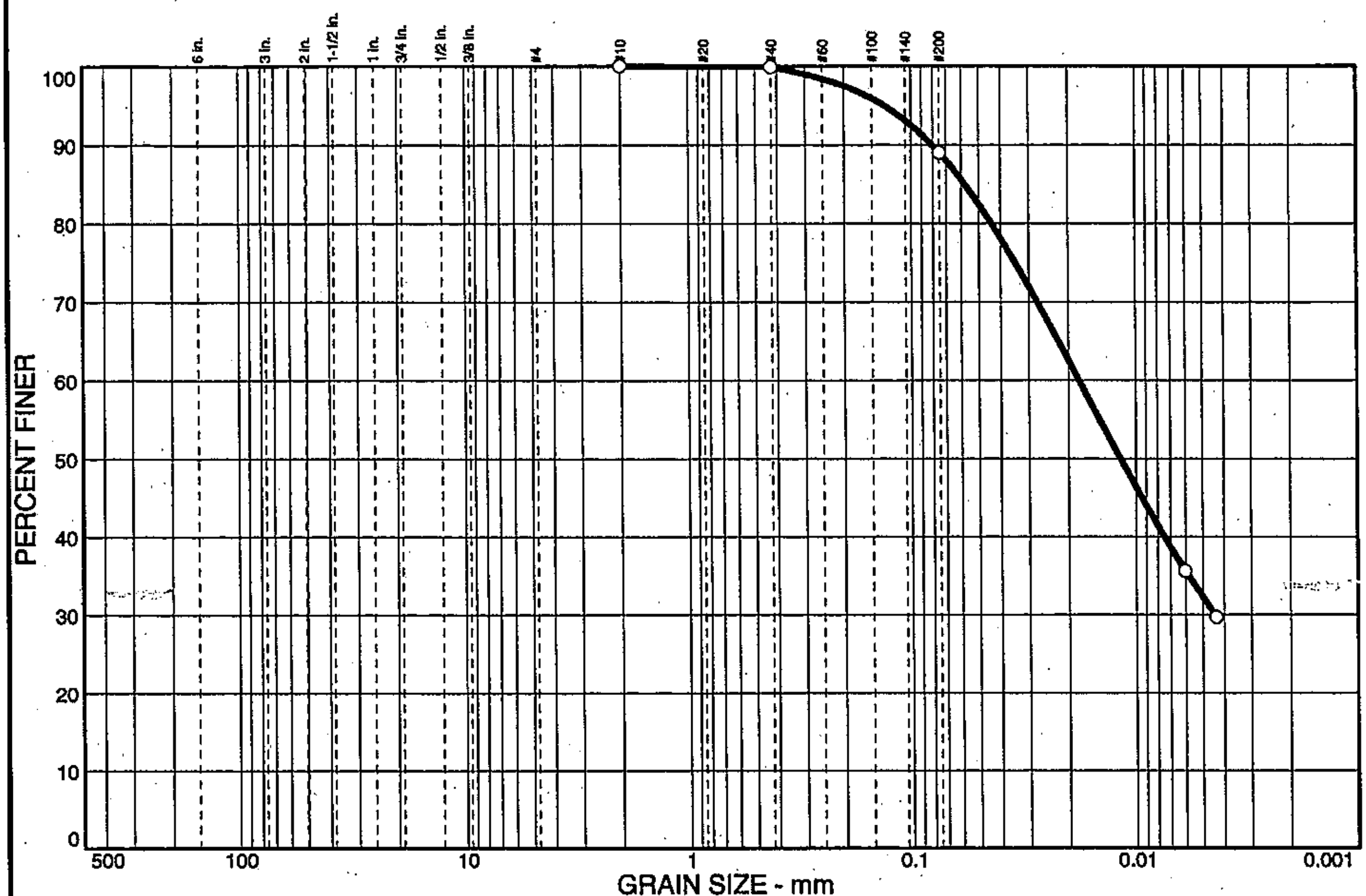
Sample No.: 8 **Source of Sample:** B-39 **Date:** 6/4/07
Location: **Elev./Depth:** 21.0

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	0.1	10.9	56.8	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	89.0		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 29 PI= 10

Coefficients

D_{85}= 0.0579 D_{60}= 0.0180 D_{50}= 0.0118
D_{30}= 0.0044 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(8)

Remarks

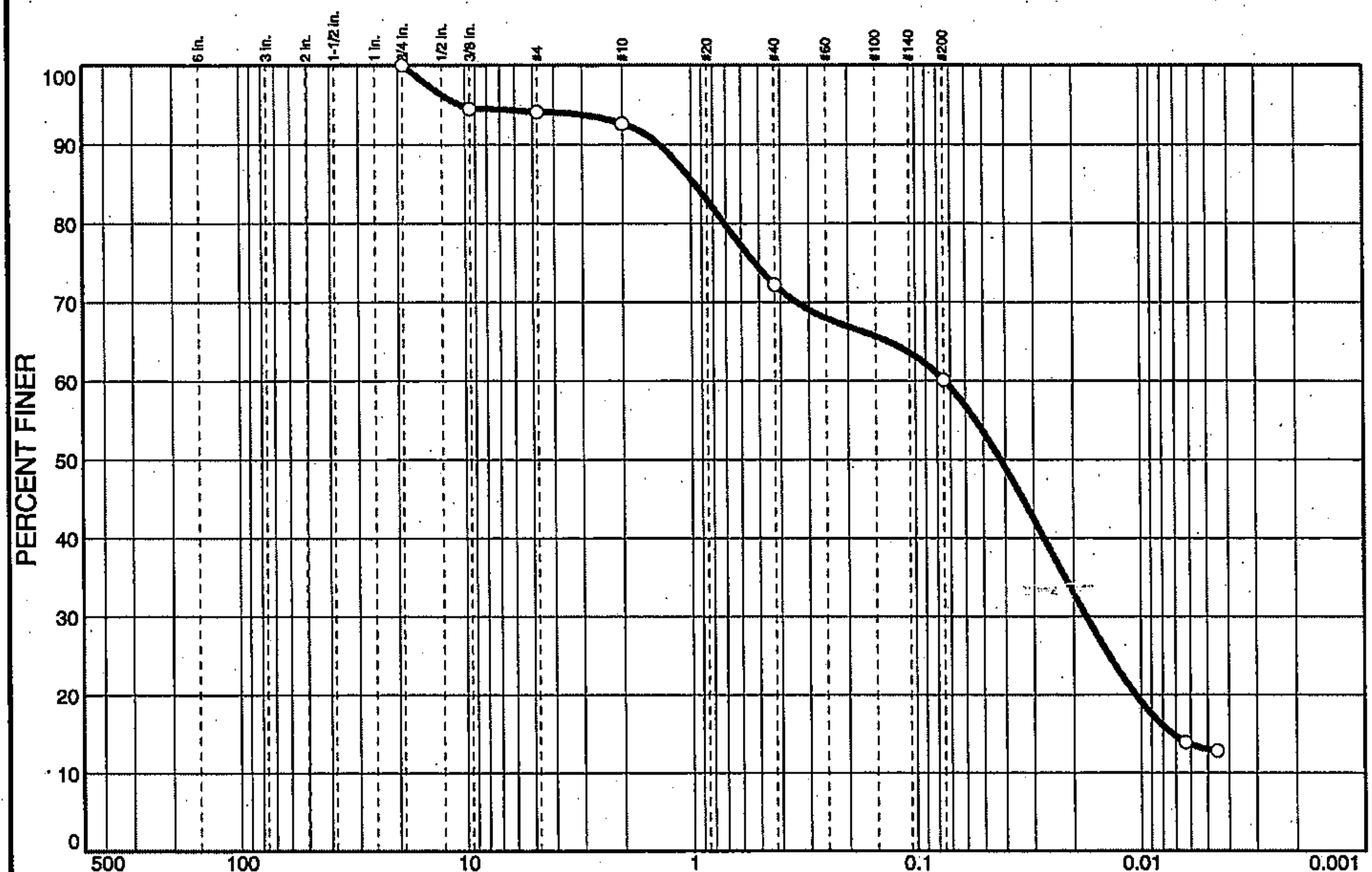
Moisture Content= 27.7%

Sample No.: ST-1 **Source of Sample:** B-39 **Date:** 6/7/07
Location: **Elev./Depth:** 10.0

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	5.9	1.5	20.4	12.1	47.1	13.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	94.5		
#4	94.1		
#10	92.6		
#40	72.2		
#200	60.1		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 19 LL= 23 PI= 14

Coefficients

D_{85}= 0.967 D_{60}= 0.0744 D_{50}= 0.0422
D_{30}= 0.0176 D_{15}= 0.0073 D_{10}=
C_u= C_c=

Classification

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

Remarks

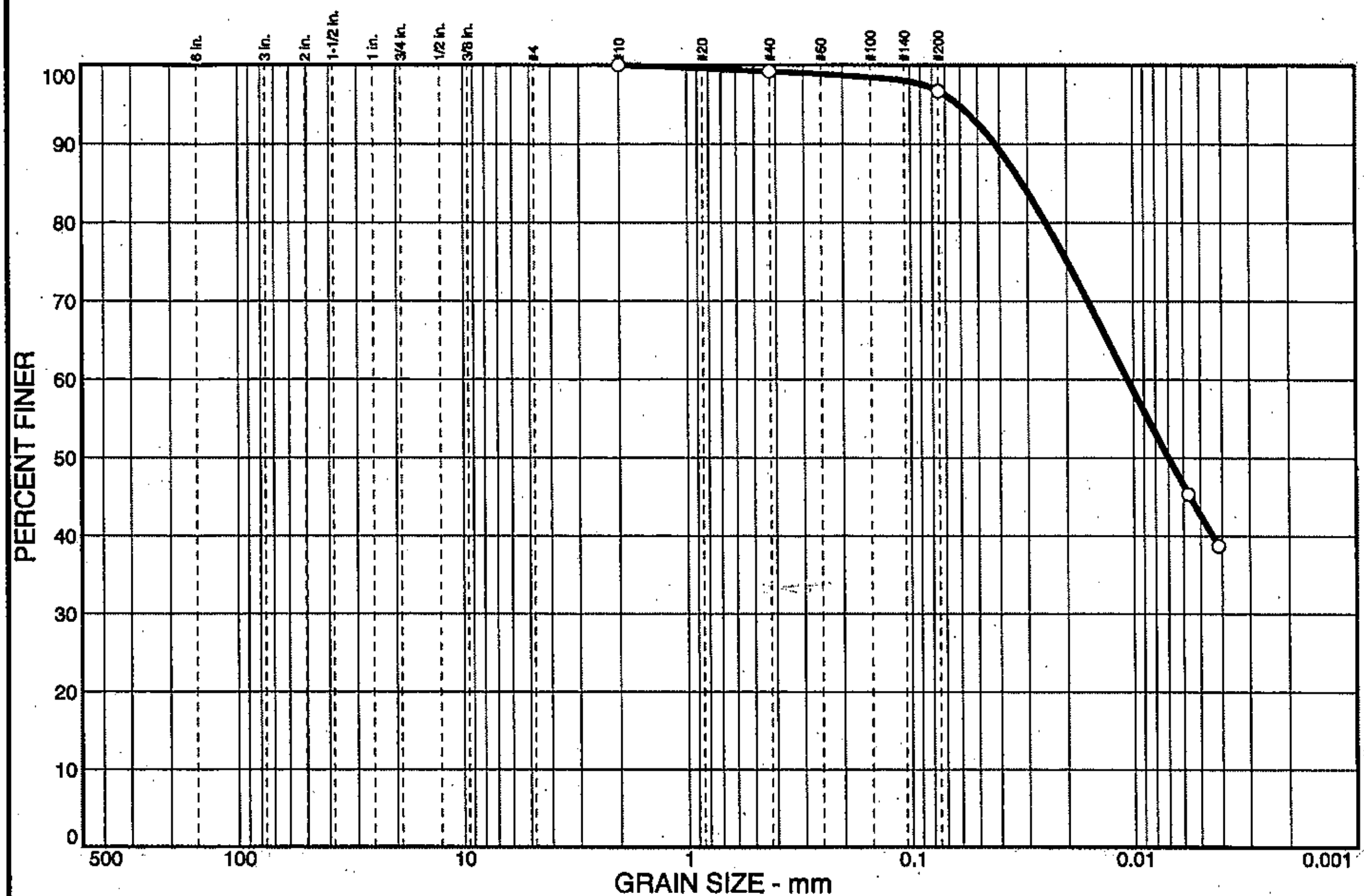
Moisture Content= 14.7%

Sample No.: 11 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 38.5

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	0.8	2.5	54.1	42.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.2		
#200	96.7		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 37 PI= 16

Coefficients

D_{85}= 0.0318 D_{60}= 0.0107 D_{50}= 0.0070
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(16)

Remarks

Moisture Content= 23.0%

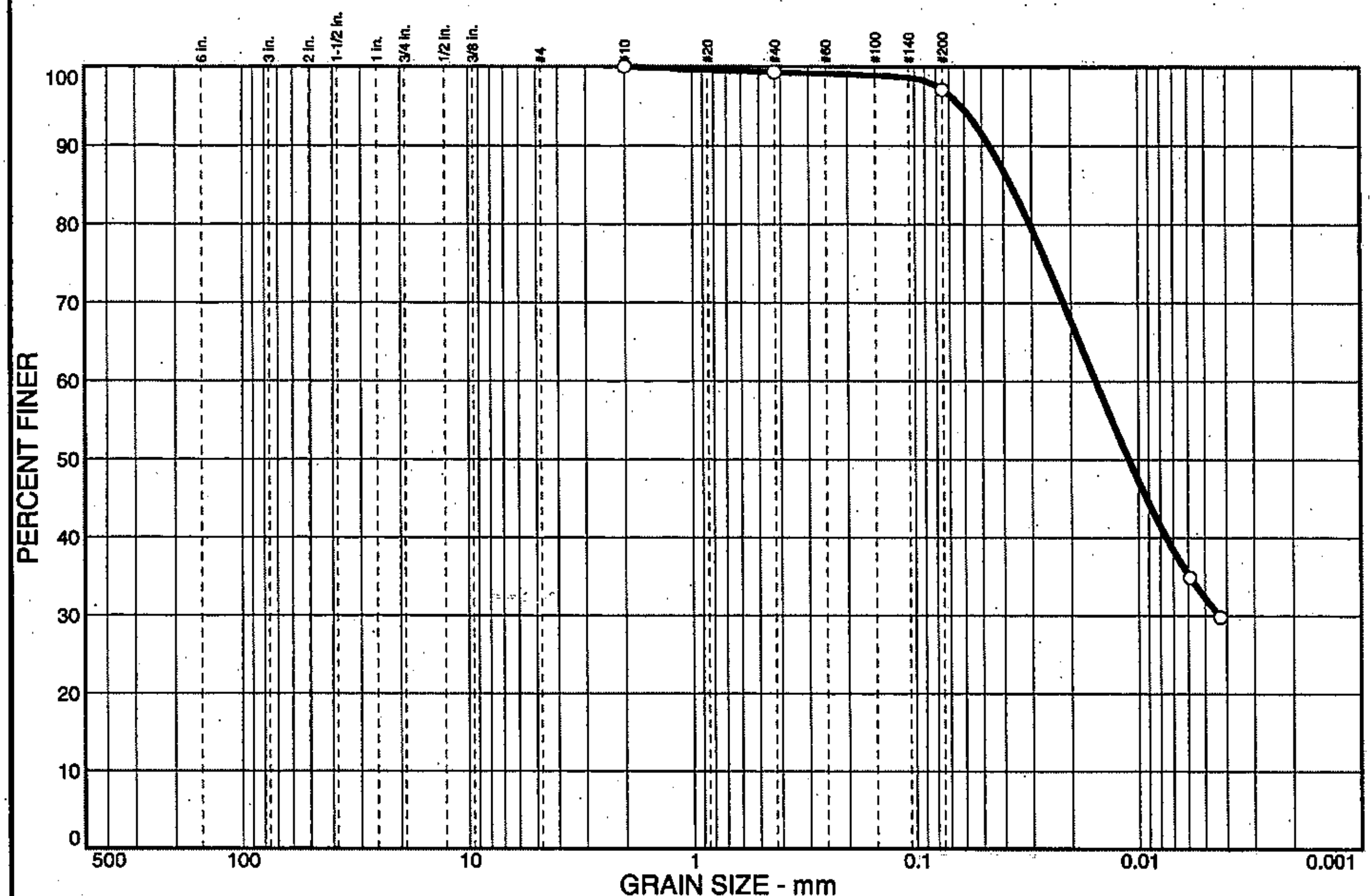
Sample No.: 2 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 3.5

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	0.0	0.7	2.2	64.9	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.3		
#200	97.1		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{85}= 0.0373 D_{60}= 0.0156 D_{50}= 0.0111
D_{30}= 0.0043 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

Moisture Content= 24.3%

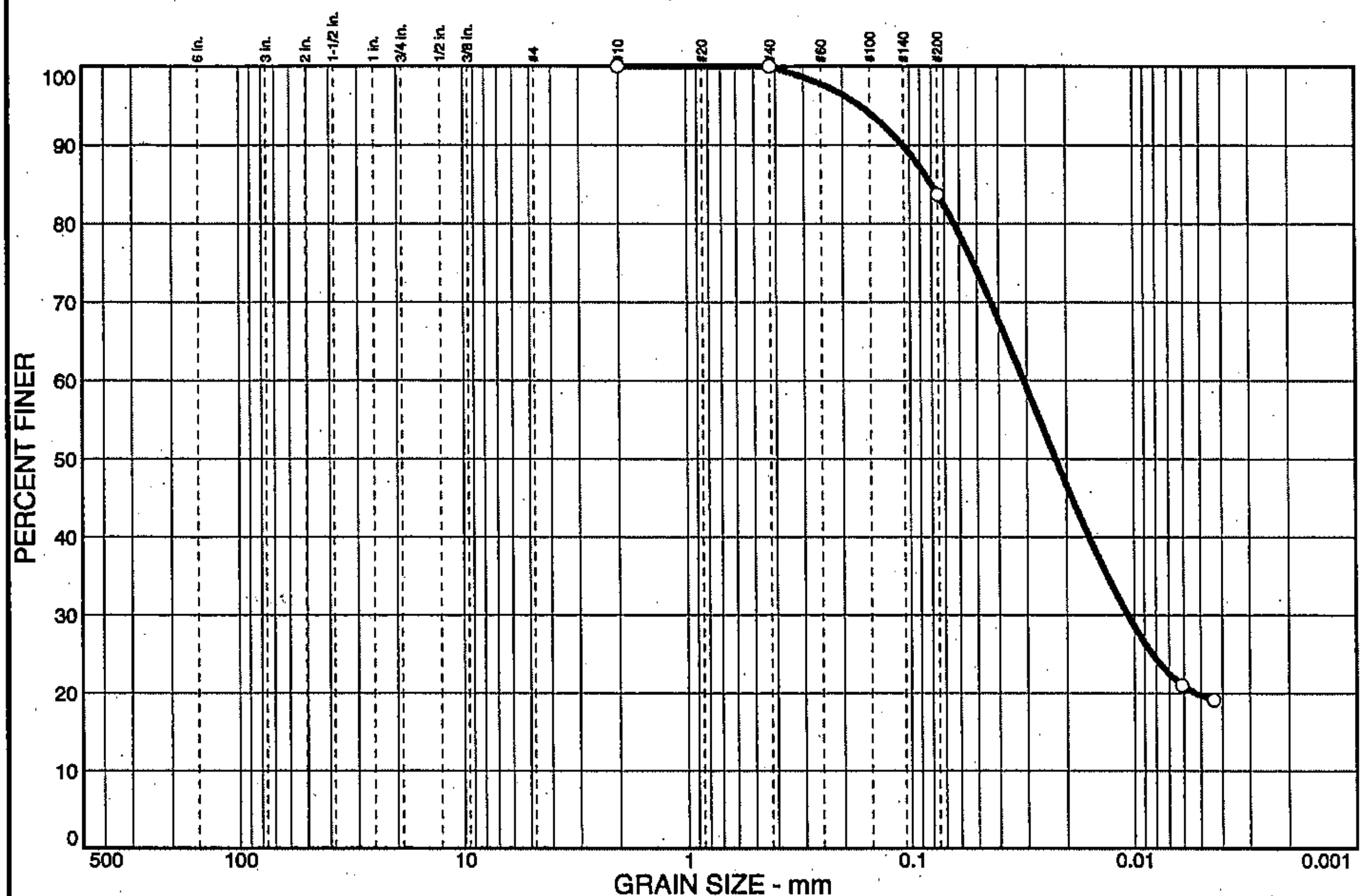
Sample No.: 3 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 8.5

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	0.0	16.3	64.0	19.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	100.0		
#200	83.7		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.0800 D_{60}= 0.0311 D_{50}= 0.0224
D_{30}= 0.0107 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

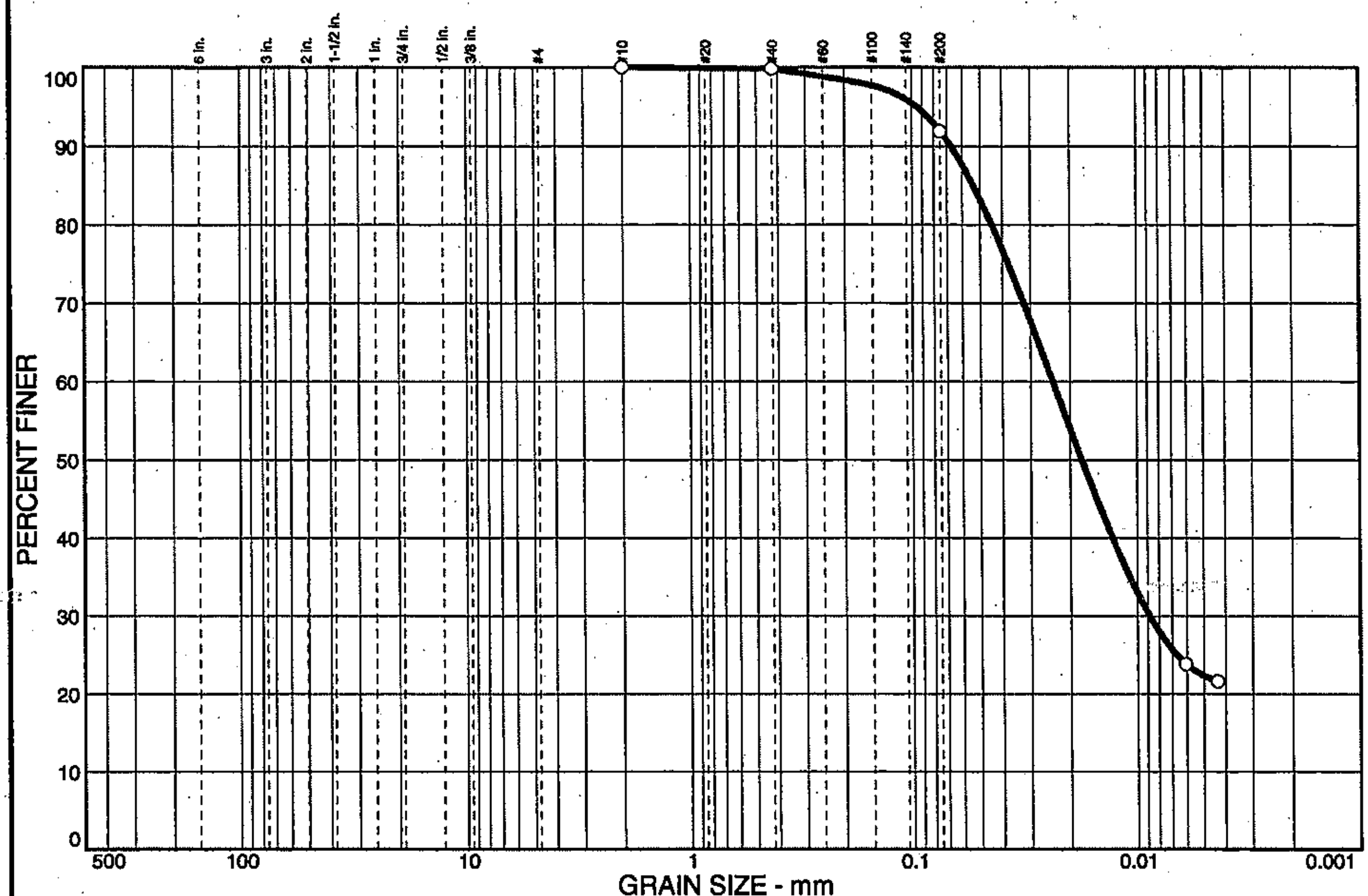
Moisture Content= 24.6%

Sample No.: 4 **Source of Sample:** B-40 **Date:** 6/12/07
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	0.2	7.9	69.6	22.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.8		
#200	91.9		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 30 PI= 10

Coefficients

D_{85}= 0.0538 D_{60}= 0.0240 D_{50}= 0.0178
D_{30}= 0.0088 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(9)

Remarks

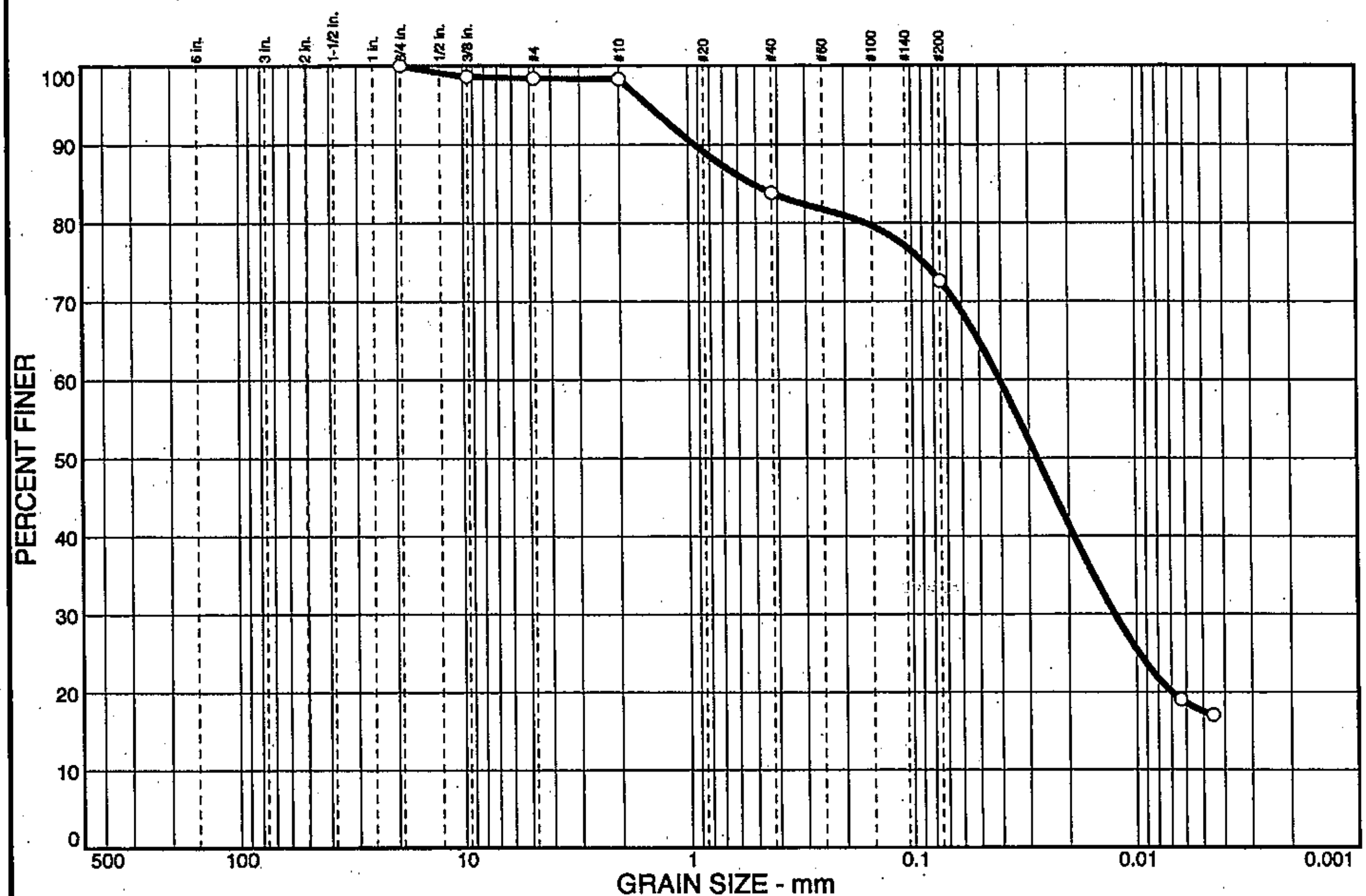
Moisture Content= 25.0%

Sample No.: 6 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 18.5

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	0.1	14.5	11.2	55.1	17.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.6		
#4	98.4		
#10	98.3		
#40	83.8		
#200	72.6		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 17 LL= 21 PI= 4

Coefficients

D_{85}= 0.519 D_{60}= 0.0408 D_{50}= 0.0277
D_{30}= 0.0125 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

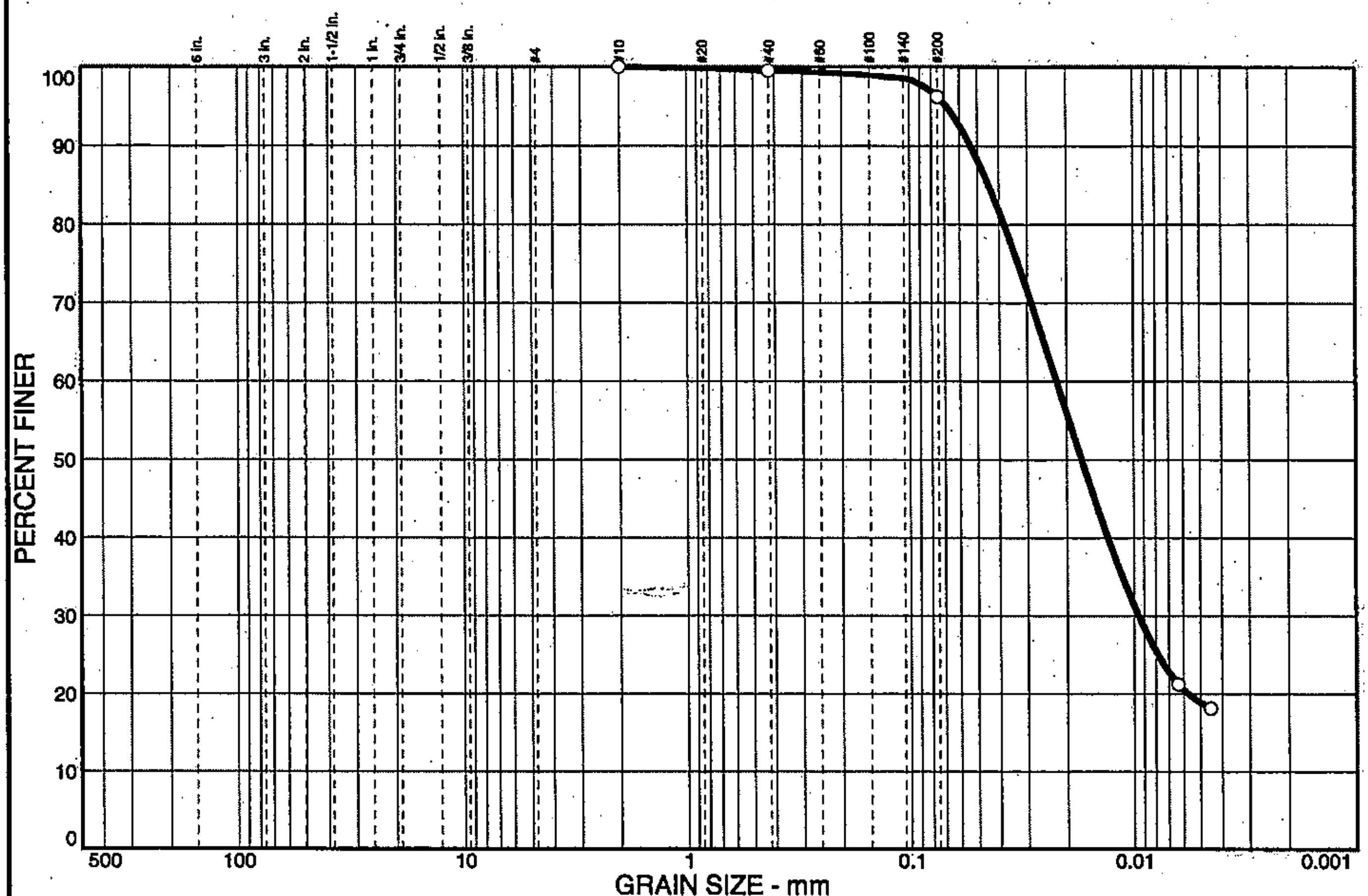
Moisture Content= 15.4%

Sample No.: 8 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 26.0

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	0.5	3.3	77.4	18.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.5		
#200	96.2		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 20 LL= 25 PI= 5

Coefficients

D_{85}= 0.0447 D_{60}= 0.0223 D_{50}= 0.0172
D_{30}= 0.0096 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

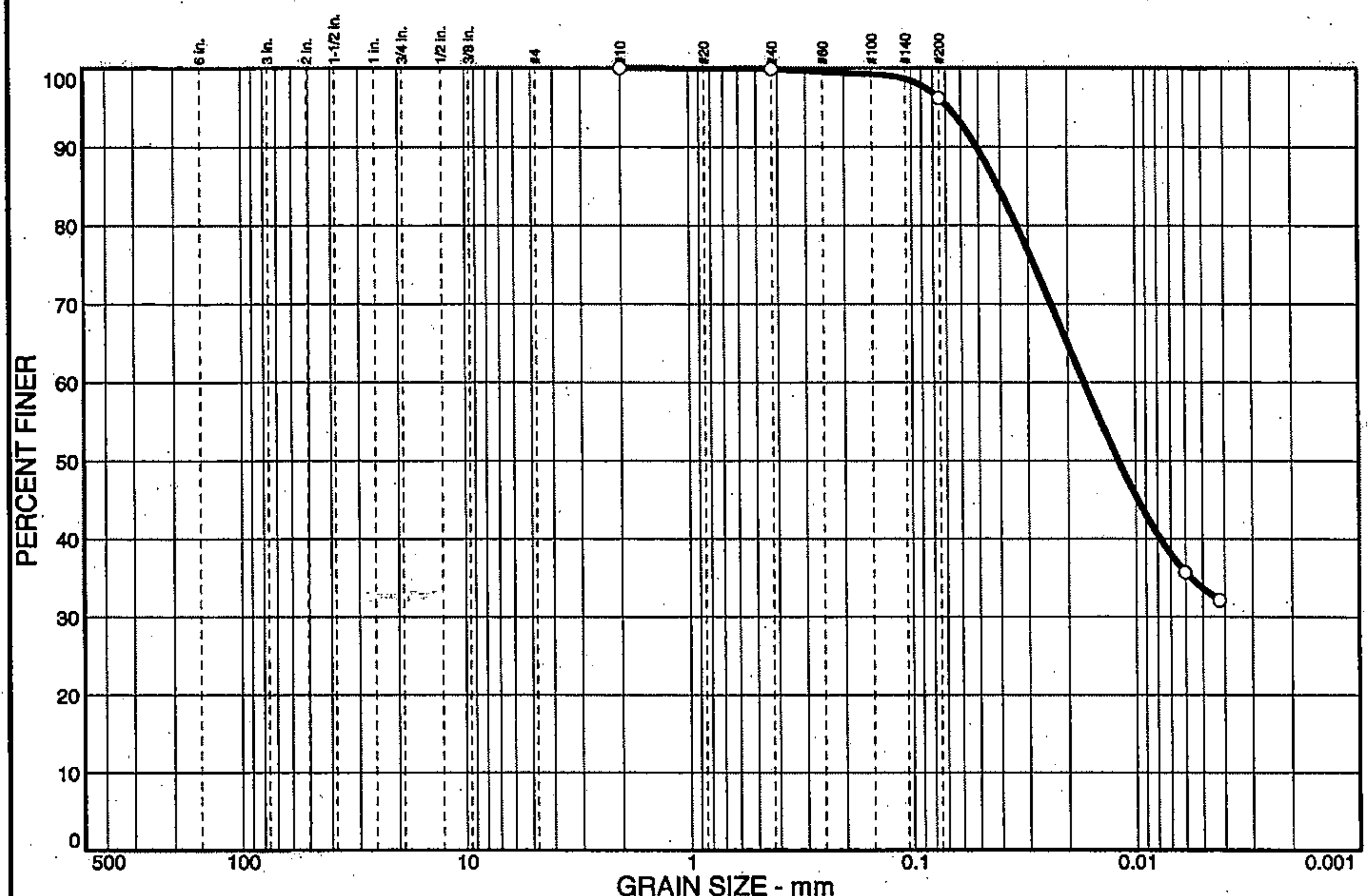
Moisture Content= 22.3%

Sample No.: ST-4 **Source of Sample:** B-40 **Date:** 6/12/07
Location: **Elev./Depth:** 20.0

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	0.1	3.7	62.6	33.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	96.2		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 31 PI= 10

Coefficients

D_{85}= 0.0410 D_{60}= 0.0170 D_{50}= 0.0120
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(9)

Remarks

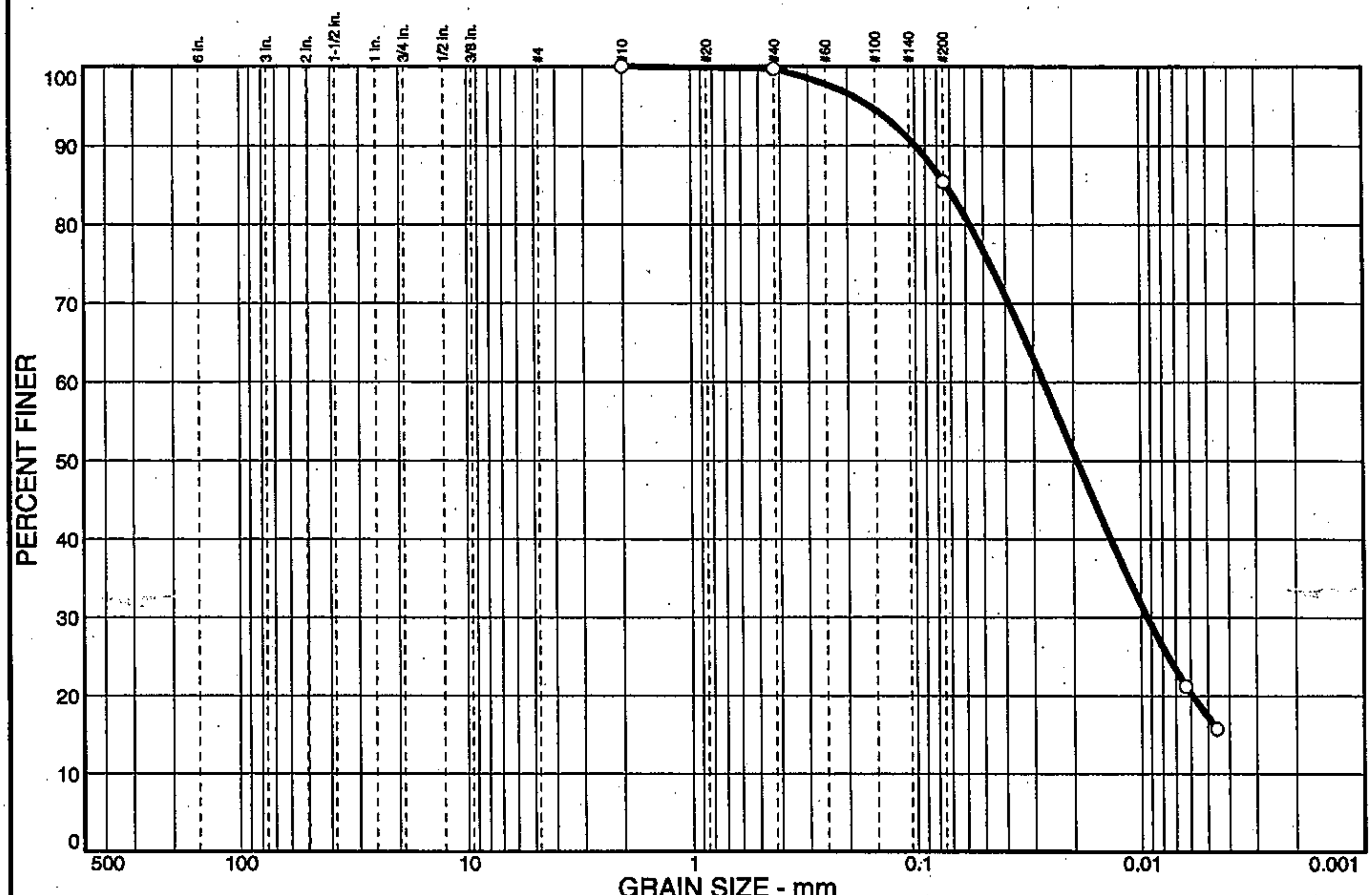
Moisture Content= 23.8%

Sample No.: 2 **Source of Sample:** B-41 **Date:** 6/4/07
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	0.0	0.3	14.3	68.2	17.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	85.4		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 21 LL= 25 PI= 4

Coefficients

D_{85}= 0.0734 D_{60}= 0.0272 D_{50}= 0.0194
D_{30}= 0.0094 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

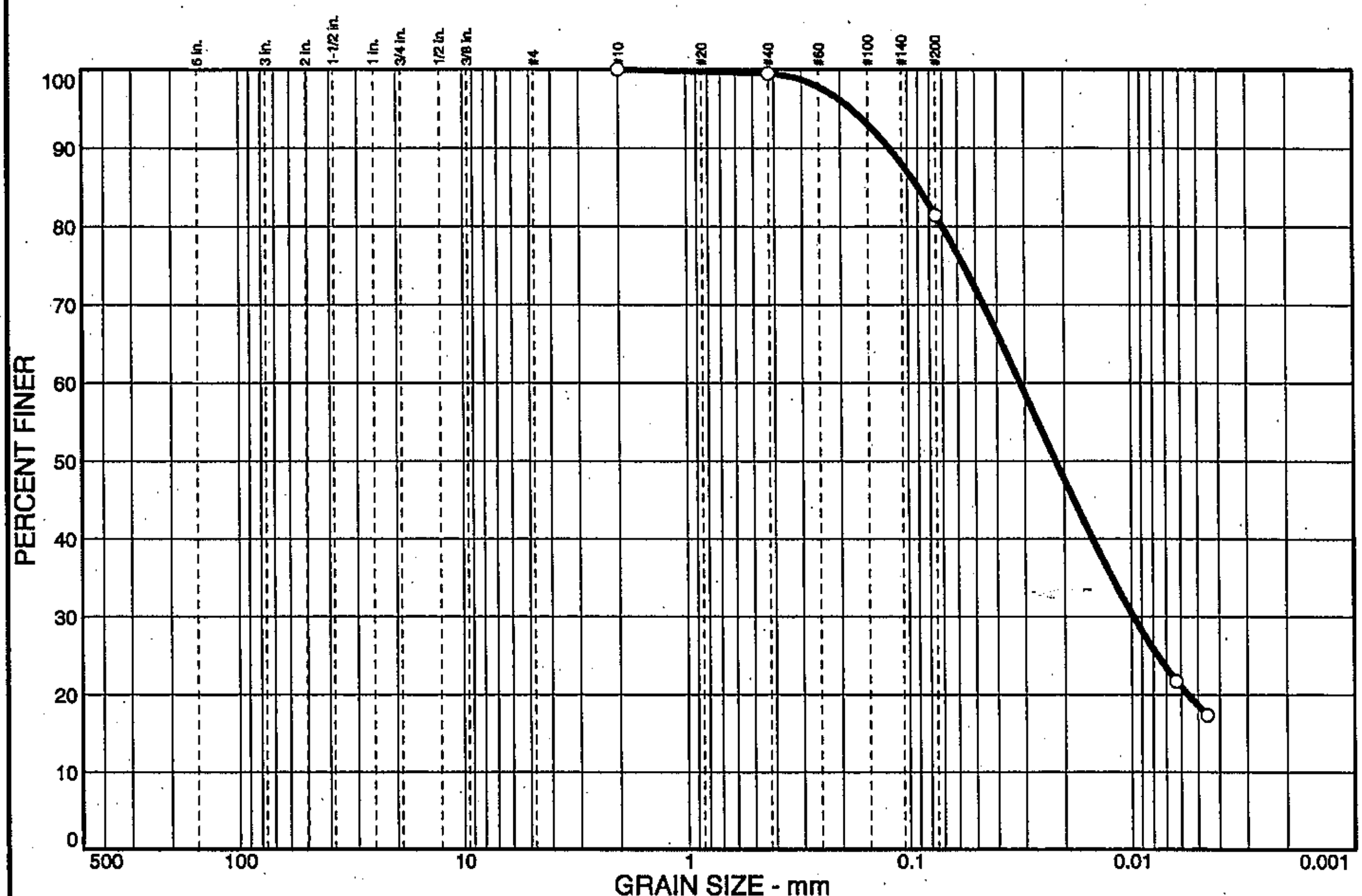
Moisture Content= 23.8%

Sample No.: 4 **Source of Sample:** B-41 **Date:** 6/4/07
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	0.5	18.1	62.9	18.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.5		
#200	81.4		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 20 LL= 25 PI= 5

Coefficients

D_{85}= 0.0899 D_{60}= 0.0316 D_{50}= 0.0219
D_{30}= 0.0098 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

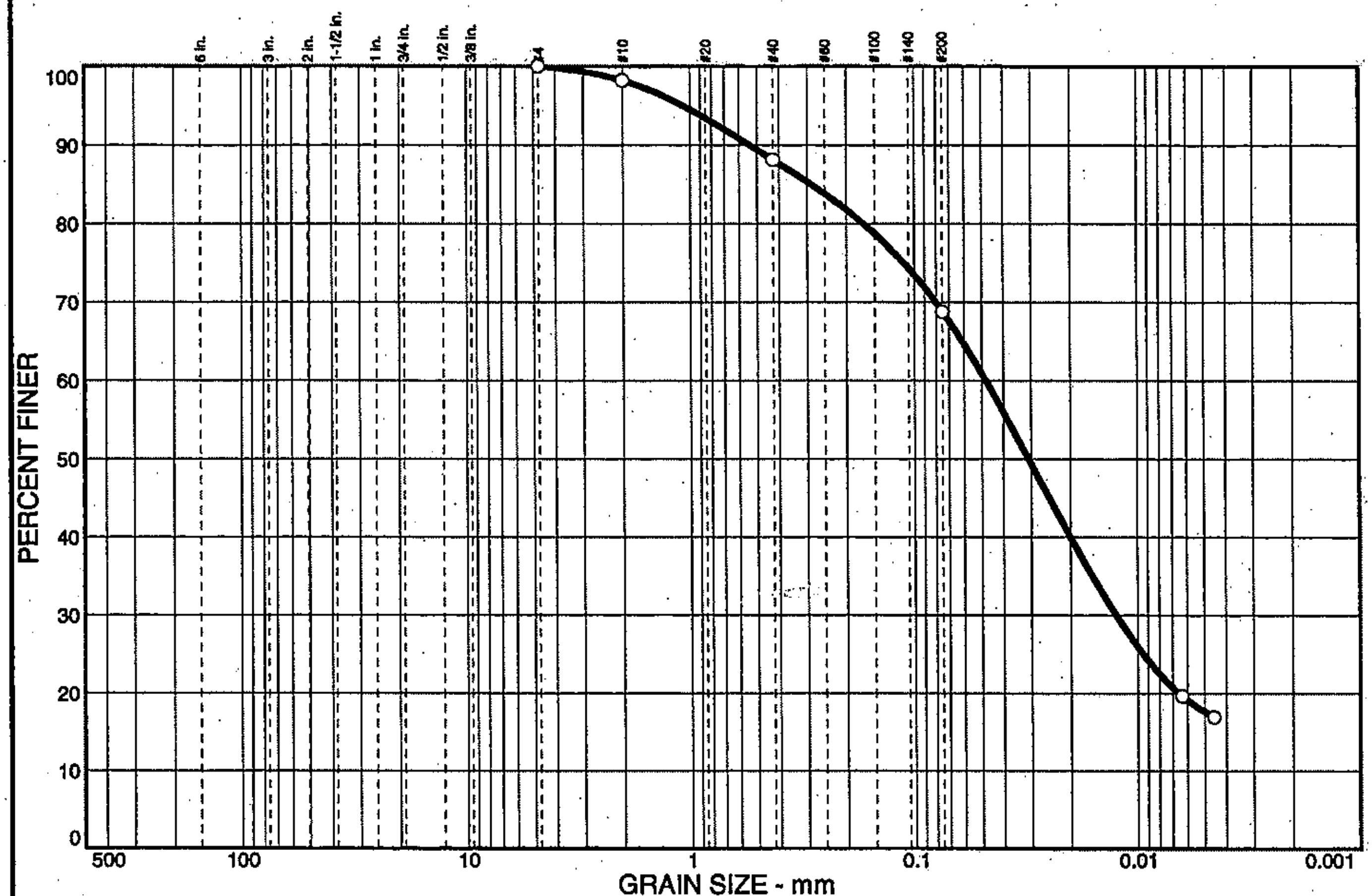
Moisture Content= 23.8%

Sample No.: 6 **Source of Sample:** B-41 **Date:** 6/4/07
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	1.8	10.1	19.3	51.2	17.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.2		
#40	88.1		
#200	68.8		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= 17 LL= 20 PI= 3

Coefficients

D_{85}= 0.289 D_{60}= 0.0482 D_{50}= 0.0310
D_{30}= 0.0125 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

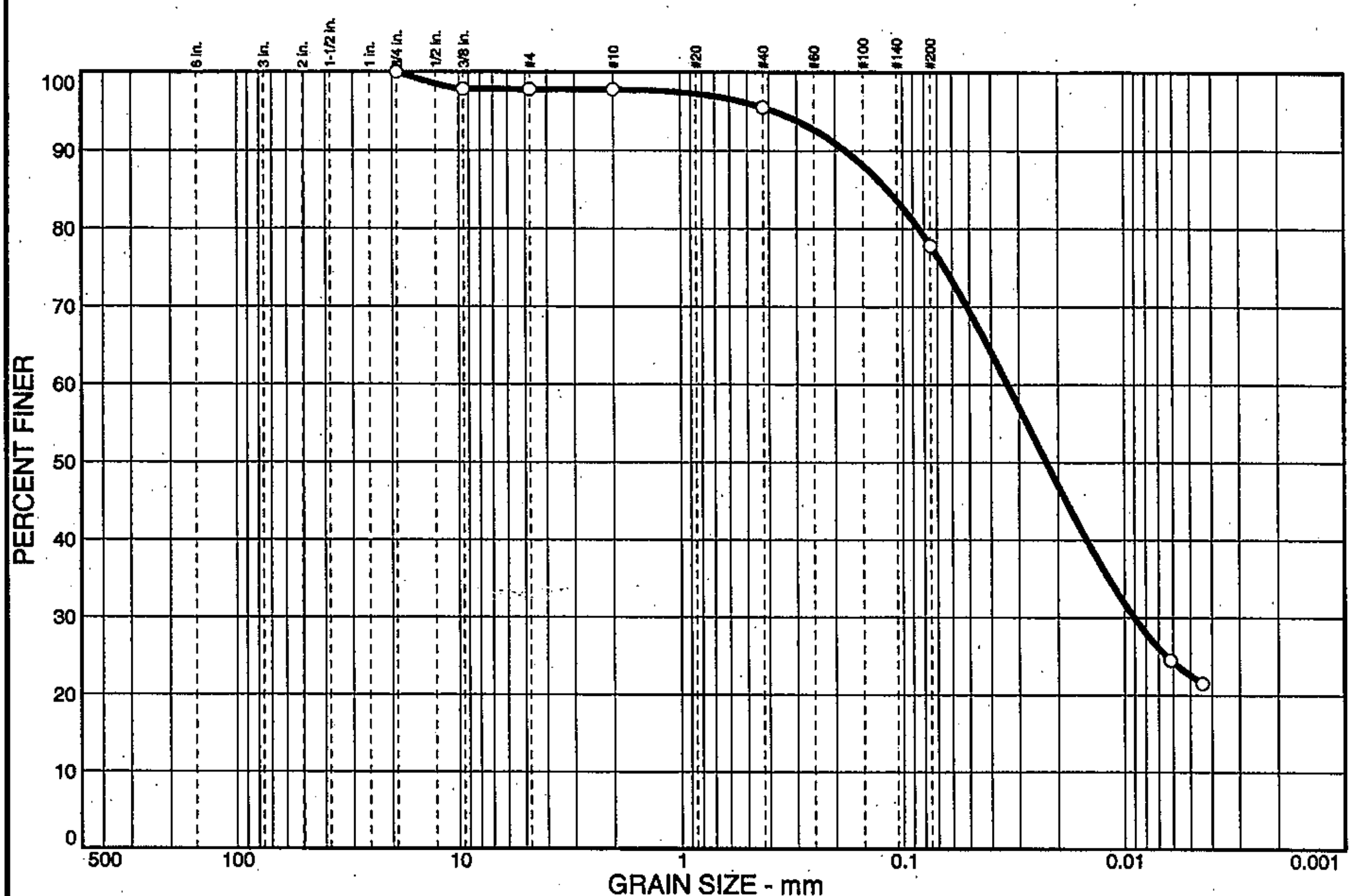
Moisture Content= 16.2%

Sample No.: 8 **Source of Sample:** B-41 **Date:** 6/4/07
Location: **Elev./Depth:** 18.5

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	2.2	0.0	2.3	17.7	55.3	22.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	97.8		
#4	97.8		
#10	97.8		
#40	95.5		
#200	77.8		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 18 LL= 25 PI= 7

Coefficients

D_{85}= 0.117 D_{60}= 0.0338 D_{50}= 0.0226
D_{30}= 0.0091 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 20.0%

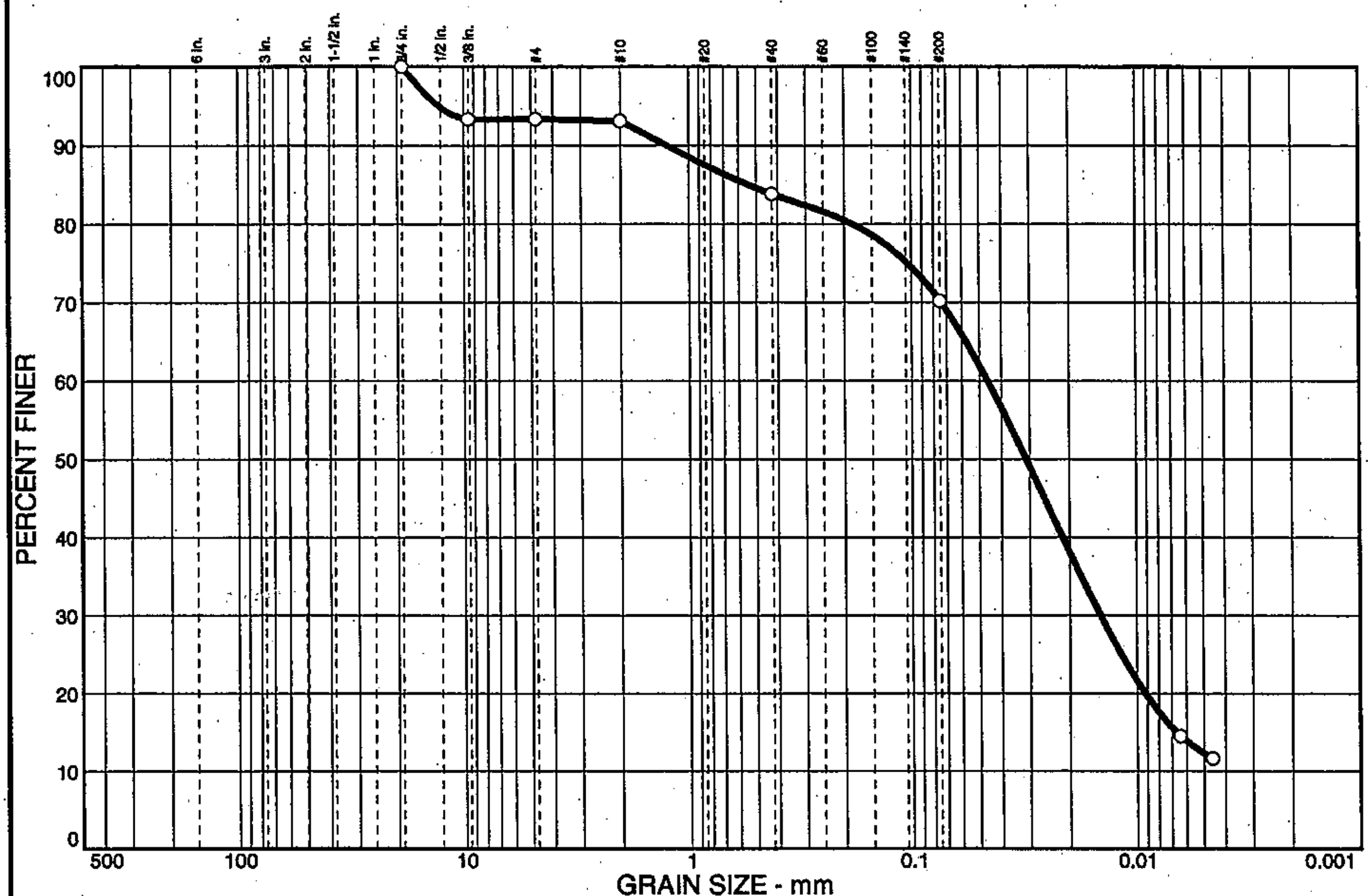
Sample No.: 10 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 22.5

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	6.7	0.2	9.3	13.6	57.9	12.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	93.3		
#4	93.3		
#10	93.1		
#40	83.8		
#200	70.2		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.545 D_{60}= 0.0462 D_{50}= 0.0312
D_{30}= 0.0146 D_{15}= 0.0067 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content= 15.6%

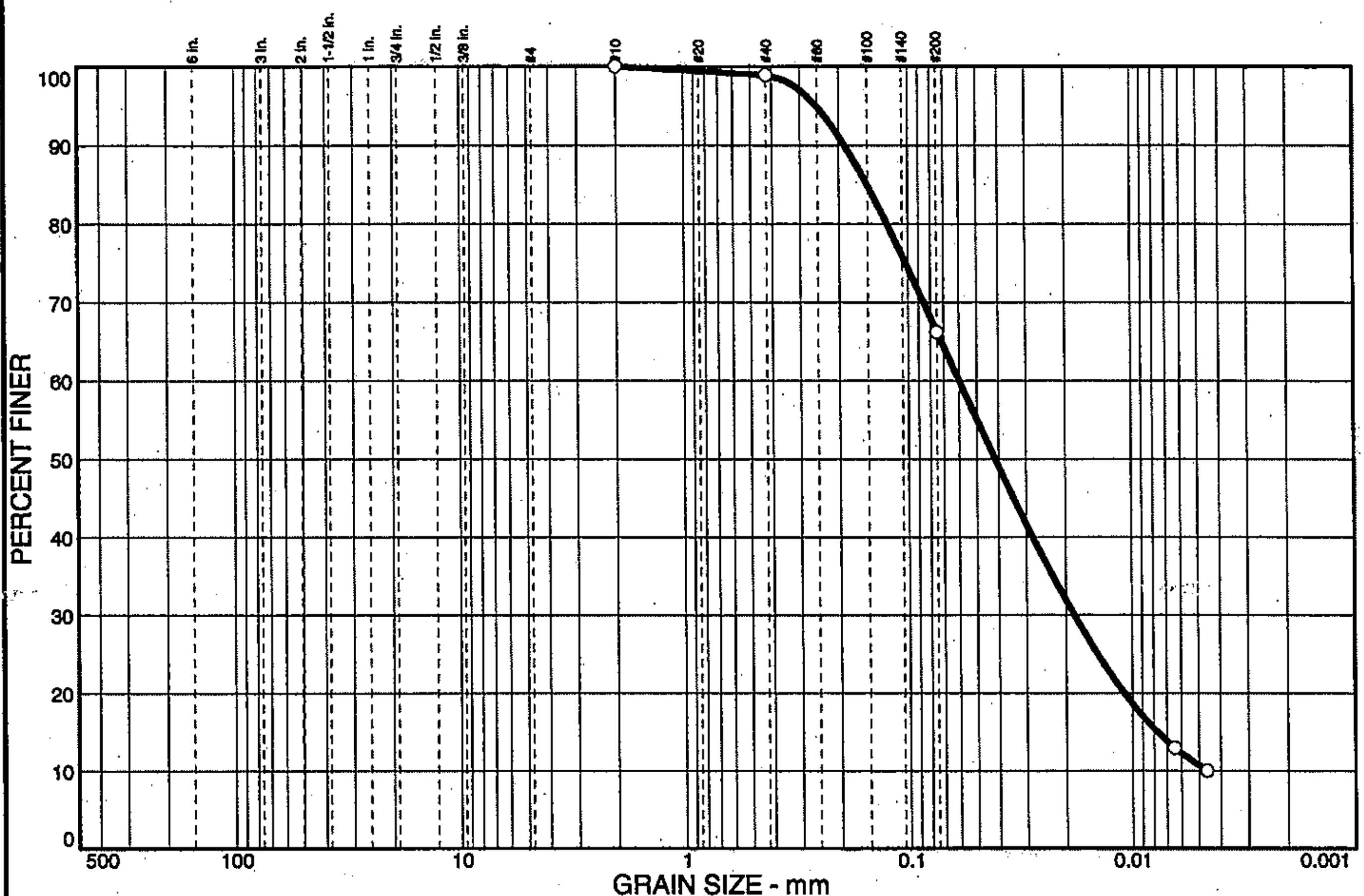
Sample No.: 11 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 25

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	1.1	32.7	55.5	10.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	98.9		
#200	66.2		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.152 D_{60}= 0.0601 D_{50}= 0.0416
D_{30}= 0.0184 D_{15}= 0.0078 D_{10}= 0.0046
C_u= 13.06 C_c= 1.22

Classification

USCS= ML AASHTO= A-4(0)

Remarks

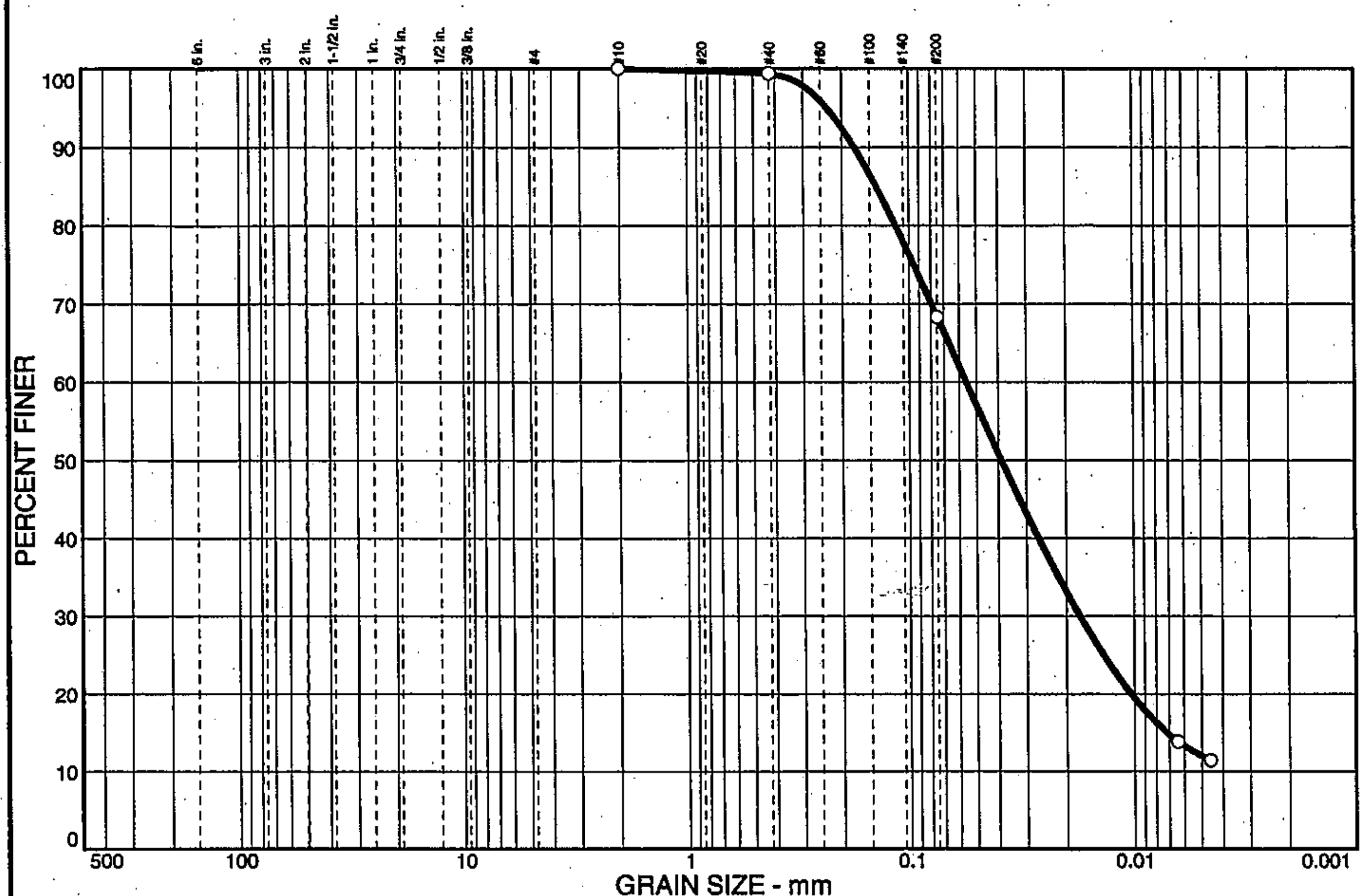
Moisture Content= 24.2%

Sample No.: 12 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 27.5

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	0.6	31.1	56.3	12.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.4		
#200	68.3		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.141 D_{60}= 0.0558 D_{50}= 0.0389
D_{30}= 0.0175 D_{15}= 0.0072 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

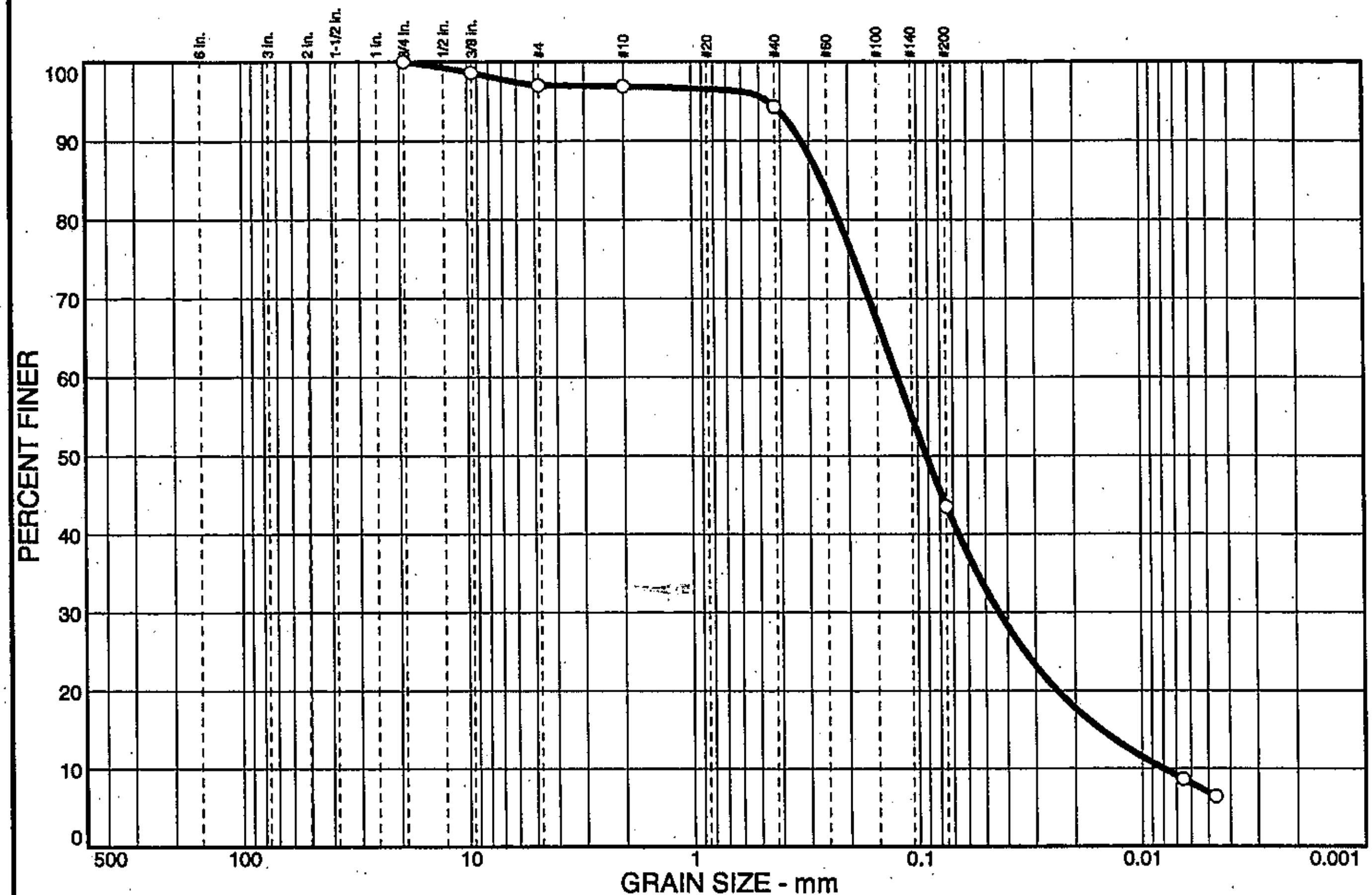
Moisture Content= 24.9%

Sample No.: 13 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 30

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	3.0	0.1	2.6	50.8	36.6	6.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.6		
#4	97.0		
#10	96.9		
#40	94.3		
#200	43.5		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.264 D_{60}= 0.123 D_{50}= 0.0921
D_{30}= 0.0439 D_{15}= 0.0154 D_{10}= 0.0081
C_u= 15.24 C_c= 1.94

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content= 24.6%

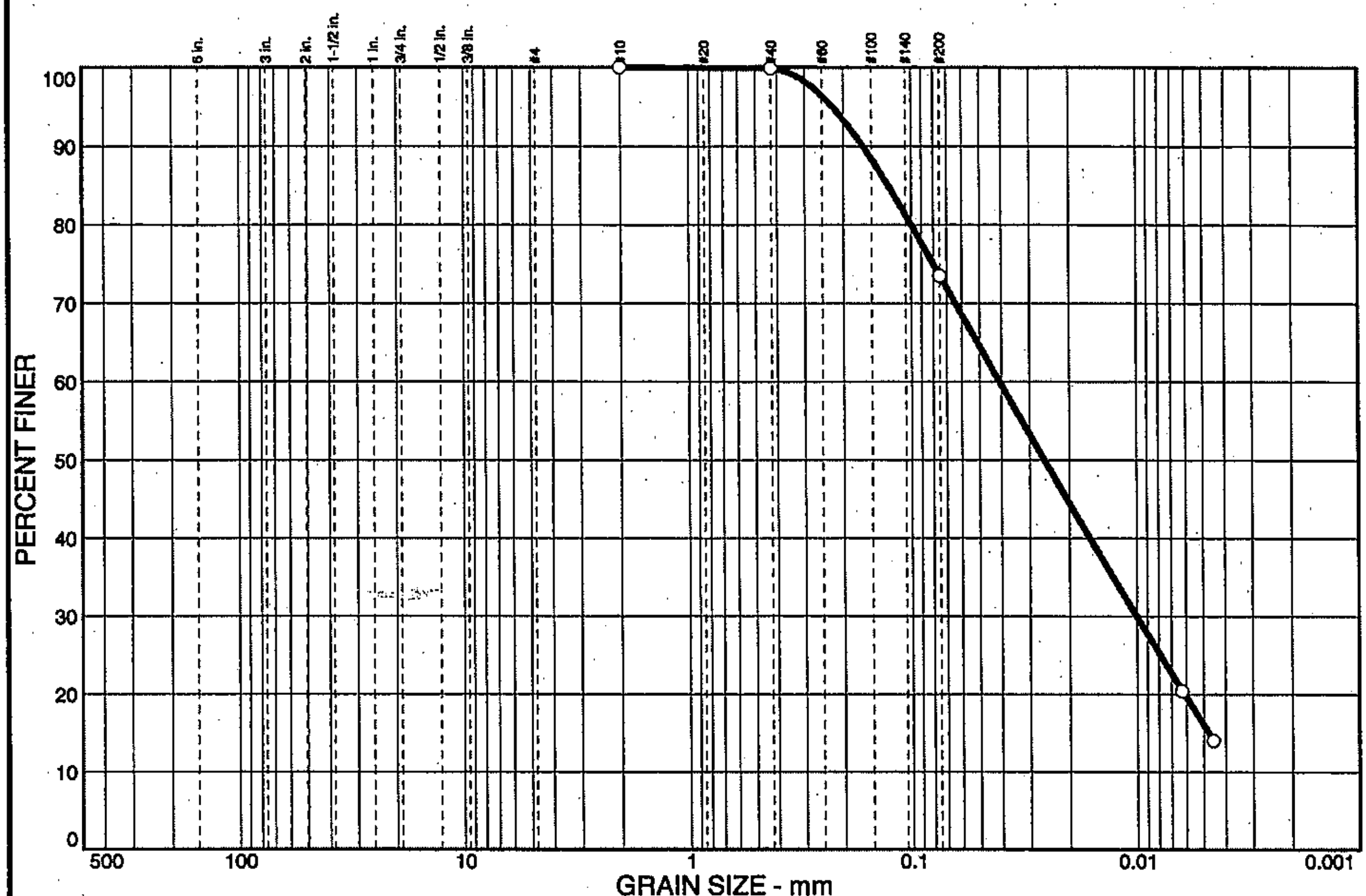
Sample No.: 14 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 35

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	0.1	26.4	57.5	16.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	73.5		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 19 LL= 23 PI= 4

Coefficients

D_{85}= 0.127 D_{60}= 0.0409 D_{50}= 0.0259
D_{30}= 0.0101 D_{15}= 0.0048 D_{10}=
C_u= C_c=

Classification

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

Remarks

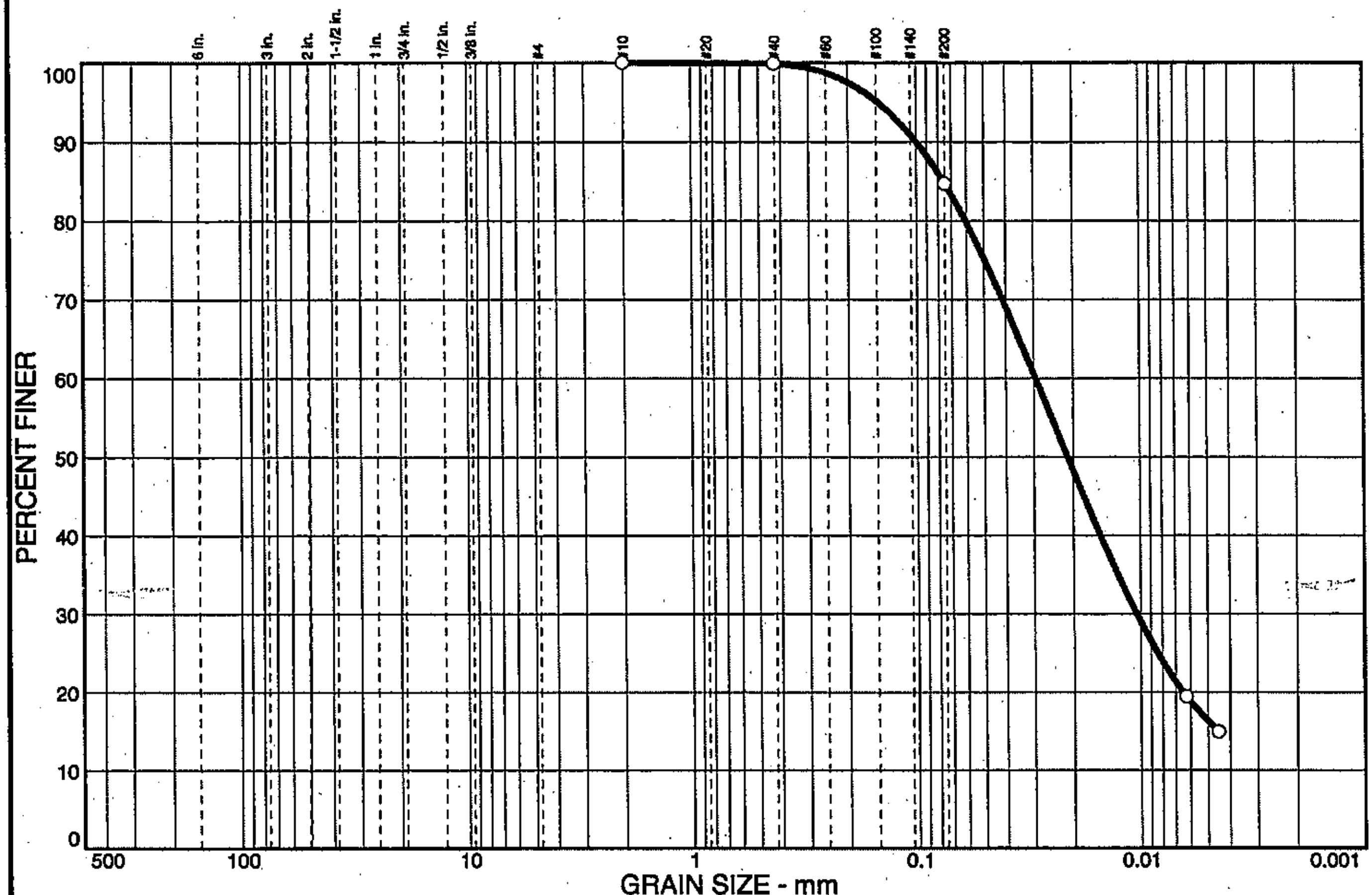
Moisture Content= 23.8%

Sample No.: 3 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 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	0.1	15.2	68.5	16.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	84.7		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 19 LL= 24 PI= 5

Coefficients

D_{85}= 0.0761 D_{60}= 0.0296 D_{50}= 0.0213
D_{30}= 0.0105 D_{15}= 0.0045 D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 24.9%

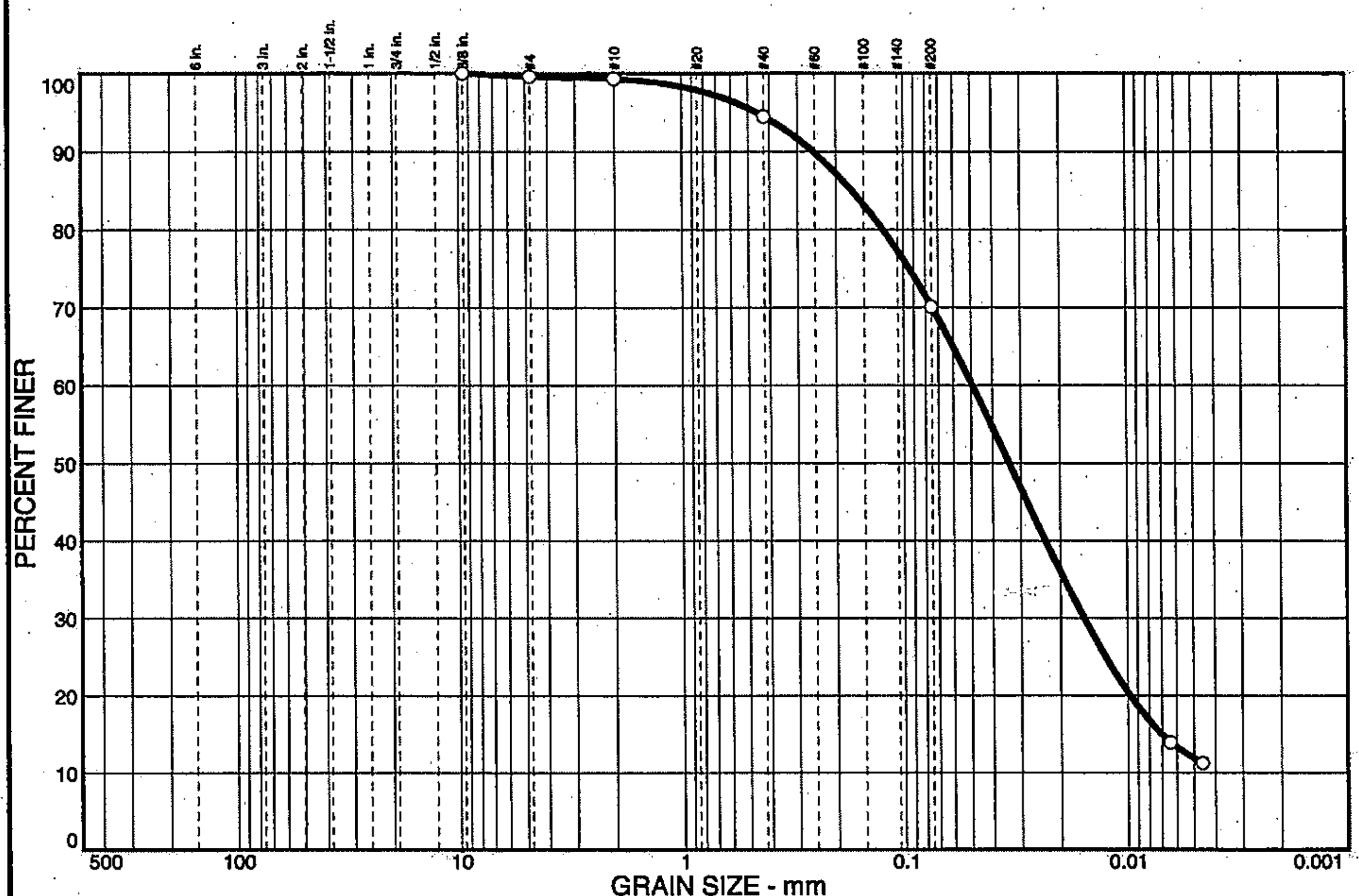
Sample No.: 4 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 7.5

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	0.4	0.3	4.8	24.4	58.3	11.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.6		
#10	99.3		
#40	94.5		
#200	70.1		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.171 D_{60}= 0.0495 D_{50}= 0.0339
D_{30}= 0.0157 D_{15}= 0.0071 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

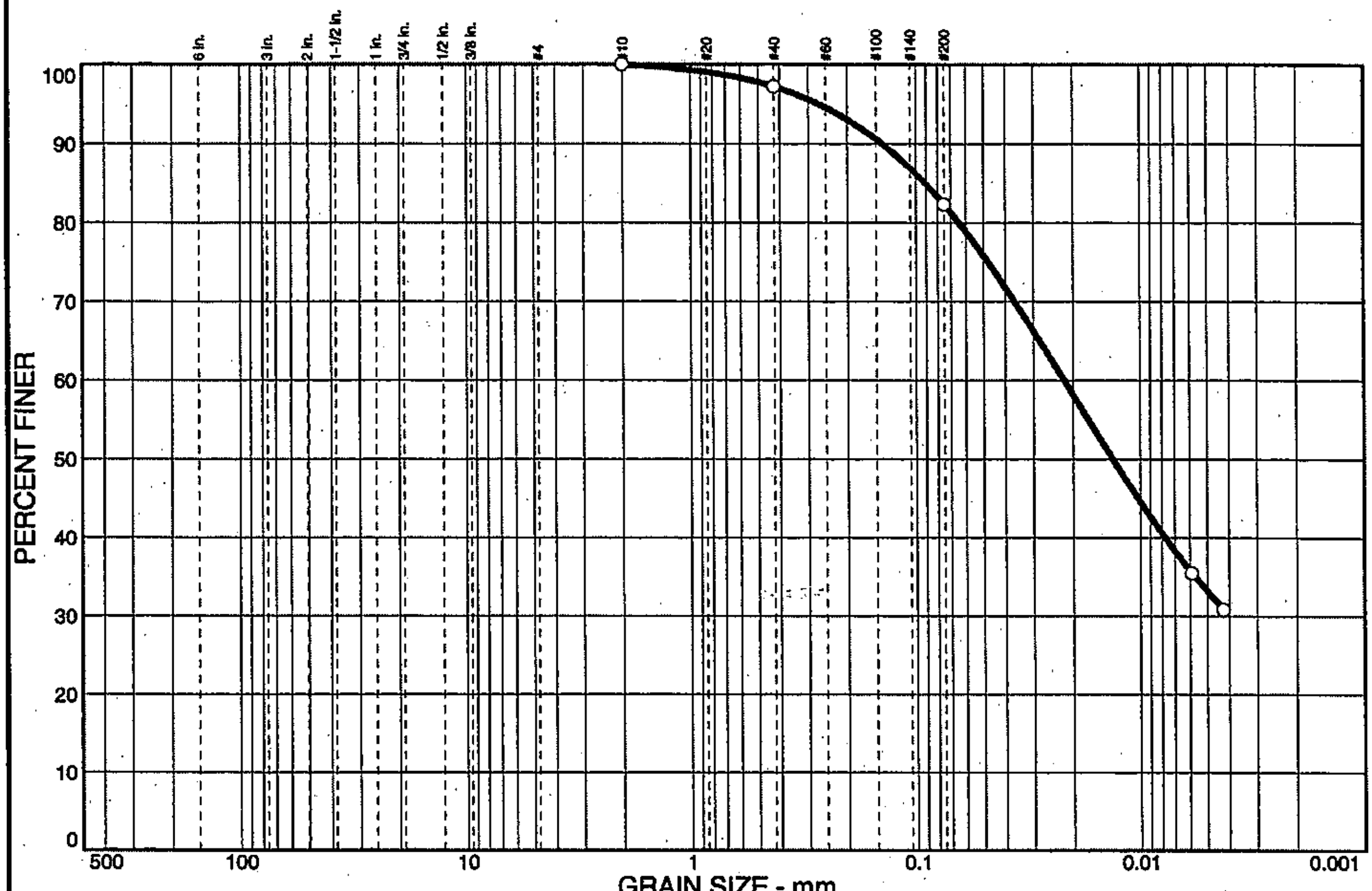
Moisture Content= 16.8%

Sample No.: 5 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 10

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	2.8	14.9	49.2	33.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	97.2		
#200	82.3		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 17 LL= 30 PI= 13

Coefficients

D_{85}= 0.0912 D_{60}= 0.0220 D_{50}= 0.0133
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

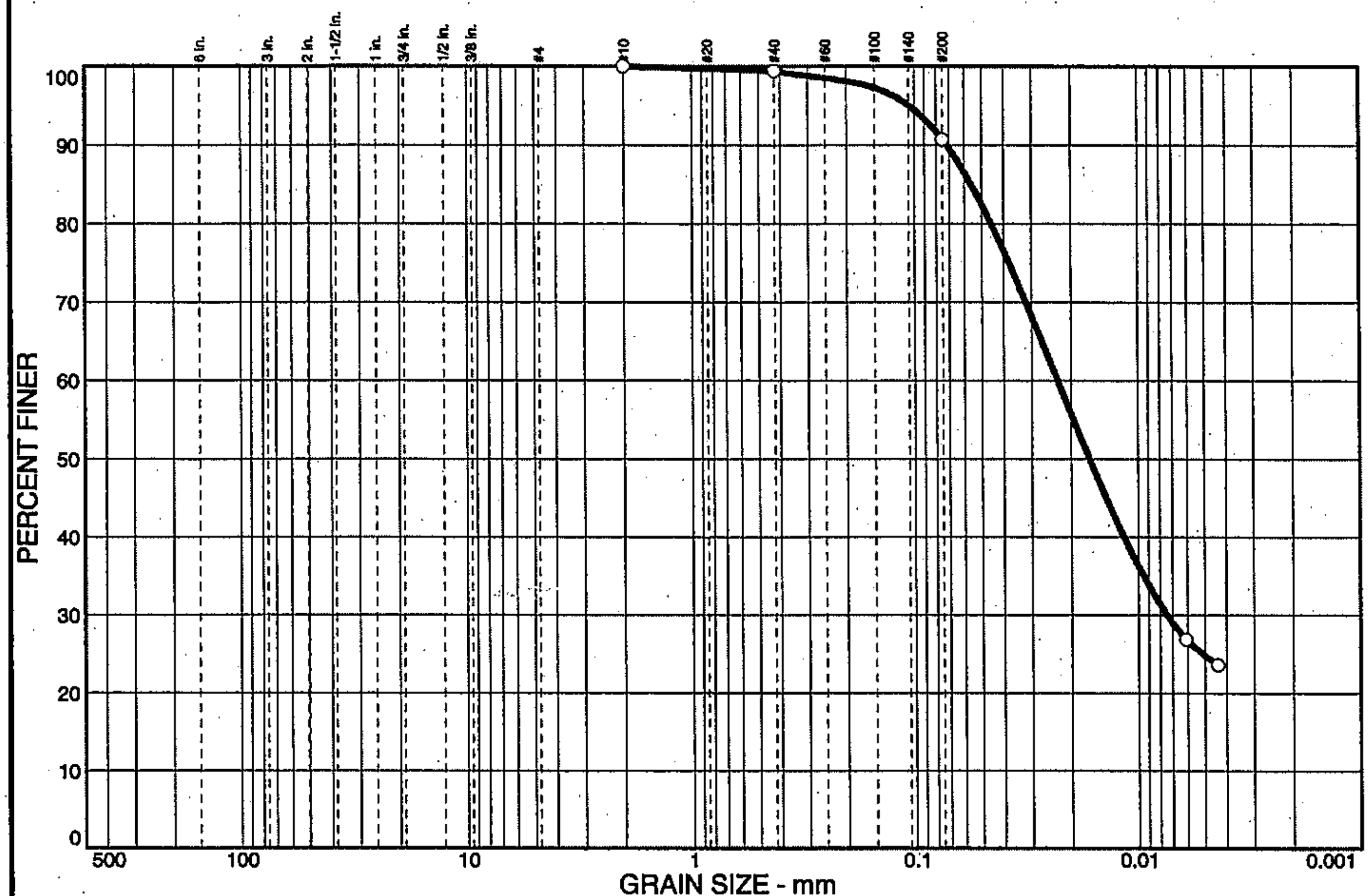
Moisture Content= 21.7%

Sample No.: 6 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 12.5

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	0.6	8.7	65.9	24.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.4		
#200	90.7		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 17 LL= 27 PI= 10

Coefficients

D_{85}= 0.0561 D_{60}= 0.0230 D_{50}= 0.0166
D_{30}= 0.0074 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(8)

Remarks

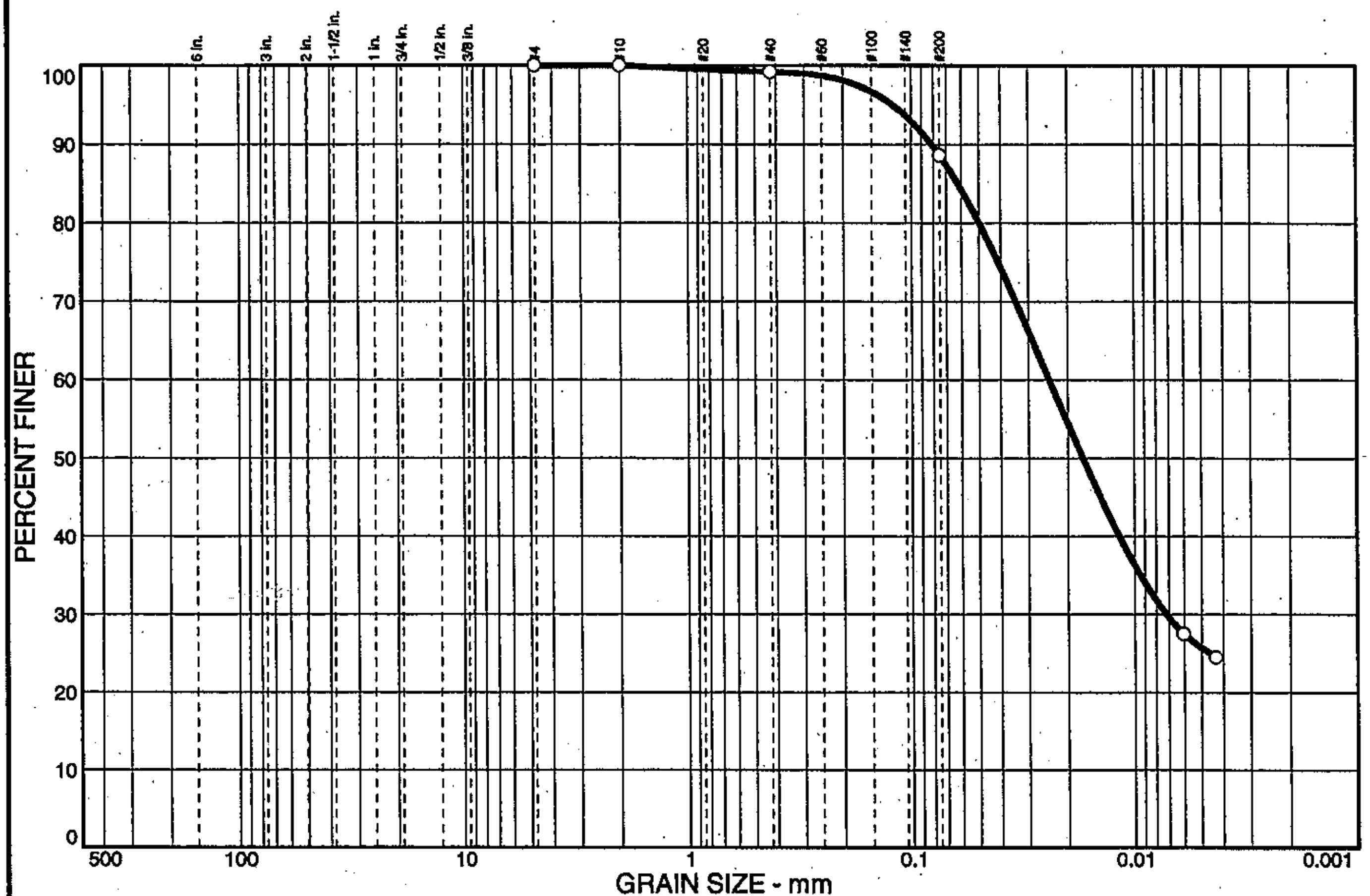
Moisture Content= 21.2%

Sample No.: 7 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 15

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	0.8	10.6	63.0	25.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#40	99.2		
#200	88.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 18 LL= 26 PI= 8

Coefficients

D_{85}= 0.0624 D_{60}= 0.0243 D_{50}= 0.0172
D_{30}= 0.0072 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(6)

Remarks

Moisture Content= 21.6%

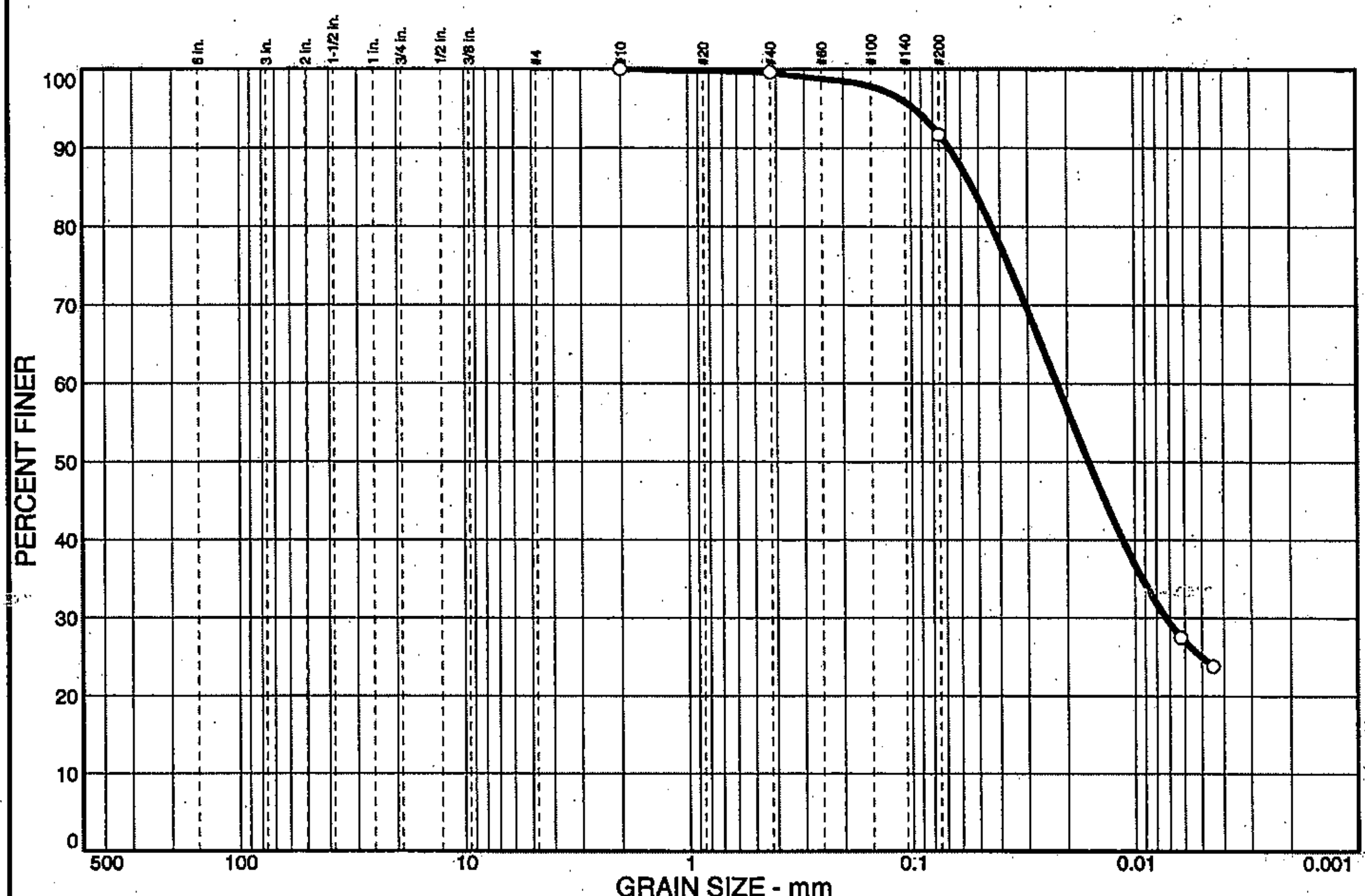
Sample No.: 8 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 17.5

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	0.4	7.9	66.8	24.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.6		
#200	91.7		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 20 LL= 27 PI= 7

Coefficients

D_{85}= 0.0534 D_{60}= 0.0223 D_{50}= 0.0161
D_{30}= 0.0073 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(5)

Remarks

Moisture Content= 25.5%

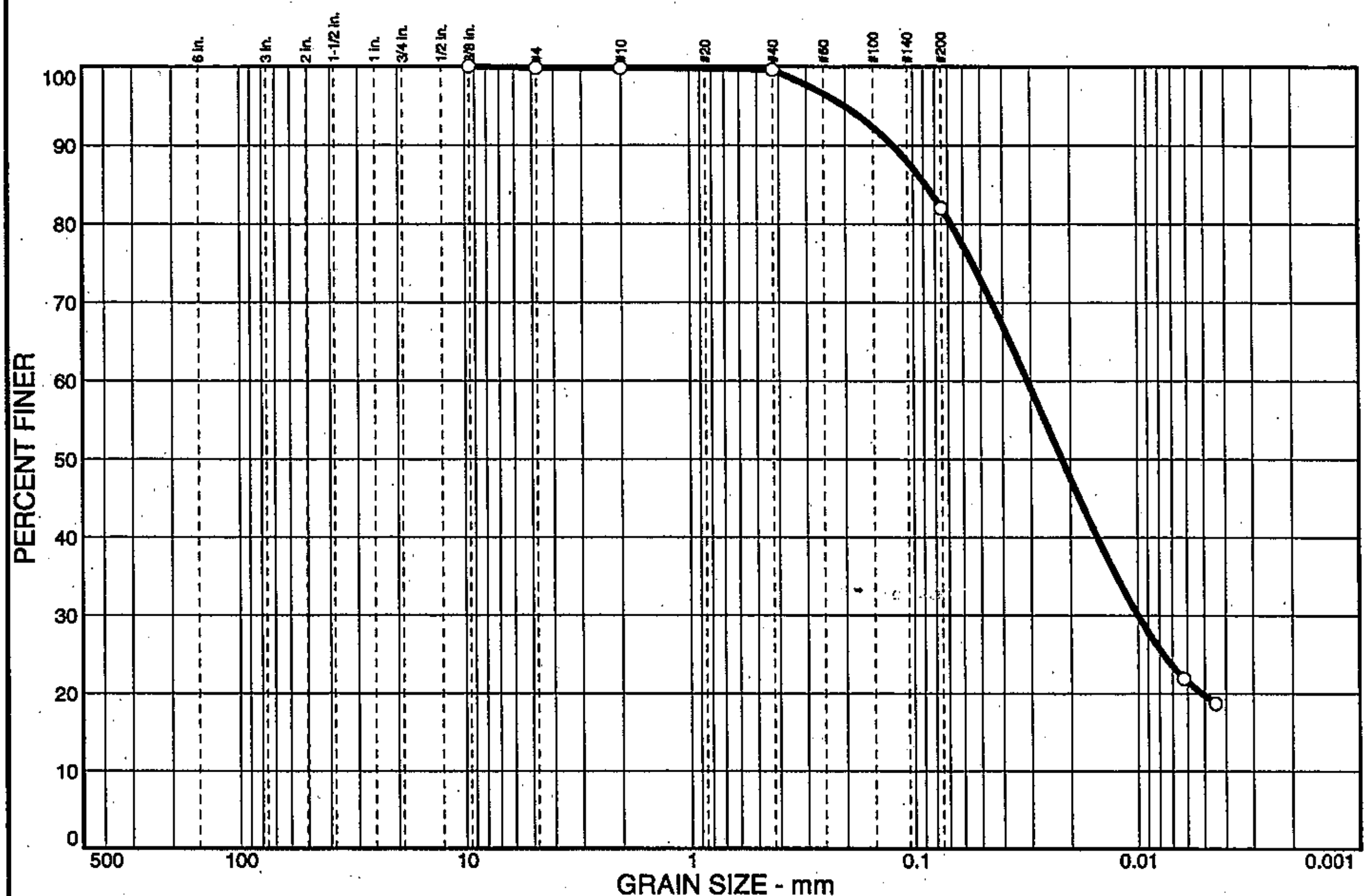
Sample No.: 9 **Source of Sample:** TR-32 **Date:** 3/25/05
Location: **Elev./Depth:** 20

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	0.2	0.0	0.2	17.6	62.3	19.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.8		
#10	99.8		
#40	99.6		
#200	82.0		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 21 LL= 29 PI= 8

Coefficients

D_{85}= 0.0882 D_{60}= 0.0312 D_{50}= 0.0219
D_{30}= 0.0100 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

Remarks

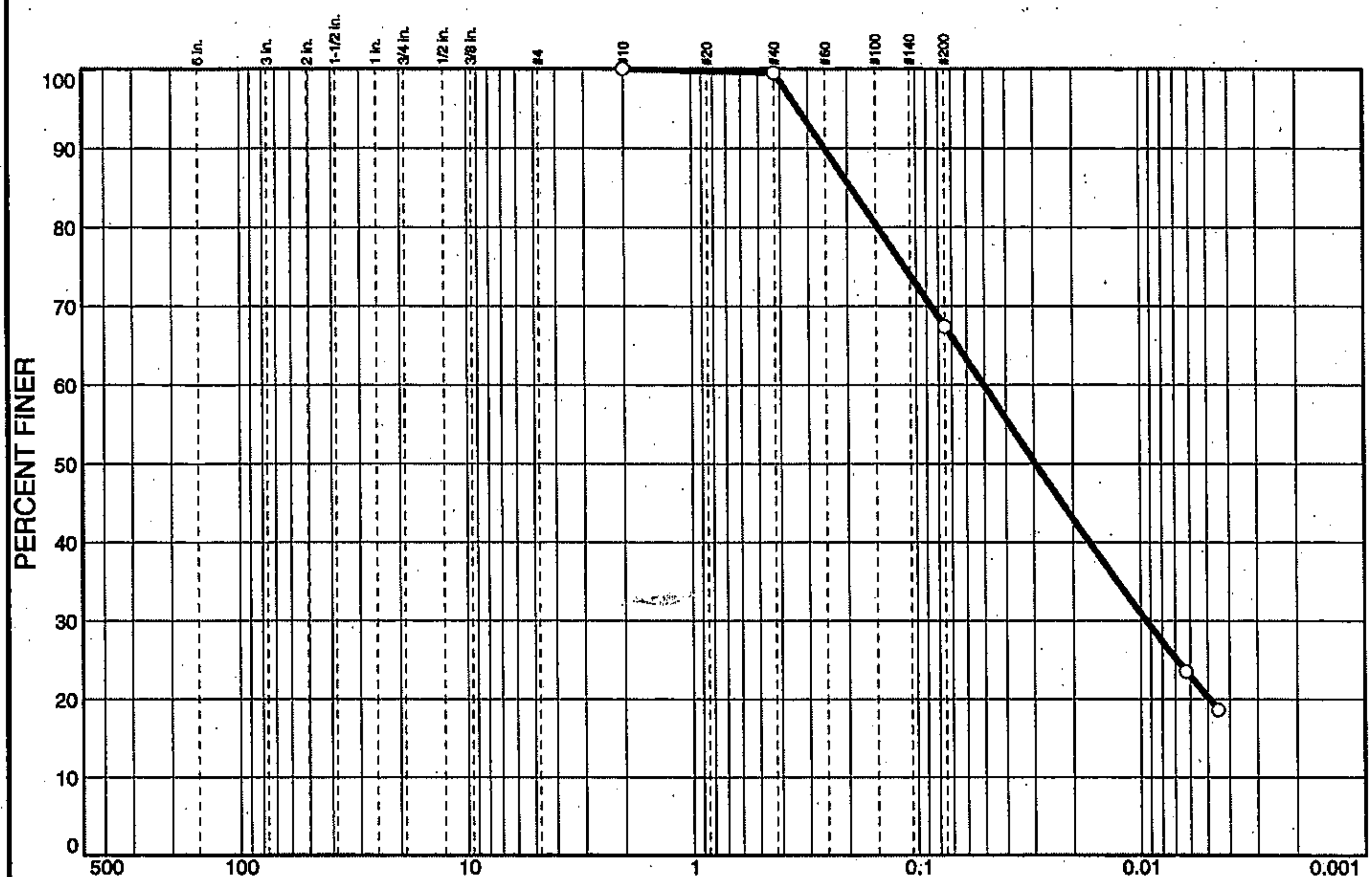
Moisture Content= 30.2%

Sample No.: 1 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 1.5

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	0.5	32.1	47.2	20.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.5		
#200	67.4		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 18 LL= 23 PI= 5

Coefficients

D_{85}= 0.193 D_{60}= 0.0505 D_{50}= 0.0295
D_{30}= 0.0094 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 26.1%

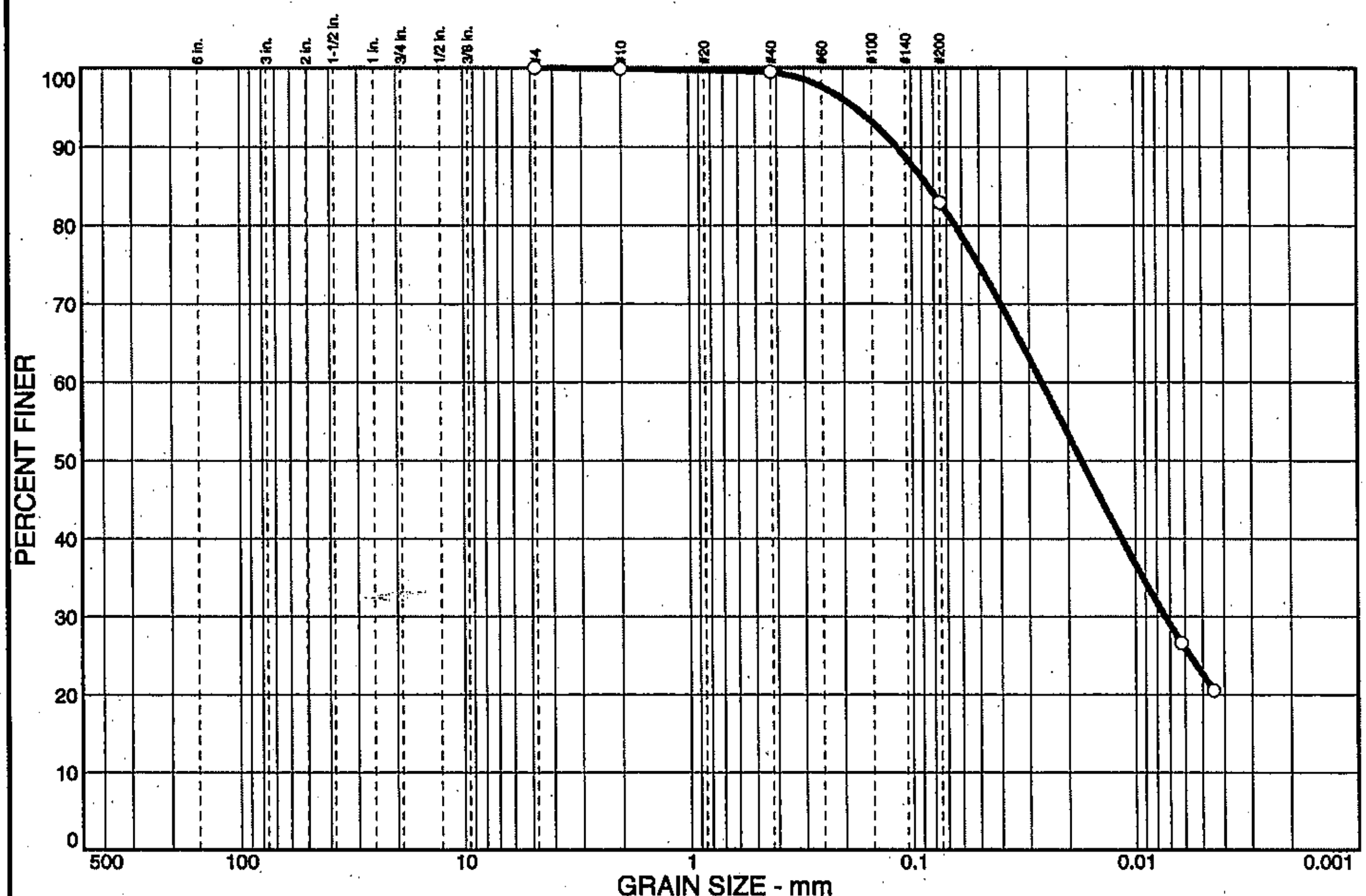
Sample No.: 2 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 4.0

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	0.1	0.4	16.6	60.4	22.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#40	99.5		
#200	82.9		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 19 LL= 25 PI= 6

Coefficients

D_{85}= 0.0845 D_{60}= 0.0264 D_{50}= 0.0175
D_{30}= 0.0074 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 27.3%

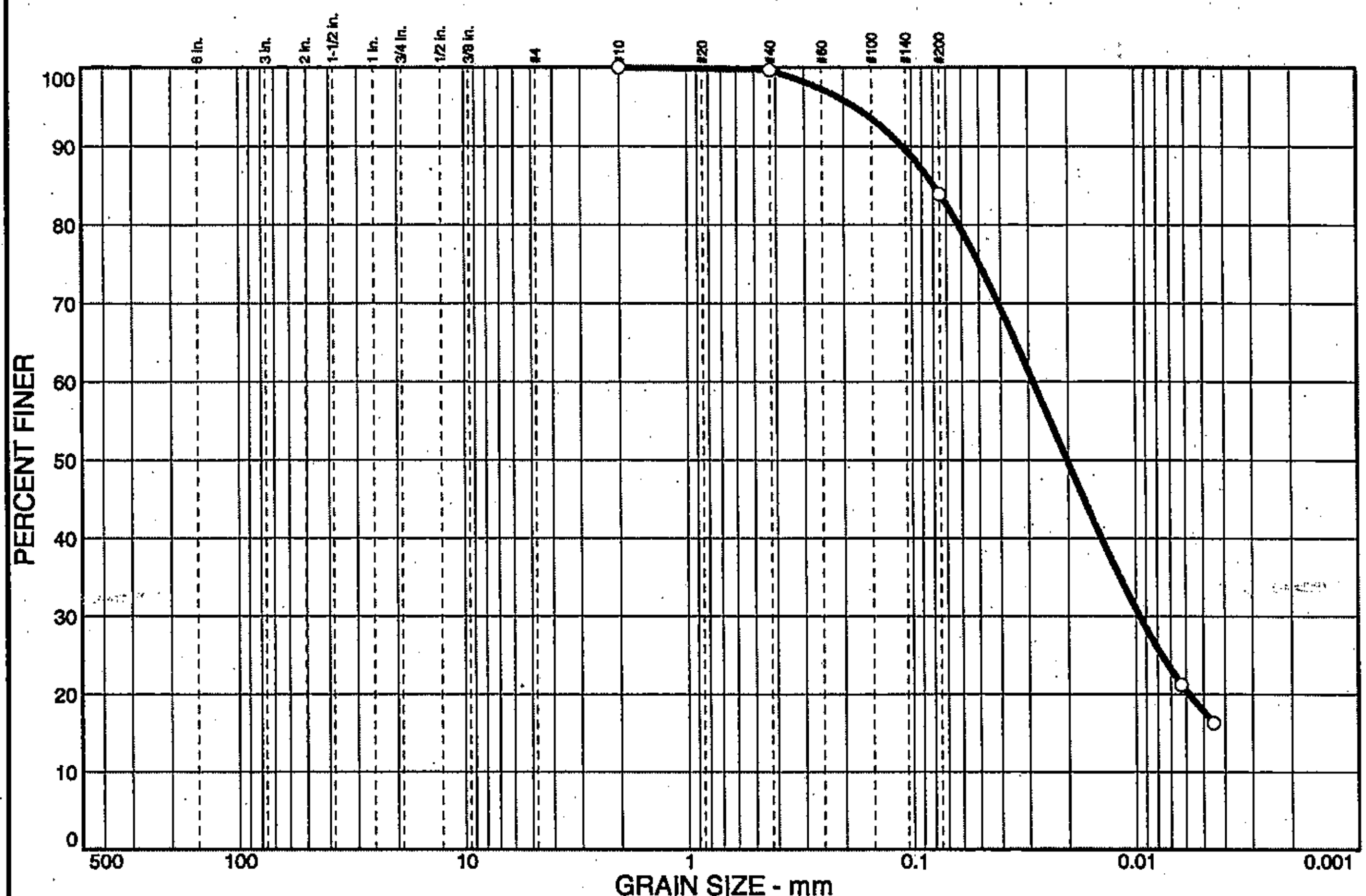
Sample No.: 3 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 6.5

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	0.3	15.8	66.1	17.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	83.9		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.0796 D_{60}= 0.0287 D_{50}= 0.0203
D_{30}= 0.0096 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

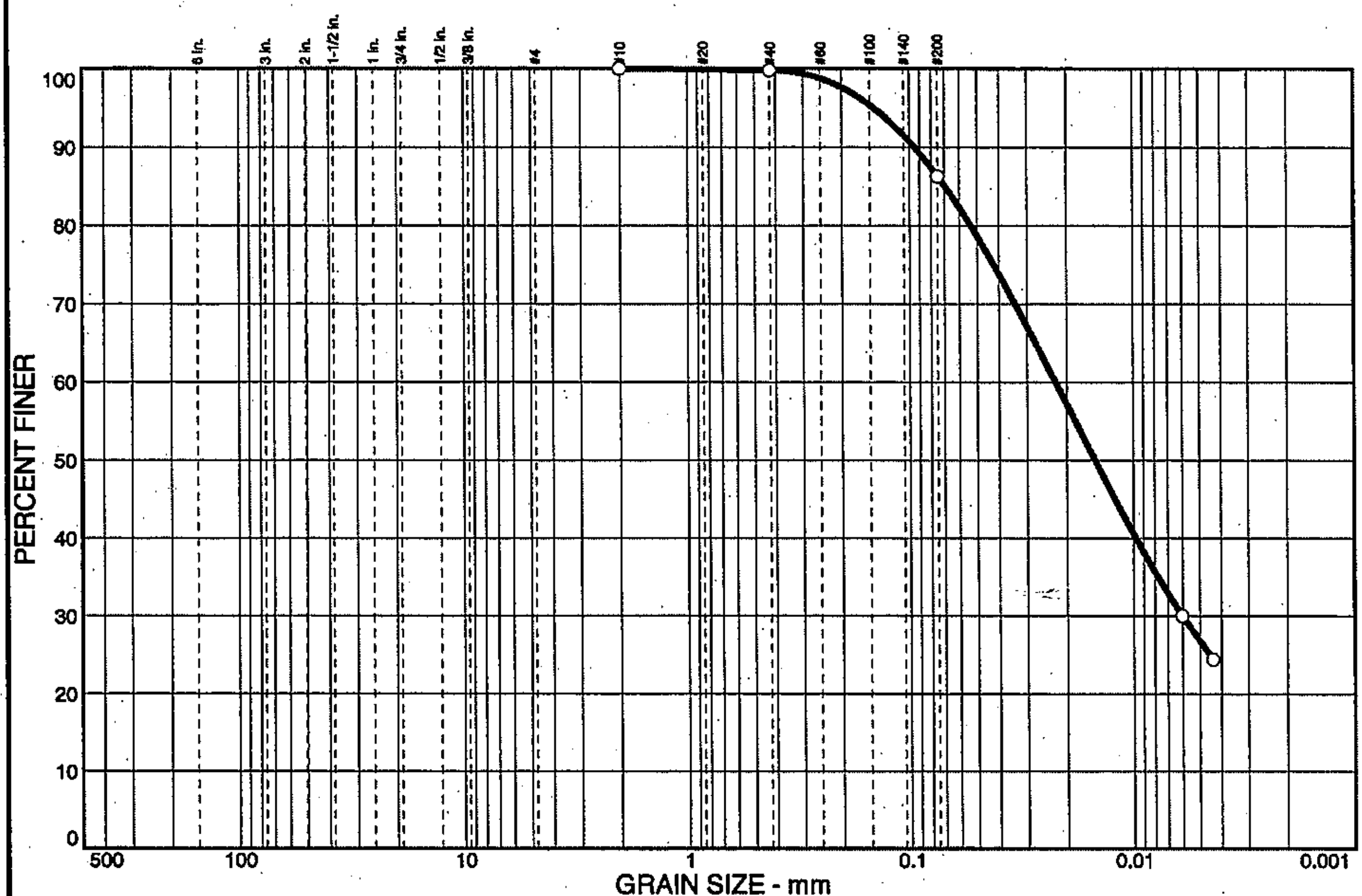
Moisture Content= 32.3%

Sample No.: 4 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 9

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	0.2	13.5	59.6	26.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.8		
#200	86.3		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 0.0696 D_{60}= 0.0225 D_{50}= 0.0150
D_{30}= 0.0060 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(6)

Remarks

Moisture Content= 32.8%

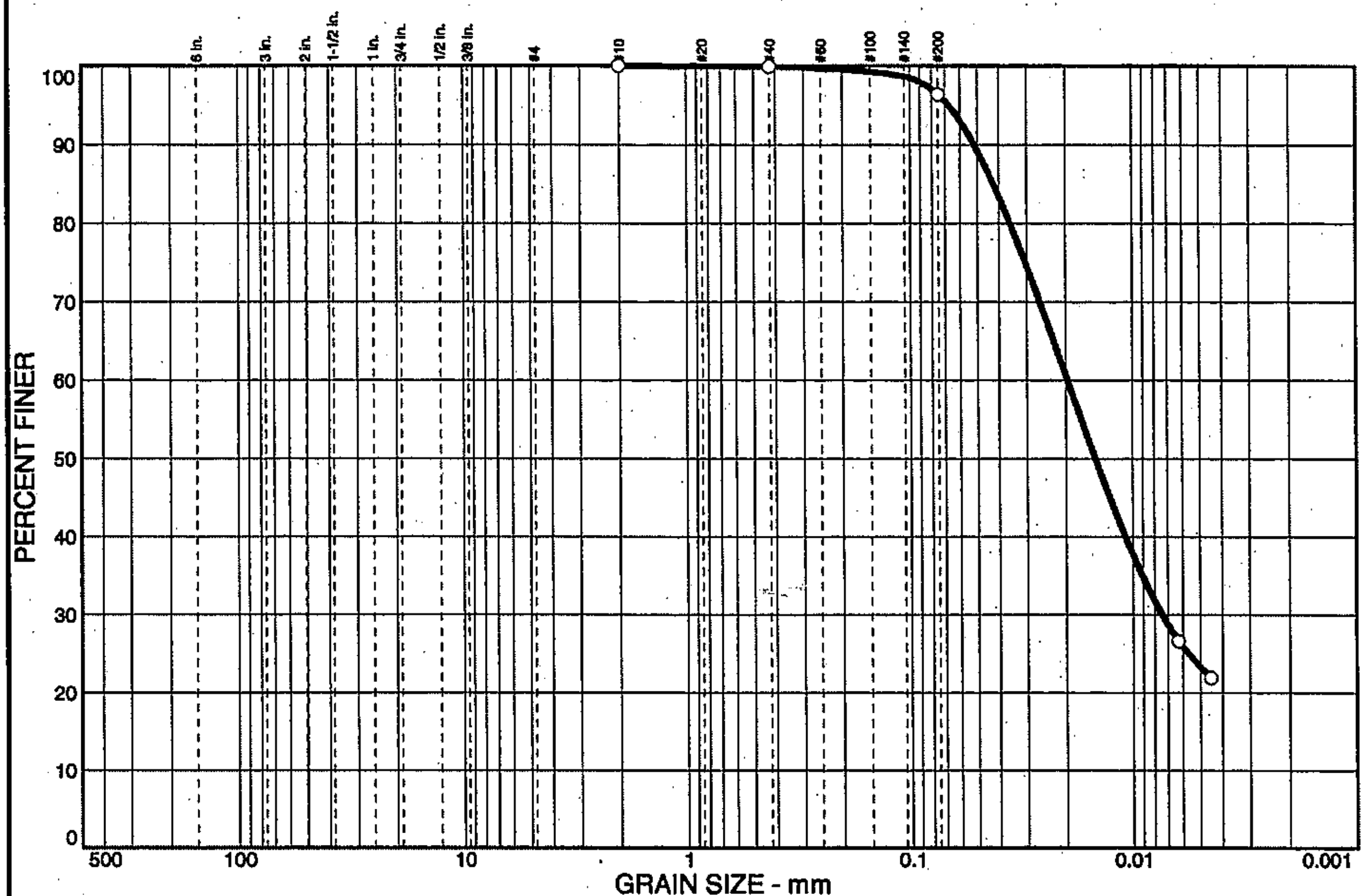
Sample No.: 5 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 11.5

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	0.1	3.5	73.2	23.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	96.4		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 21 LL= 28 PI= 7

Coefficients

D_{85}= 0.0425 D_{60}= 0.0197 D_{50}= 0.0148
D_{30}= 0.0074 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(6)

Remarks

Moisture Content= 34.3%

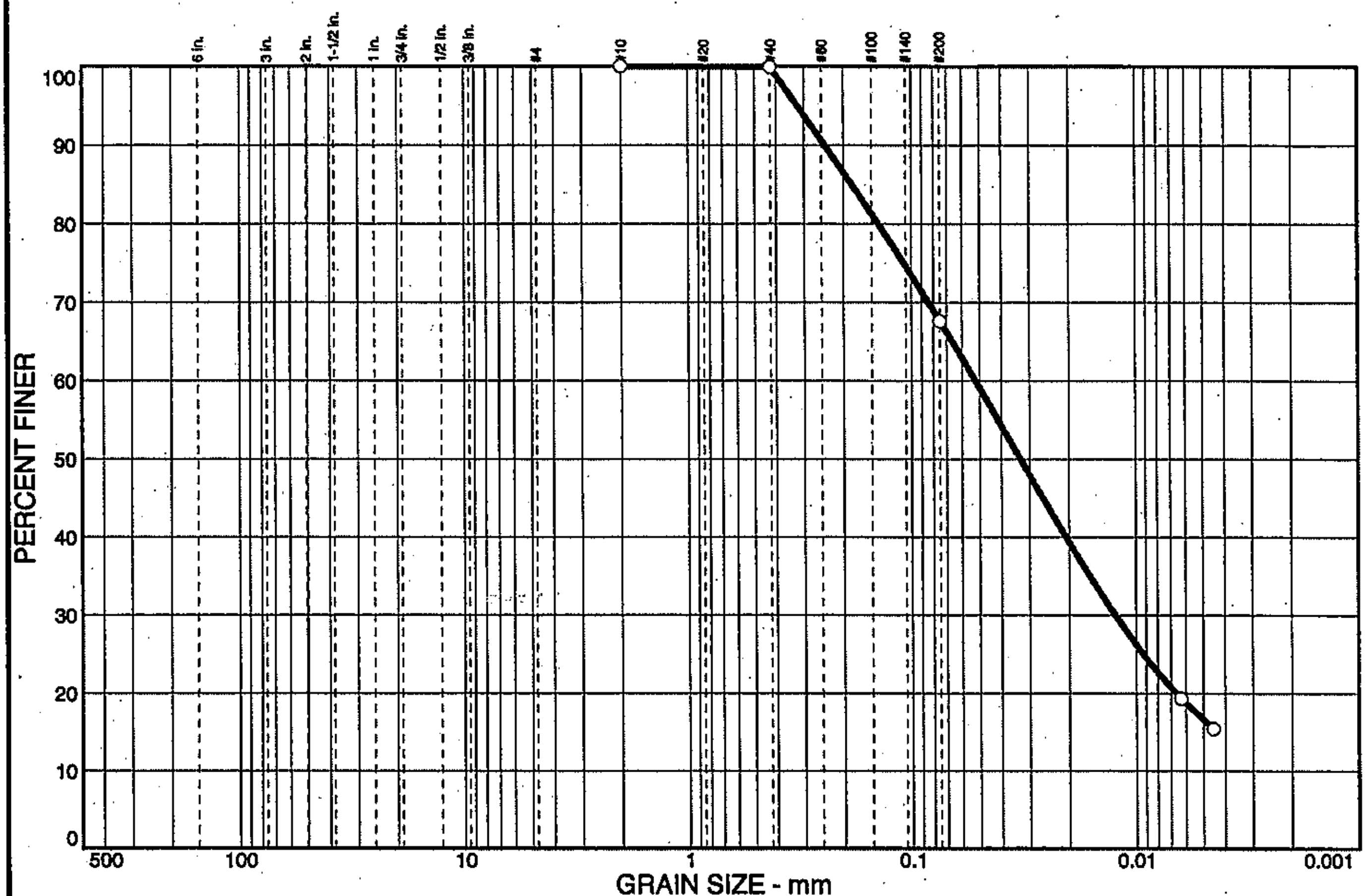
Sample No.: 6 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 14

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	0.1	32.3	51.0	16.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	67.6		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= 18 LL= 20 PI= 2

Coefficients

D_{85}= 0.184 D_{60}= 0.0524 D_{50}= 0.0331
D_{30}= 0.0124 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content= 29.4%

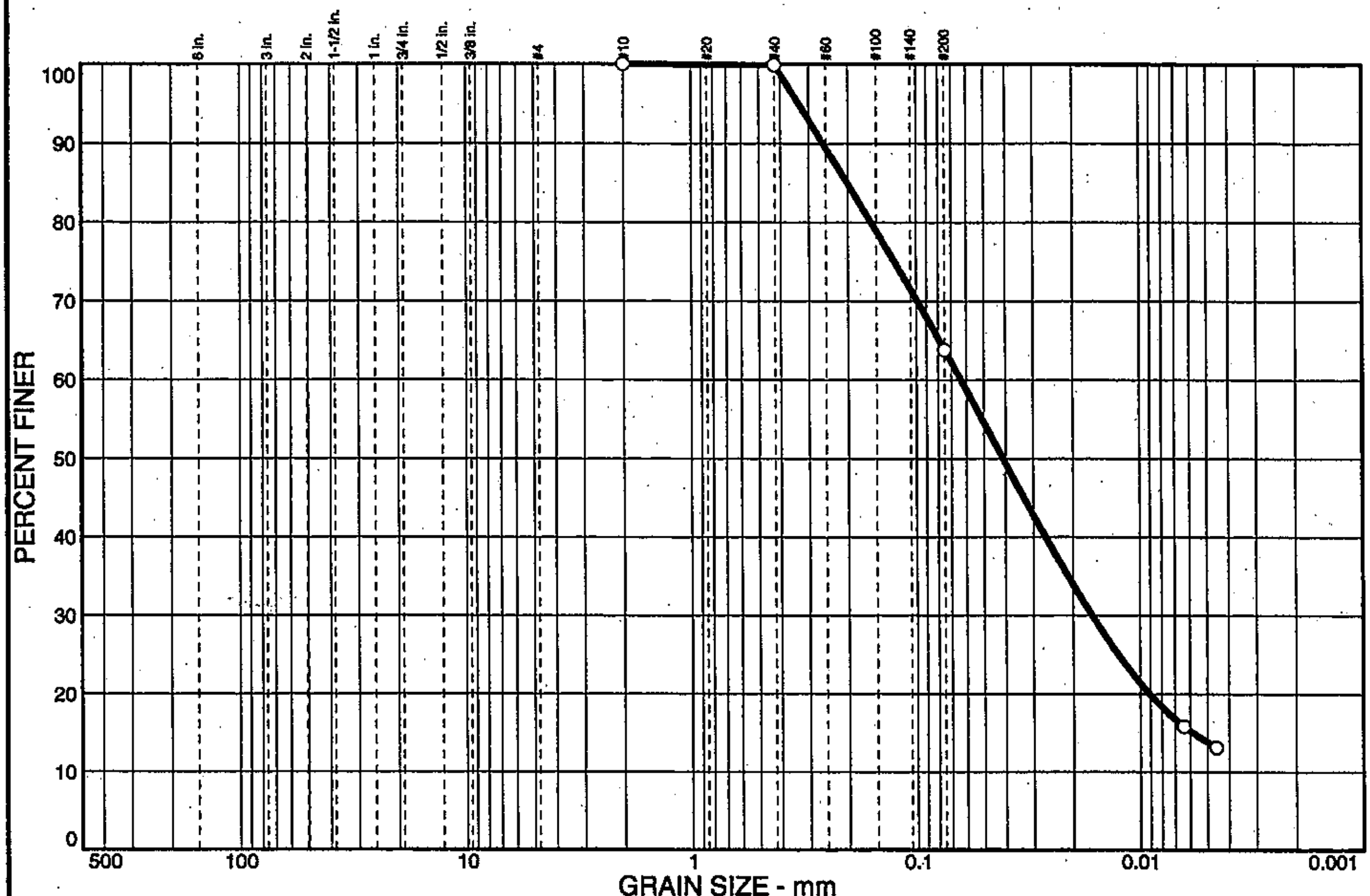
Sample No.: 7 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 16.5

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	0.1	36.1	50.0	13.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.9		
#200	63.8		

* (no specification provided)

Soil Description

Sandy silt

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.202 D_{60}= 0.0635 D_{50}= 0.0413
D_{30}= 0.0165 D_{15}= 0.0058 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content= 28.4%

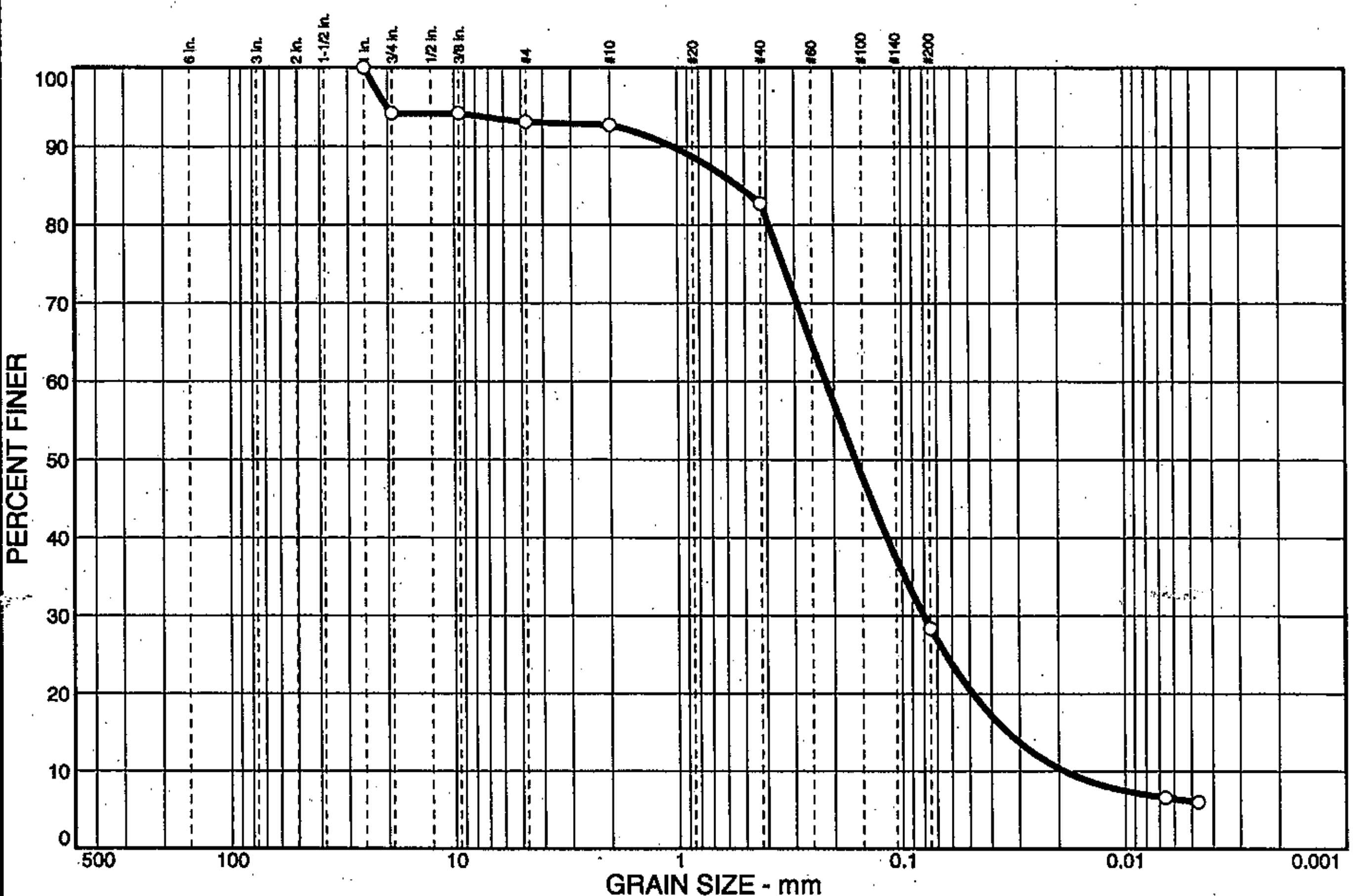
Sample No.: 8 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 19.0

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	5.8	1.1	0.4	10.0	54.3	22.3	6.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00 in.	100.0		
0.75 in.	94.2		
0.375 in.	94.2		
#4	93.1		
#10	92.7		
#40	82.7		
#200	28.4		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.483 D_{60}= 0.194 D_{50}= 0.146
D_{30}= 0.0795 D_{15}= 0.0381 D_{10}= 0.0230
C_u= 8.44 C_c= 1.41

Classification

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

Remarks

Moisture Content= 21.8%

Sample No.: 9 **Source of Sample:** TR-33 **Date:** 3/21/05
Location: **Elev./Depth:** 21.5

DLZ

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

Project No: 0121-3070.03 **Figure**

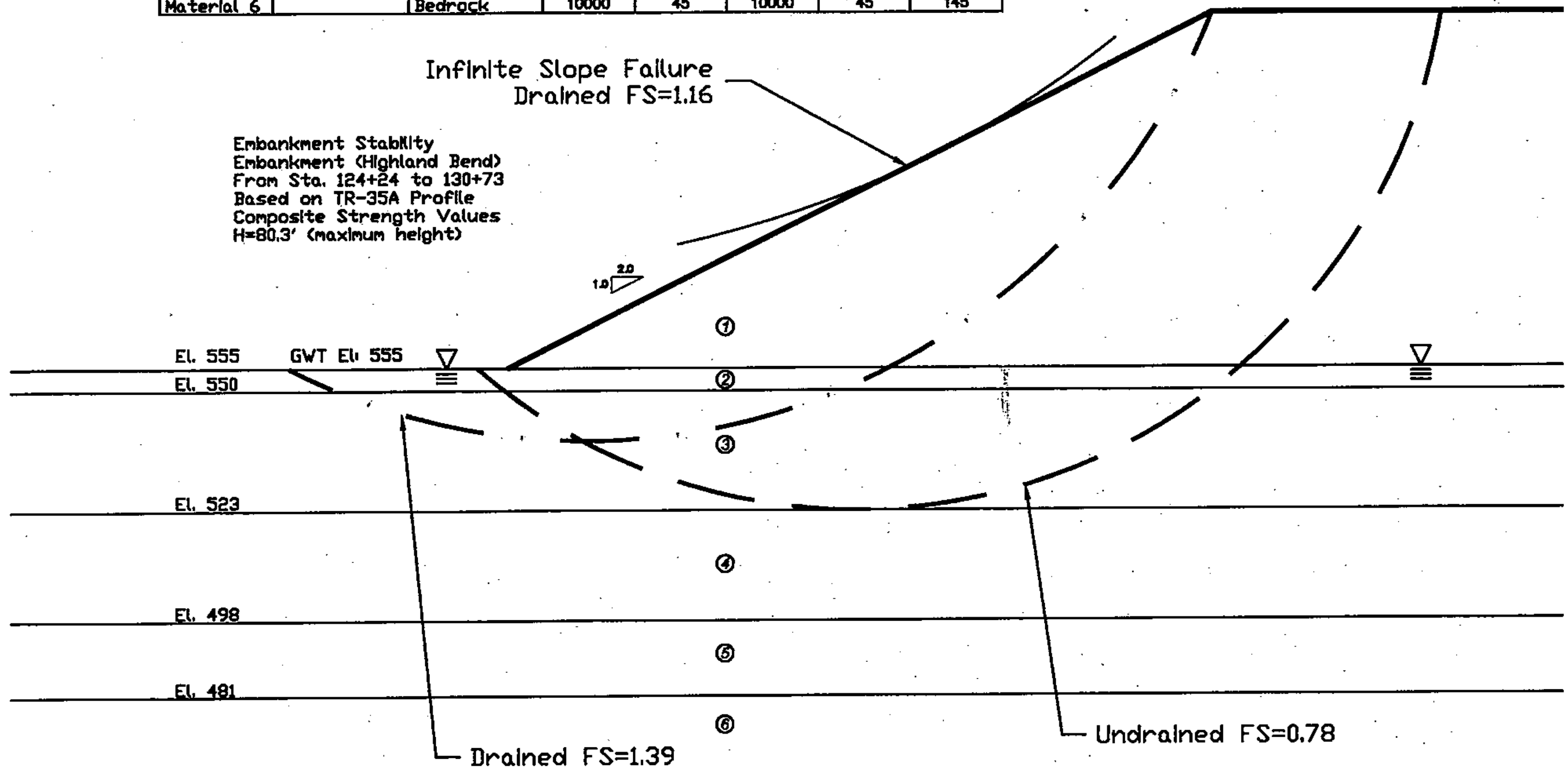
SECTION 4

DLZ

APPENDIX IV

Slope Stability Analyses
Settlement Calculations
Driven-Pile Analyses
Forward Abutment Retaining Wall Analyses

Material	Consistency	Soil Type	Undrained c (psf)	Undrained ϕ (deg)	Drained c' (psf)	Drained ϕ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	Stiff	Clay	1700	0	0	30	125
Material 3	Stiff	Silt	900	0	0	29	120
Material 4	Stiff	Silty Clay	2700	0	0	29	120
Material 5	M. Dense	Sandy Silt	0	30	0	30	115
Material 6		Bedrock	10000	45	10000	45	145



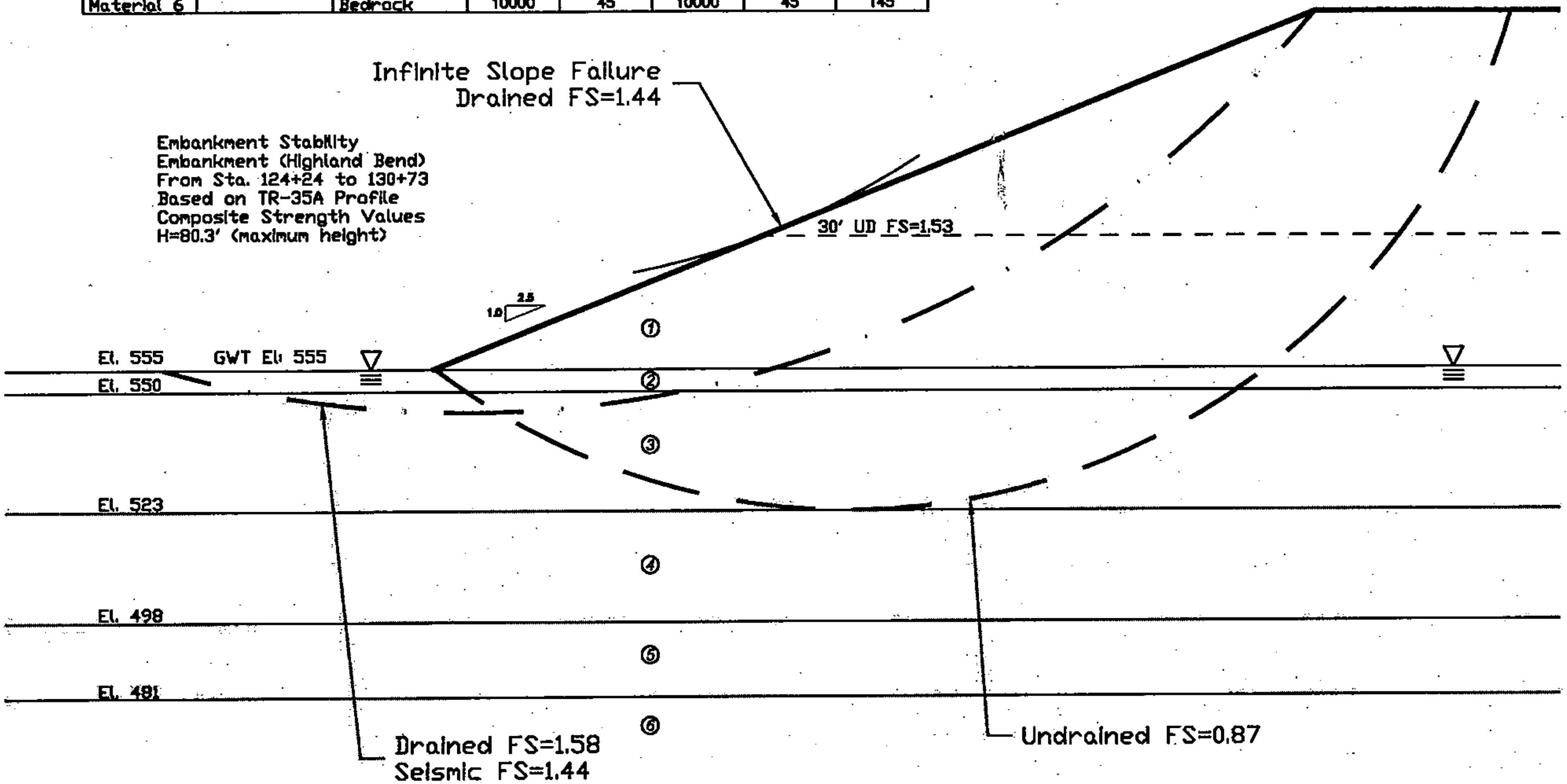
EMBANKMENT FILL MATERIAL; ϕ'=30 DEG

Refer to Report of Highland Bend Roadway Embankments, dated 8-2-07

Highland Bend Embankments Station 124+24 to 130+73 Undrained and Drained Analyses
EMBANKMENT STABILITY ANALYSIS
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 7/3/07

Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	Stiff	Clay	1700	0	0	30	125
Material 3	Stiff	Silt	900	0	0	29	120
Material 4	Stiff	Silty Clay	2700	0	0	29	120
Material 5	M. Dense	Sandy Silt	0	30	0	30	115
Material 6		Bedrock	10000	45	10000	45	145



EMBANKMENT FILL MATERIAL; φ'=30 DEG

Highland Bend Embankments
Station 124+24 to 130+73
Undrained and Drained Analyses

EMBANKMENT STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 7/3/07

Refer to Report of Highland Bend Roadway Embankments, dated 8-2-07

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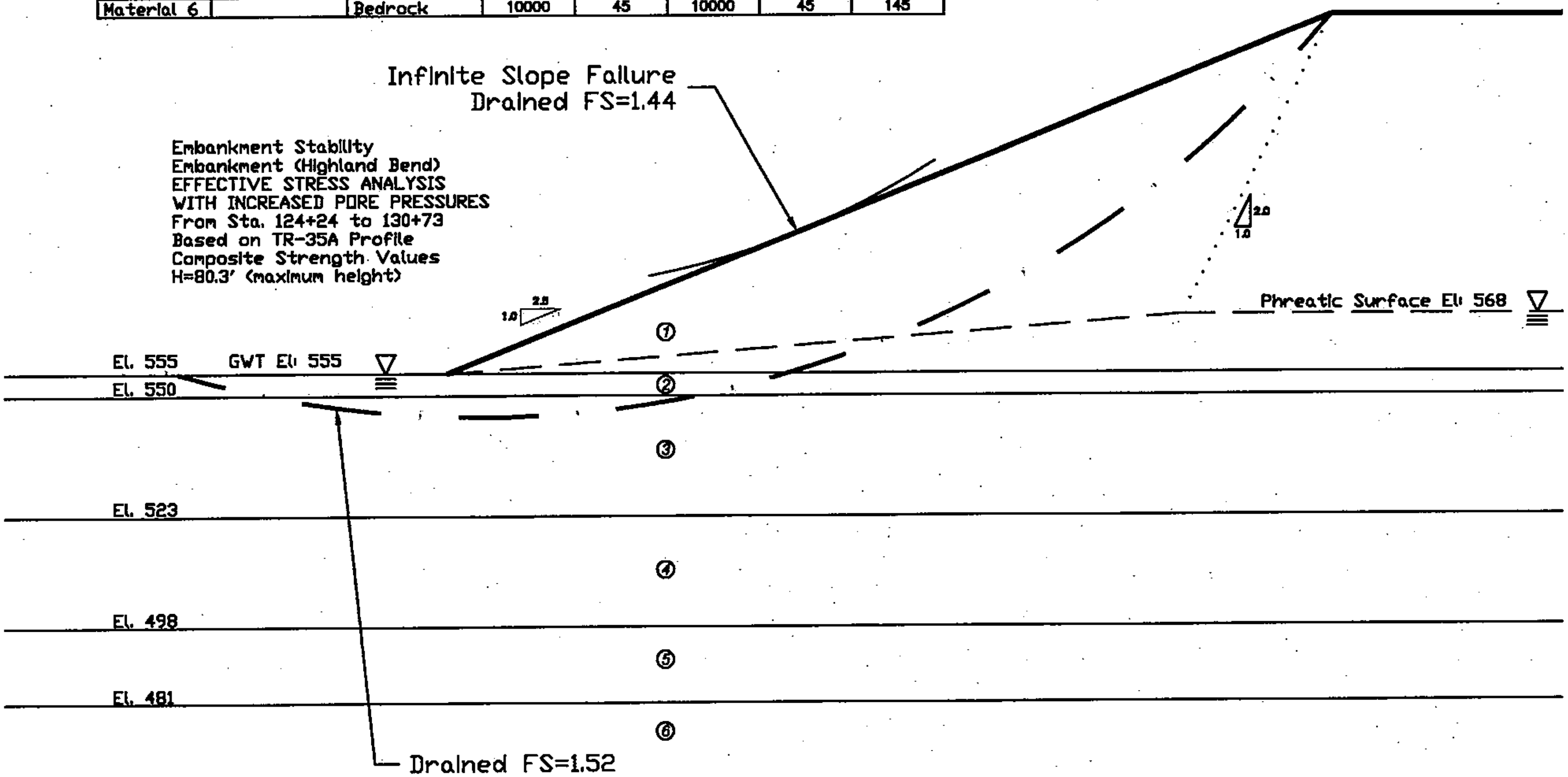
Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	Stiff	Clay	1700	0	0	30	125
Material 3	Stiff	Silt	900	0	0	29	120
Material 4	Stiff	Silty Clay	2700	0	0	29	120
Material 5	M. Dense	Sandy Silt	0	30	0	30	115
Material 6		Bedrock	10000	45	10000	45	145



EMBANKMENT FILL MATERIAL; φ'=30 DEG

Refer to Report of Highland Bend Roadway Embankments, dated 8-2-07

Highland Bend Embankments
Station 124+24 to 130+73
STAGED CONSTRUCTION ANALYSIS

EFFECTIVE STRESS ANALYSIS WITH INCREASED PORE WATER PRESSURES
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 7/3/07

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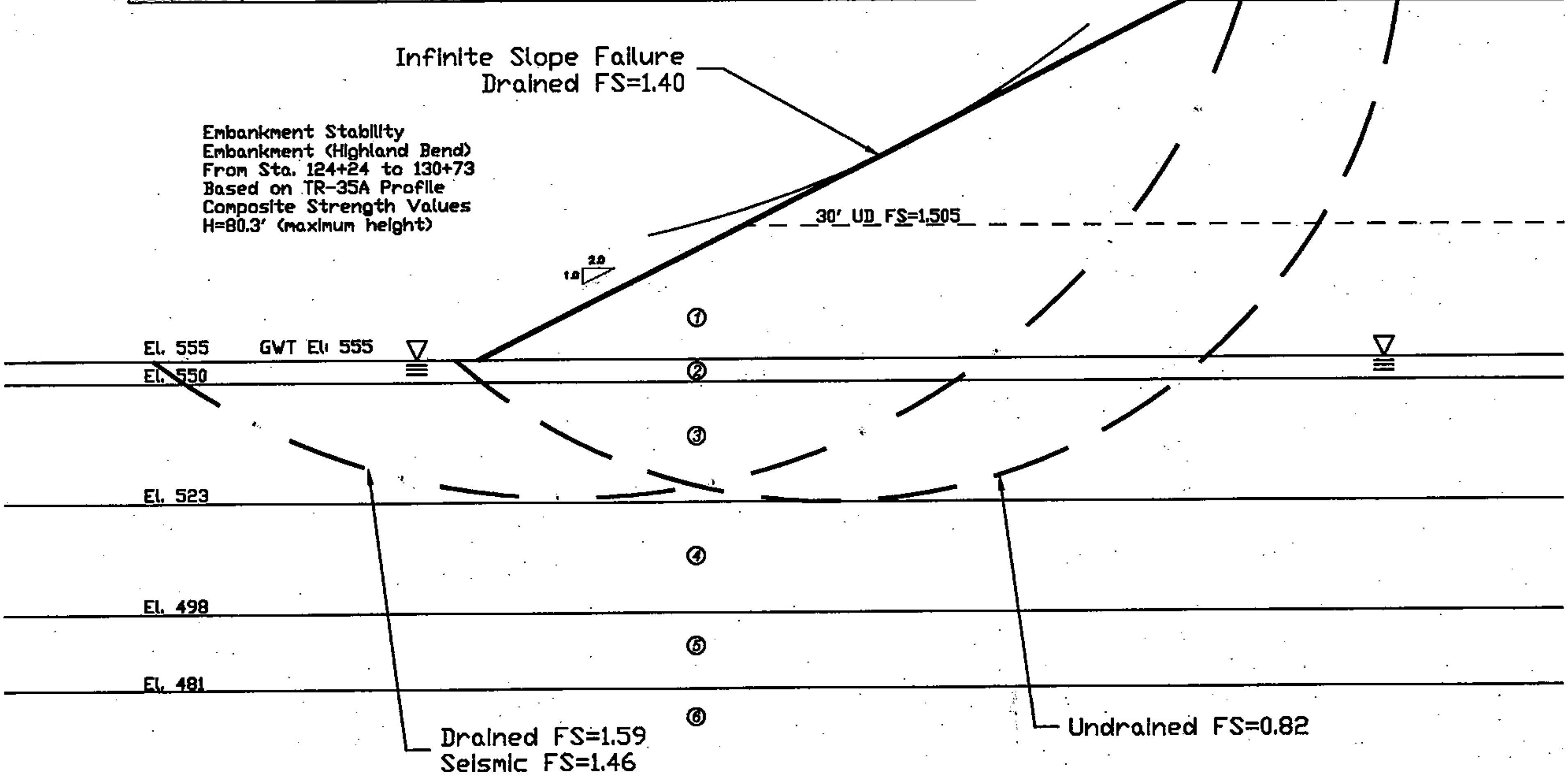
Material	Consistency	Soil Type	Undrained c (psf)	Undrained ϕ (deg)	Drained c' (psf)	Drained ϕ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	35	0	35	120
Material 2	Stiff	Clay	1700	0	0	30	125
Material 3	Stiff	Silt	900	0	0	29	120
Material 4	Stiff	Silty Clay	2700	0	0	29	120
Material 5	M. Dense	Sandy Silt	0	30	0	30	115
Material 6		Bedrock	10000	45	10000	45	145



EMBANKMENT FILL MATERIAL; ϕ'=35 DEG

Highland Bend Embankments
Station 124+24 to 130+73
Undrained and Drained Analyses

EMBANKMENT STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 7/3/07

Refer to Report of Highland Bend Roadway Embankments, dated 8-2-07

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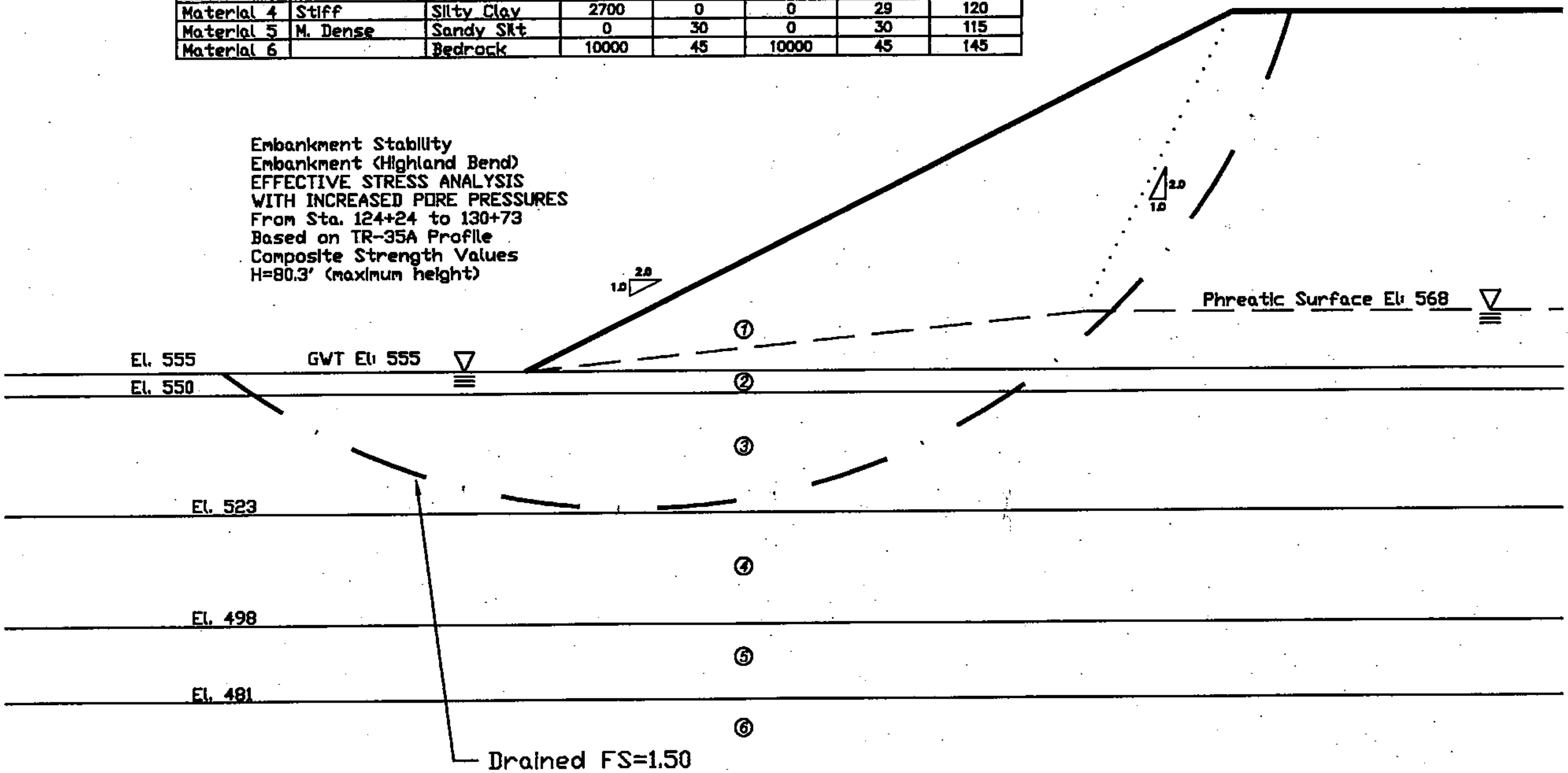
Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	35	0	35	120
Material 2	Stiff	Clay	1700	0	0	30	125
Material 3	Stiff	Silt	900	0	0	29	120
Material 4	Stiff	Silty Clay	2700	0	0	29	120
Material 5	M. Dense	Sandy Silt	0	30	0	30	115
Material 6		Bedrock	10000	45	10000	45	145

Embankment Stability
Embankment (Highland Bend)
EFFECTIVE STRESS ANALYSIS
WITH INCREASED PORE PRESSURES
From Sta. 124+24 to 130+73
Based on TR-35A Profile
Composite Strength Values
H=80.3' (maximum height)



EMBANKMENT FILL MATERIAL; φ'=35 DEG

Refer to Report of Highland Bend Roadway Embankments, dated 8-2-07

Highland Bend Embankments
Station 124+24 to 130+73
STAGED CONSTRUCTION ANALYSIS

EFFECTIVE STRESS ANALYSIS WITH INCREASED PORE WATER PRESSURES
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 7/3/07

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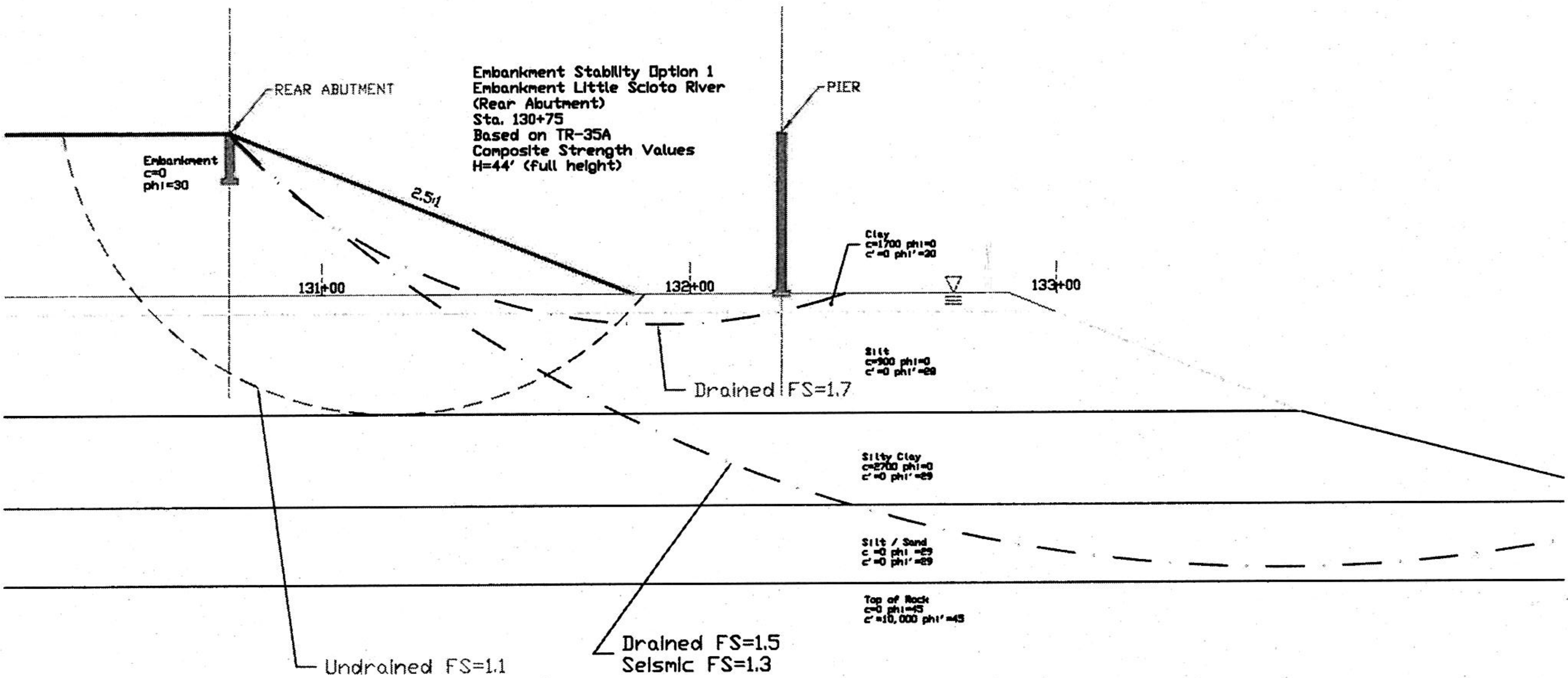

Preliminary Approach Embankment Analyses

HIGHLAND BEND SLOPE STABILITY (OPTION 1) ABUTMENT BEARING STA. 130+75 PIER BEARING STA. 132+25
STABILITY ANALYSIS SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 06-06-06

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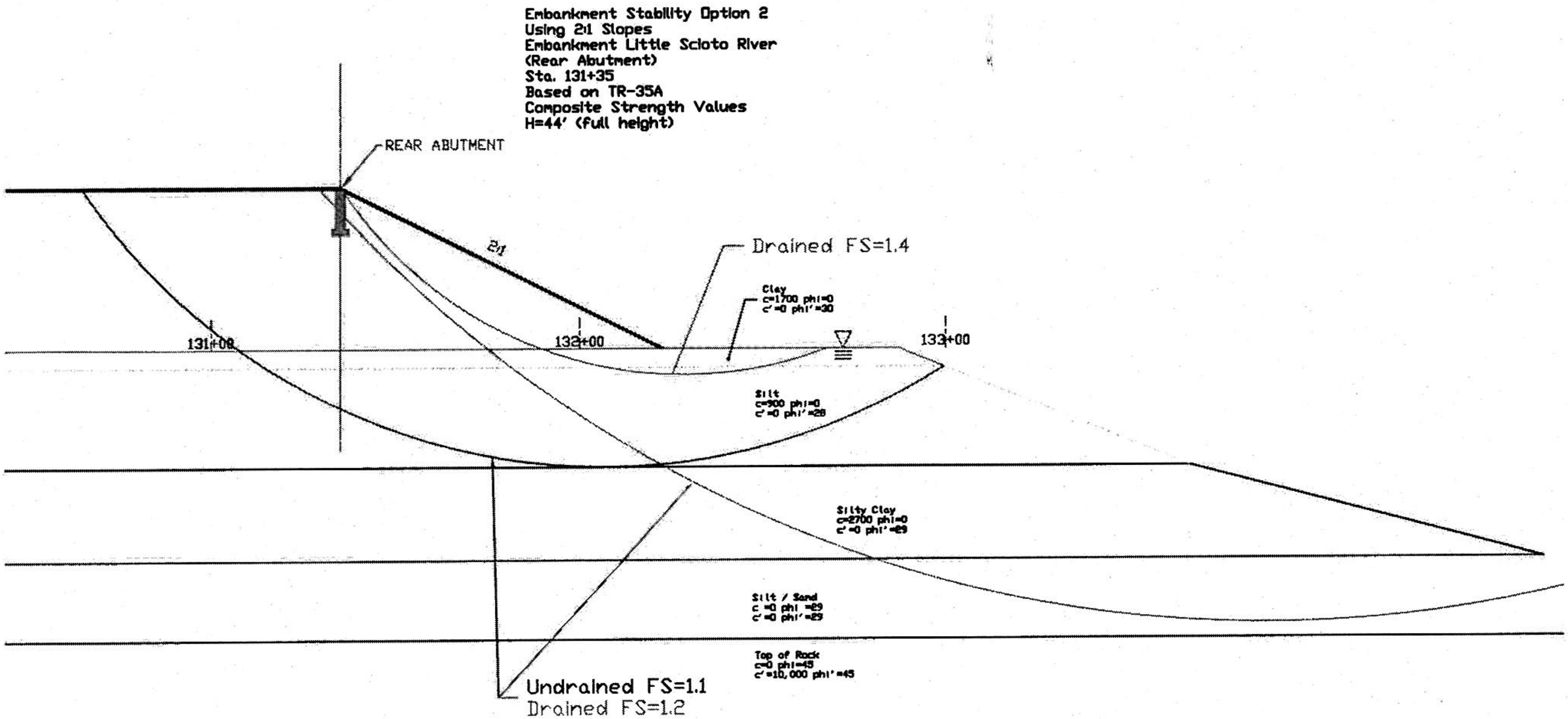

Preliminary Approach Embankment Analyses

HIGHLAND BEND SLOPE STABILITY (OPTION 2)
ABUTMENT BEARING STA. 131+35

STABILITY ANALYSIS
2.0: 1 SLOPES
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 06-06-06

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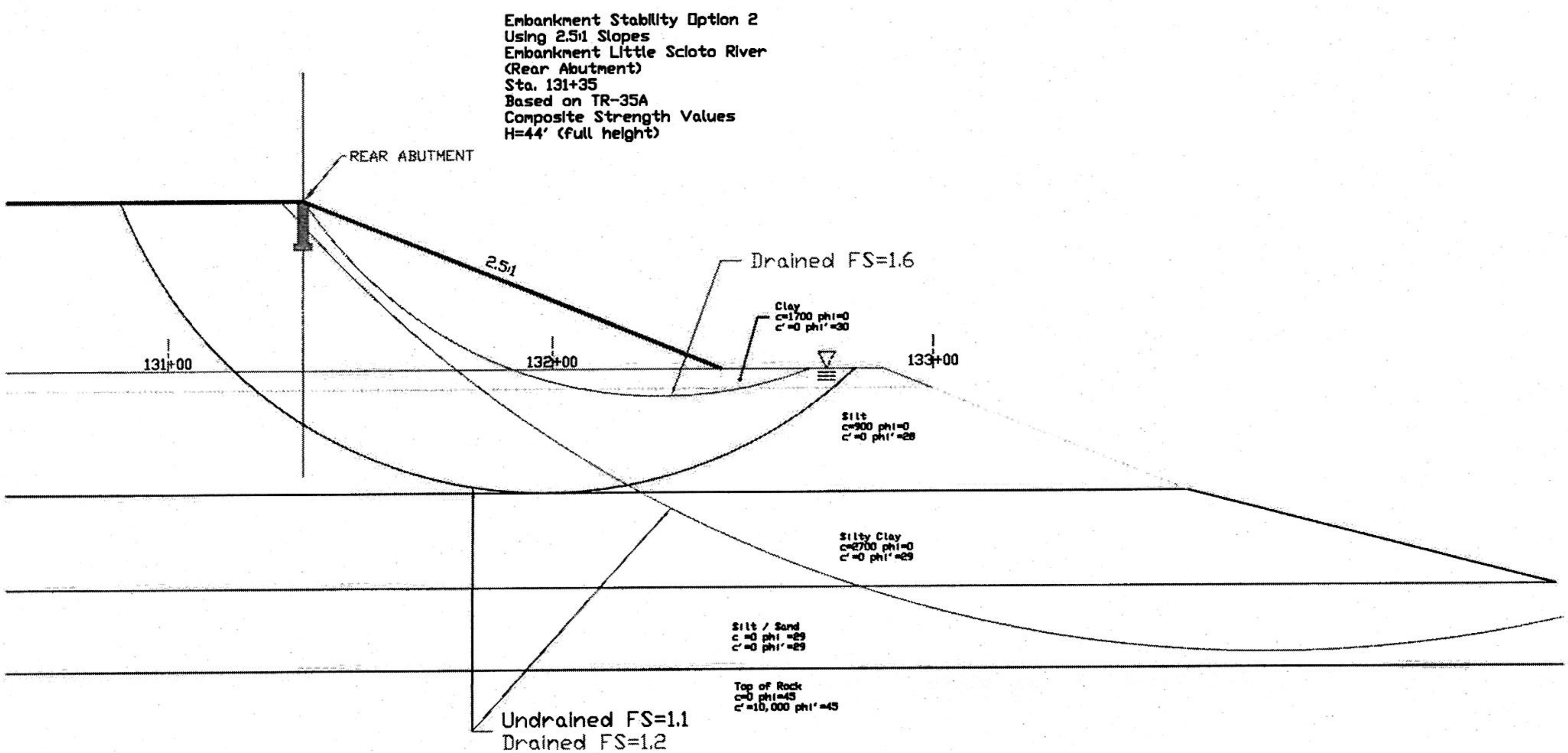

Preliminary Approach Embankment Analyses

HIGHLAND BEND SLOPE STABILITY (OPTION 2) ABUTMENT BEARING STA. 131+35		
STABILITY ANALYSIS 2.5: 1 SLOPES SCI-823-0.00		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 06-06-06

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DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: Transystems Corp
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Settlement / Consolidation
Rear Abutment, Approach Embankment

PROJECT NO.: 0121-3070.03
SHEET NO. 1 OF 7
COMP. BY: SJK DATE 2-8-08
CHECKED BY: SWT DATE 2-8-08

Profile — Based on boring TR-35A

Assume groundwater table at D = 24.5'

$H = 46'$ Embankment, $\gamma = 120$ pcf — Existing Ground Surface

Depth	Layer	Properties	Source
0' – 5'	① Clay (A-7-6)	$\omega\% = 27\%$, $C_c = .27$, $C_r = .03$, $e_o = .729$	Estimated from $\omega\%$, *See Calculations*
5' – 32'	② Silt (A-4b)	$C_c = .27$, $C_r = .08$, $e_o = .928$, $P_c \approx 3400$ psf, $\gamma = 120$ pcf	Based on consolidation testing from TR-35A, P-3
32' – 62'	③ Clay (A-7-6)	$C_c = .19$, $C_r = .07$, $e_o = .706$, $P_c \approx 4428$ psf, $\gamma = 120$ pcf	Based on consolidation testing from TR-38A, P-4B
62' – 81'	④ Sand & Silt	$C' \approx 37$	Estimated from SPT (N-Values) *See Calculations*
81'	BEDROCK		

*Calculations: Ref: FHWA NHI-00-045 "Soils and Foundations Workshop"

• Soil layer ① Clay $\omega\% = 27\%$ *Assume $P_c = P_c$ from layer 2 = 3400 psf

Assume $G_s \approx 2.7$

$C_c = \omega/100 = .27$ $C_c = \omega/1000 = .03$ $e_o = G_s \cdot \omega(1/100) = .729$

• Soil layer ④ Sand and Silt $\bar{N} = 9$ bpf $\sigma' = 5647$ psf

$N'/N = .65 \rightarrow N' = 6$ bpf $\rightarrow C' \approx 37$

Estimate C_v — Based upon estimates from LL% ; Ref NAVFAC DM 7.01, figure 4

Soil layer ①, $\overline{LL}\% \approx 46\% \rightarrow 0.2$ ft²/day
Soil layer ②, $LL\% \approx 30\% \rightarrow 0.5$ ft²/day
Soil layer ③, $LL\% \approx 43\% \rightarrow 0.25$ ft²/day
Soil layer ④ Assume free draining

Use "weighted" average for time-rate of consolidation calculations.

$$\bar{C_v} = \frac{(5')(.2\ \text{ft}^2/\text{day}) + (27')(.5\ \text{ft}^2/\text{day}) + (30')(.25\ \text{ft}^2/\text{day})}{62'} = 0.35\ \text{ft}^2/\text{day}$$

Time-Rate of Consolidation (without wick drains)

$U = 90\%$

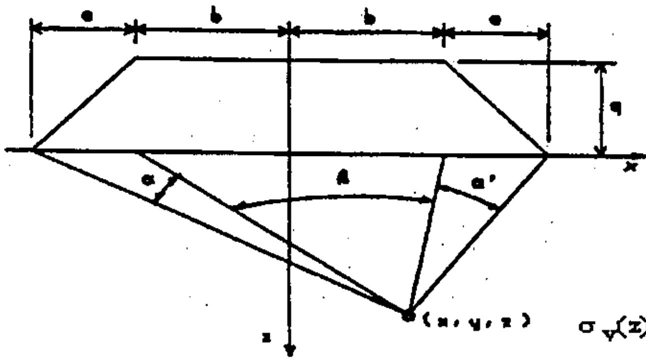
$$t_{90} = \frac{(0.848)(62/2)^2}{.35\ \text{ft}^2/\text{day}} = 2328\ \text{days} \simeq 6\ \text{years}$$

DLZ SUBJECT

Client Transystems Corp.	JOB NUMBER	0121-3070.03	
Project SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 7	
Item Embankment Settlement - Under CL	COMP. BY	SJK	DATE 2-8-08
At Sta. 130+75, Based on boring TR-35A	CHECKED BY	GWT	DATE 2-8-08

SETTLEMENT ANALYSIS - EMBANKMENT

Embankment Informaiton:



Groundwater Table:	D=	24.5	ft	
Embankment Height:	H =	46	ft	
Fill Unit Weight:	γ_{emb} =	120	pcf	q = 5,520 psf
Width of Slope:	a =	115		Assuming 2.5H:1V side slopes
Top half-width of Emb:	b =	51.5		
Distance from CL:	x =	0		
Output Range:	z =	0	to	81 ft

**See Data output Attached*

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

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

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

Soil Properites: *Settlement is calculated at mid-point of layer*

No.	Bot. of Laye		Soil Type	γ_{soil} (pcf)	σ'_c (psf)	σ'_o (psf)	$\Delta\sigma z$ (psf)	σ'_f (psf)	Cohesionless Soils C'	Cohesive Soils C_r	C_c	e_o
1	5.0	ft	Clay	120	3,400	300	5,520	5,820	0.0	0.03	0.27	0.729
2	15.0	ft	Silt	120	3,400	1,200	5,517	6,717	0.0	0.08	0.27	0.928
3	25.0	ft	Silt	120	3,400	2,400	5,497	7,897	0.0	0.08	0.27	0.928
4	32.0	ft	Silt	120	3,400	3,170	5,454	8,624	0.0	0.08	0.27	0.928
5	42.0	ft	Clay	120	4,428	3,660	5,393	9,053	0.0	0.07	0.19	0.706
6	52.0	ft	Clay	120	4,428	4,236	5,285	9,521	0.0	0.07	0.19	0.706
7	62.0	ft	Clay	120	4,428	4,812	5,162	9,974	0.0	0.07	0.19	0.706
8	72.0	ft	Sand & Silt	120	0	5,388	5,023	10,411	37.0	0.00	0.00	0.000
9	81.0	ft	Sand & Silt	120	0	5,935	4,874	10,809	37.0	0.00	0.00	0.000
10	0.0			0	0							

No.	Settlement:	
1	0.274	ft
2	0.602	ft
3	0.575	ft
4	0.405	ft
5	0.380	ft
6	0.378	ft
7	0.353	ft
8	0.077	ft
9	0.063	ft
10		

Total Settlement

$\boxed{3.107 \text{ ft}}$

$\boxed{37.3 \text{ in}}$

Reference: Geotechnical Engineering Principles and Practices; Coduto, 1999

Overconsolidated Soils - Case I ($\sigma'_0<\sigma'_c$) Eqn:11.24

$$(\delta_c)_{ult} = \Sigma \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Overconsolidated Soils - Case II ($\sigma'_0<\sigma'_c<\sigma_f$) Eqn:11.25

$$(\delta_c)_{ult} = \Sigma\left[\frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_c}{\sigma'_0}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_c}\right)\right]$$

Normally Consolidated Soils ($\sigma'_0=\sigma'_c$) Eqn: 11.23

$$(\delta_c)_{ult} = \Sigma \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Reference: FHWA NHI-00-045

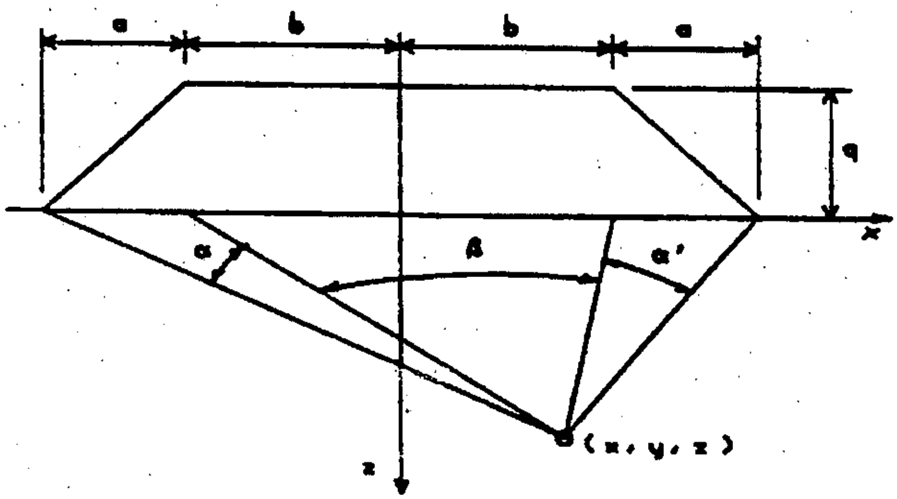
Cohesionless Soils ($\sigma'_0=\sigma'_c$)

$$(\delta_c)_{ult} = \Sigma \frac{1}{C'} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

DLZ

SUBJECT			
Client	Transystems Corp.	JOB NUMBER	
Project	SCI-823 Portsmouth Bypass	SHEET NO.	3 OF 7
Item	Embankment Settlement - Under CL	COMP. BY	SJK DATE 2-8-08
At Sta. 130+75, Based on boring TR-35A		CHECKED BY	SWT DATE 2-8-08

INCREASE IN VERTICAL STRESS DUE TO EMBANKMENT LOADING

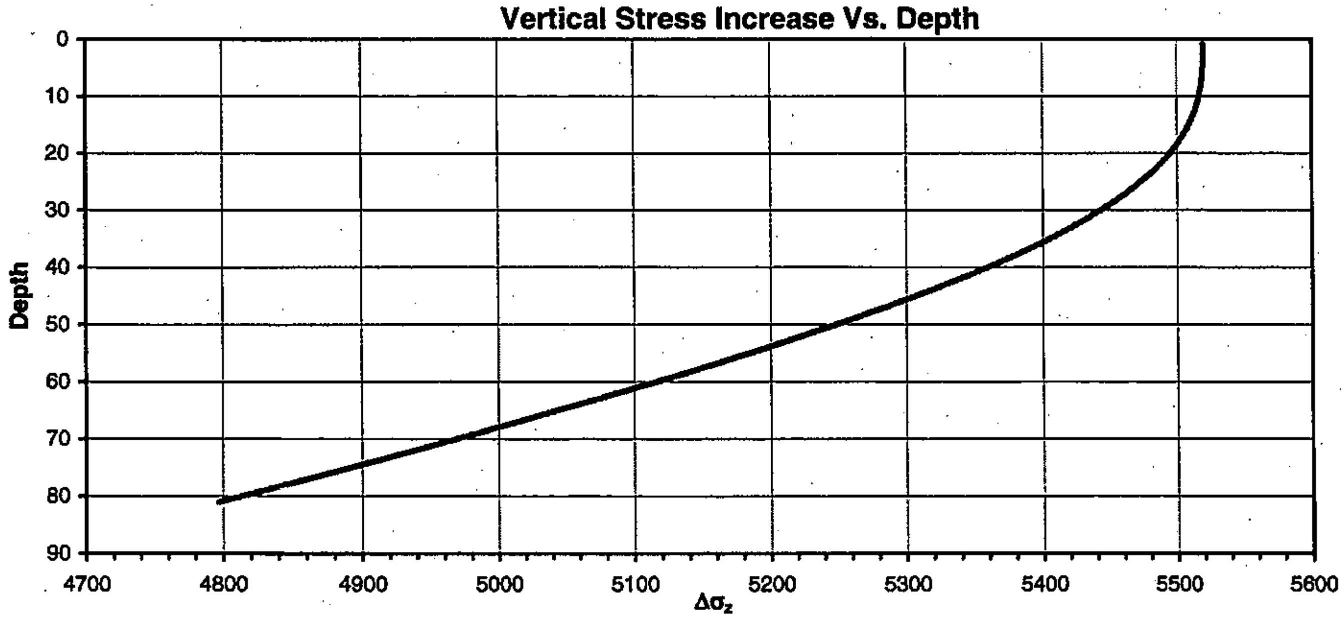


q =	5520	load
a =	115	width of slope
b =	51.5	top half-width of embankment
x =	0	distance from CL
z =	0 to 81	depth range

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

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

Vertical Stress Increase Vs. Depth



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

Sheet 4 of 7

DLZ

Time Rate of Consolidation of Foundation Soils with Wick Drians
Highland Bend Embankments **Based upon boring TR-35A**
Reference: FHWA-RD-86-168 Station 130+75

SJK 2-8-08
[illegible] 2-8-08

Wick Drain Spacing **5.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	*Remaining* δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	35.6	5.25	**0.35**	**31**	**35.6**
5	0.0635	0.0018	0.29	0.09	35.0	12.5	23.1				
10	0.1270	0.0036	0.49	0.09	53.4	19.0	16.6				
15	0.1905	0.0055	0.63	0.10	66.9	23.8	11.8				
20	0.2540	0.0073	0.74	0.11	76.4	27.2	8.4				
25	0.3175	0.0091	0.81	0.11	83.0	29.6	6.0				
30	0.3810	0.0109	0.86	0.12	87.5	31.1	4.5				
35	0.4444	0.0127	0.89	0.12	90.5	32.2	3.4				
40	0.5079	0.0146	0.91	0.13	92.5	32.9	2.7				
45	0.5714	0.0164	0.93	0.14	94.0	33.5	2.1				
50	0.6349	0.0182	0.94	0.14	95.2	33.9	1.7				
55	0.6984	0.0200	0.96	0.15	96.3	34.3	1.3				
60	0.7619	0.0219	0.97	0.15	97.4	34.7	0.9				
65	0.8254	0.0237	0.98	0.16	98.4	35.0	0.6				
70	0.8889	0.0255	0.99	0.16	99.1	35.3	0.3				

Assumes double drainage
Spacing = 5 ft (triangular)

SJK 2-8-08

DLZ

Time Rate of Consolidation of Foundation Soils with Wick Drians
Highland Bend Embankments **Based upon boring TR-35A**
Reference: FHWA-RD-86-168 Station 130+75

Wick Drain Spacing **7.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	Remaining δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	35.6	7.35	**0.35**	**31**	**35.6**
5	0.0324	0.0018	0.17	0.09	24.0	8.6	27.0				
10	0.0648	0.0036	0.29	0.09	35.9	12.8	22.8				
15	0.0972	0.0055	0.40	0.10	46.0	16.4	19.2	*Assumes double drainage*			
20	0.1296	0.0073	0.49	0.11	54.7	19.5	16.1	*Spacing = 7 ft (triangular)*			
25	0.1620	0.0091	0.57	0.11	62.0	22.1	13.5				
30	0.1944	0.0109	0.64	0.12	68.2	24.3	11.3				
35	0.2268	0.0127	0.70	0.12	73.4	26.1	9.5				
40	0.2592	0.0146	0.74	0.13	77.7	27.7	7.9				
45	0.2915	0.0164	0.78	0.14	81.2	28.9	6.7				
50	0.3239	0.0182	0.82	0.14	84.1	29.9	5.7				
55	0.3563	0.0200	0.84	0.15	86.5	30.8	4.8				
60	0.3887	0.0219	0.86	0.15	88.4	31.5	4.1				
65	0.4211	0.0237	0.88	0.16	90.0	32.0	3.6				
70	0.4535	0.0255	0.89	0.16	91.2	32.5	3.1				
75	0.4859	0.0273	0.91	0.17	92.3	32.8	2.8				
80	0.5183	0.0291	0.92	0.17	93.1	33.2	2.4				
85	0.5507	0.0310	0.93	0.18	93.9	33.4	2.2				
90	0.5831	0.0328	0.93	0.19	94.5	33.7	1.9				
95	0.6155	0.0346	0.94	0.19	95.1	33.9	1.7				
100	0.6479	0.0364	0.95	0.20	95.7	34.1	1.5				
105	0.6803	0.0382	0.95	0.20	96.3	34.3	1.3				
110	0.7127	0.0401	0.96	0.21	96.8	34.5	1.1				
115	0.7451	0.0419	0.97	0.21	97.3	34.6	1.0				
120	0.7775	0.0437	0.97	0.22	97.8	34.8	0.8				
125	0.8098	0.0455	0.98	0.22	98.3	35.0	0.6				

DLZ

Time Rate of Consolidation of Foundation Soils with Wick Drians
Highland Bend Embankments **Based upon boring TR-35A**
Reference: FHWA-RD-86-168 Station 130+75

Sheet 6 of 7
SJK 2-8-08
[illegible] 2-8-08

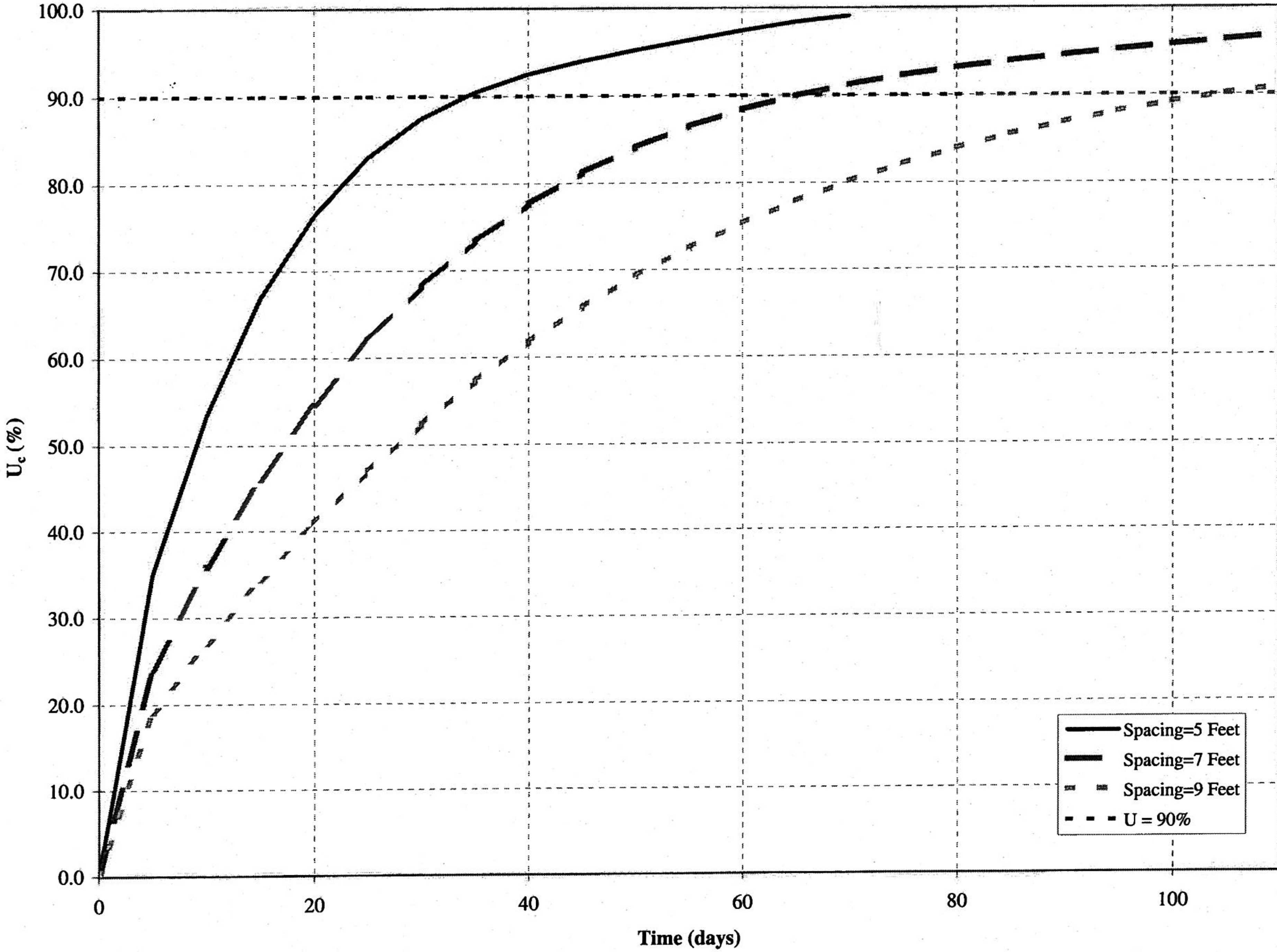
Wick Drain Spacing **9.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	*Remaining* δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	35.6	9.45	0.35	31	35.6
5	0.0196	0.0018	0.11	0.09	19.1	6.8	28.8				
10	0.0392	0.0036	0.20	0.09	27.0	9.6	26.0				
15	0.0588	0.0055	0.27	0.10	34.3	12.2	23.4				
20	0.0784	0.0073	0.34	0.11	40.9	14.6	21.0				
25	0.0980	0.0091	0.40	0.11	46.9	16.7	18.9				
30	0.1176	0.0109	0.46	0.12	52.4	18.7	16.9				
35	0.1372	0.0127	0.51	0.12	57.3	20.4	15.2				
40	0.1568	0.0146	0.56	0.13	61.7	22.0	13.6				
45	0.1764	0.0164	0.60	0.14	65.7	23.4	12.2				
50	0.1960	0.0182	0.64	0.14	69.3	24.7	10.9				
55	0.2156	0.0200	0.68	0.15	72.5	25.8	9.8				
60	0.2352	0.0219	0.71	0.15	75.4	26.8	8.8				
65	0.2548	0.0237	0.74	0.16	77.9	27.7	7.9				
70	0.2743	0.0255	0.76	0.16	80.2	28.5	7.1				
75	0.2939	0.0273	0.79	0.17	82.2	29.3	6.3				
80	0.3135	0.0291	0.81	0.17	83.9	29.9	5.7				
85	0.3331	0.0310	0.82	0.18	85.5	30.4	5.2				
90	0.3527	0.0328	0.84	0.19	86.9	30.9	4.7				
95	0.3723	0.0346	0.85	0.19	88.1	31.4	4.2				
100	0.3919	0.0364	0.86	0.20	89.1	31.7	3.9				
105	0.4115	0.0382	0.88	0.20	90.1	32.1	3.5				
110	0.4311	0.0401	0.89	0.21	90.9	32.4	3.2				
115	0.4507	0.0419	0.89	0.21	91.6	32.6	3.0				
120	0.4703	0.0437	0.90	0.22	92.3	32.9	2.7				
125	0.4899	0.0455	0.91	0.22	92.9	33.1	2.5				
130	0.5095	0.0473	0.91	0.23	93.4	33.2	2.4				
135	0.5291	0.0492	0.92	0.23	93.8	33.4	2.2				
140	0.5487	0.0510	0.92	0.24	94.3	33.6	2.0				
145	0.5683	0.0528	0.93	0.24	94.7	33.7	1.9				
150	0.5879	0.0546	0.93	0.25	95.0	33.8	1.8				
155	0.6075	0.0565	0.94	0.25	95.4	34.0	1.6				
160	0.6271	0.0583	0.94	0.26	95.7	34.1	1.5				
165	0.6467	0.0601	0.95	0.26	96.0	34.2	1.4				
170	0.6663	0.0619	0.95	0.27	96.4	34.3	1.3				
175	0.6859	0.0637	0.95	0.27	96.7	34.4	1.2				
180	0.7055	0.0656	0.96	0.28	97.0	34.5	1.1				
185	0.7251	0.0674	0.96	0.28	97.3	34.6	1.0				
190	0.7447	0.0692	0.97	0.28	97.6	34.7	0.9				
195	0.7643	0.0710	0.97	0.29	97.9	34.8	0.8				
200	0.7839	0.0728	0.97	0.29	98.1	34.9	0.7				
205	0.8034	0.0747	0.98	0.30	98.4	35.0	0.6				
210	0.8230	0.0765	0.98	0.30	98.6	35.1	0.5				

Assumes double drainage
Spacing = 9 ft (triangular)

DLZ

Percent Consolidation (combined radial and vertical) vs Time Using Wick Drains
SR 823 Bridge over Little Scioto River and SR 335, Rear Abutment (Station 130+75)



Sheet 7 of 7

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DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\LSR1.DVN
Project Name: SCI-823 Project Date: 11/26/2007
Project Client: Transystems
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 5.00 ft
Diameter of Pile: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	24.50 ft
	- Driving/Restrike	24.50 ft
	- Ultimate:	24.50 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	5.00 ft	0.00%	120.00 pcf	1700.00 psf	T-79 Steel
2	Cohesive	27.00 ft	0.00%	120.00 pcf	900.00 psf	T-79 Steel
3	Cohesive	30.00 ft	0.00%	120.00 pcf	2700.00 psf	T-79 Steel
4	Cohesionless	19.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund

14" CIP

Q_{ULT} = 140 tons = 280 kips

D = 66'

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1120.00 psf	0.00 Kips
5.01 ft	Cohesive	N/A	N/A	732.50 psf	0.03 Kips
14.01 ft	Cohesive	N/A	N/A	742.04 psf	24.50 Kips
23.01 ft	Cohesive	N/A	N/A	778.68 psf	51.40 Kips
31.99 ft	Cohesive	N/A	N/A	815.24 psf	80.65 Kips
32.01 ft	Cohesive	N/A	N/A	1225.49 psf	80.73 Kips
41.01 ft	Cohesive	N/A	N/A	1369.49 psf	125.91 Kips
50.01 ft	Cohesive	N/A	N/A	1460.00 psf	177.06 Kips
59.01 ft	Cohesive	N/A	N/A	1460.00 psf	225.22 Kips
61.99 ft	Cohesive	N/A	N/A	1460.00 psf	241.16 Kips
62.01 ft	Cohesionless	5100.29 psf	19.99	N/A	241.28 Kips
71.01 ft	Cohesionless	5359.49 psf	19.99	N/A	301.51 Kips
80.01 ft	Cohesionless	5618.69 psf	19.99	N/A	367.57 Kips
80.99 ft	Cohesionless	5646.91 psf	19.99	N/A	375.12 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	16.36 Kips
5.01 ft	Cohesive	N/A	N/A	N/A	8.66 Kips
14.01 ft	Cohesive	N/A	N/A	N/A	8.66 Kips
23.01 ft	Cohesive	N/A	N/A	N/A	8.66 Kips
31.99 ft	Cohesive	N/A	N/A	N/A	8.66 Kips
32.01 ft	Cohesive	N/A	N/A	N/A	25.98 Kips
41.01 ft	Cohesive	N/A	N/A	N/A	25.98 Kips
50.01 ft	Cohesive	N/A	N/A	N/A	25.98 Kips
59.01 ft	Cohesive	N/A	N/A	N/A	25.98 Kips
61.99 ft	Cohesive	N/A	N/A	N/A	25.98 Kips
62.01 ft	Cohesionless	5100.58 psf	30.00	14.24 Kips	14.24 Kips
71.01 ft	Cohesionless	5618.98 psf	30.00	14.24 Kips	14.24 Kips
80.01 ft	Cohesionless	6137.38 psf	30.00	14.24 Kips	14.24 Kips
80.99 ft	Cohesionless	6193.82 psf	30.00	14.24 Kips	14.24 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	16.36 Kips	16.36 Kips
5.01 ft	0.03 Kips	8.66 Kips	8.69 Kips
14.01 ft	24.50 Kips	8.66 Kips	33.16 Kips
23.01 ft	51.40 Kips	8.66 Kips	60.06 Kips
31.99 ft	80.65 Kips	8.66 Kips	89.31 Kips
32.01 ft	80.73 Kips	25.98 Kips	106.70 Kips
41.01 ft	125.91 Kips	25.98 Kips	151.88 Kips
50.01 ft	177.06 Kips	25.98 Kips	203.03 Kips
59.01 ft	225.22 Kips	25.98 Kips	251.19 Kips
61.99 ft	241.16 Kips	25.98 Kips	267.14 Kips
62.01 ft	241.28 Kips	14.24 Kips	255.52 Kips
71.01 ft	301.51 Kips	14.24 Kips	315.75 Kips
80.01 ft	367.57 Kips	14.24 Kips	381.81 Kips
80.99 ft	375.12 Kips	14.24 Kips	389.36 Kips

$\Delta = 60.23^{k}$

$\Delta/ft = 6.69^{k}/ft.$

D= 66'

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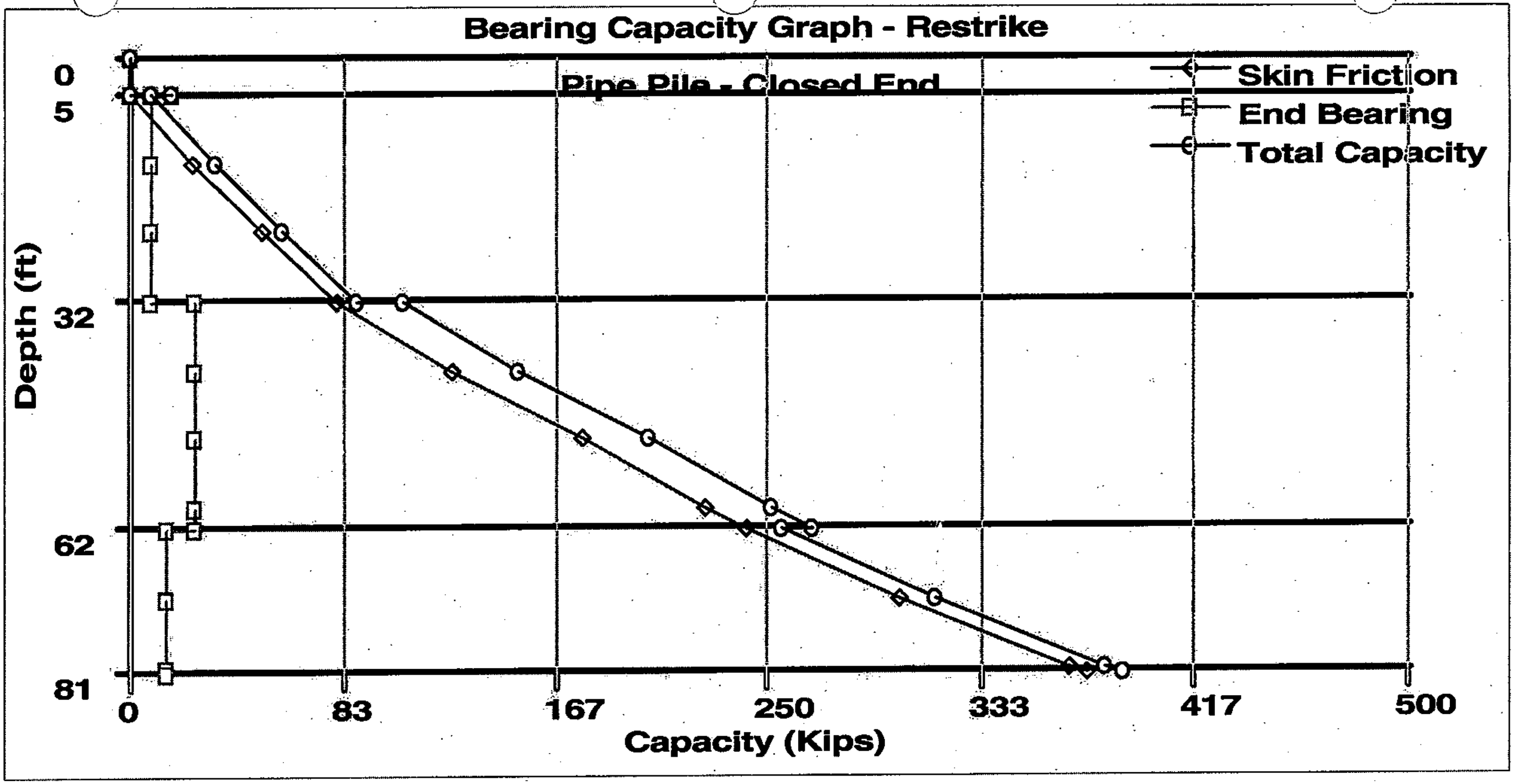

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\LSR1.DVN
Project Name: SCI-823 Project Date: 11/26/2007
Project Client: Transystems
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 5.00 ft
Diameter of Pile: 16.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	24.50 ft
	- Driving/Restrike	24.50 ft
	- Ultimate:	24.50 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	5.00 ft	0.00%	120.00 pcf	1700.00 psf	T-79 Steel
2	Cohesive	27.00 ft	0.00%	120.00 pcf	900.00 psf	T-79 Steel
3	Cohesive	30.00 ft	0.00%	120.00 pcf	2700.00 psf	T-79 Steel
4	Cohesionless	19.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund

16" CIP Q_{ult} = 180 tons = 360 Kips

D = 70'

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1120.00 psf	0.00 Kips
5.01 ft	Cohesive	N/A	N/A	732.50 psf	0.03 Kips
14.01 ft	Cohesive	N/A	N/A	734.91 psf	27.74 Kips
23.01 ft	Cohesive	N/A	N/A	766.97 psf	57.86 Kips
31.99 ft	Cohesive	N/A	N/A	798.96 psf	90.33 Kips
32.01 ft	Cohesive	N/A	N/A	1161.47 psf	90.41 Kips
41.01 ft	Cohesive	N/A	N/A	1287.47 psf	138.96 Kips
50.01 ft	Cohesive	N/A	N/A	1413.47 psf	197.00 Kips
59.01 ft	Cohesive	N/A	N/A	1460.00 psf	255.55 Kips
61.99 ft	Cohesive	N/A	N/A	1460.00 psf	273.77 Kips
62.01 ft	Cohesionless	5100.29 psf	21.97	N/A	273.92 Kips
71.01 ft	Cohesionless	5359.49 psf	21.97	N/A	355.08 Kips
80.01 ft	Cohesionless	5618.69 psf	21.97	N/A	444.08 Kips
80.99 ft	Cohesionless	5646.91 psf	21.97	N/A	454.25 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	21.36 Kips
5.01 ft	Cohesive	N/A	N/A	N/A	11.31 Kips
14.01 ft	Cohesive	N/A	N/A	N/A	11.31 Kips
23.01 ft	Cohesive	N/A	N/A	N/A	11.31 Kips
31.99 ft	Cohesive	N/A	N/A	N/A	11.31 Kips
32.01 ft	Cohesive	N/A	N/A	N/A	33.93 Kips
41.01 ft	Cohesive	N/A	N/A	N/A	33.93 Kips
50.01 ft	Cohesive	N/A	N/A	N/A	33.93 Kips
59.01 ft	Cohesive	N/A	N/A	N/A	33.93 Kips
61.99 ft	Cohesive	N/A	N/A	N/A	33.93 Kips
62.01 ft	Cohesionless	5100.58 psf	30.00	18.60 Kips	18.60 Kips
71.01 ft	Cohesionless	5618.98 psf	30.00	18.60 Kips	18.60 Kips
80.01 ft	Cohesionless	6137.38 psf	30.00	18.60 Kips	18.60 Kips
80.99 ft	Cohesionless	6193.82 psf	30.00	18.60 Kips	18.60 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	21.36 Kips	21.36 Kips
5.01 ft	0.03 Kips	11.31 Kips	11.34 Kips
14.01 ft	27.74 Kips	11.31 Kips	39.05 Kips
23.01 ft	57.86 Kips	11.31 Kips	69.17 Kips
31.99 ft	90.33 Kips	11.31 Kips	101.64 Kips
32.01 ft	90.41 Kips	33.93 Kips	124.34 Kips
41.01 ft	138.96 Kips	33.93 Kips	172.88 Kips
50.01 ft	197.00 Kips	33.93 Kips	230.93 Kips
59.01 ft	255.55 Kips	33.93 Kips	289.48 Kips
61.99 ft	273.77 Kips	33.93 Kips	307.70 Kips
62.01 ft	273.92 Kips	18.60 Kips	292.52 Kips
71.01 ft	355.08 Kips	18.60 Kips	373.67 Kips
80.01 ft	444.08 Kips	18.60 Kips	462.68 Kips
80.99 ft	454.25 Kips	18.60 Kips	472.84 Kips

$\Delta = 81.15^{K}$
$\Delta/ft = 9.02\ ^{K}/ft.$

$\boxed{D = 70'}$

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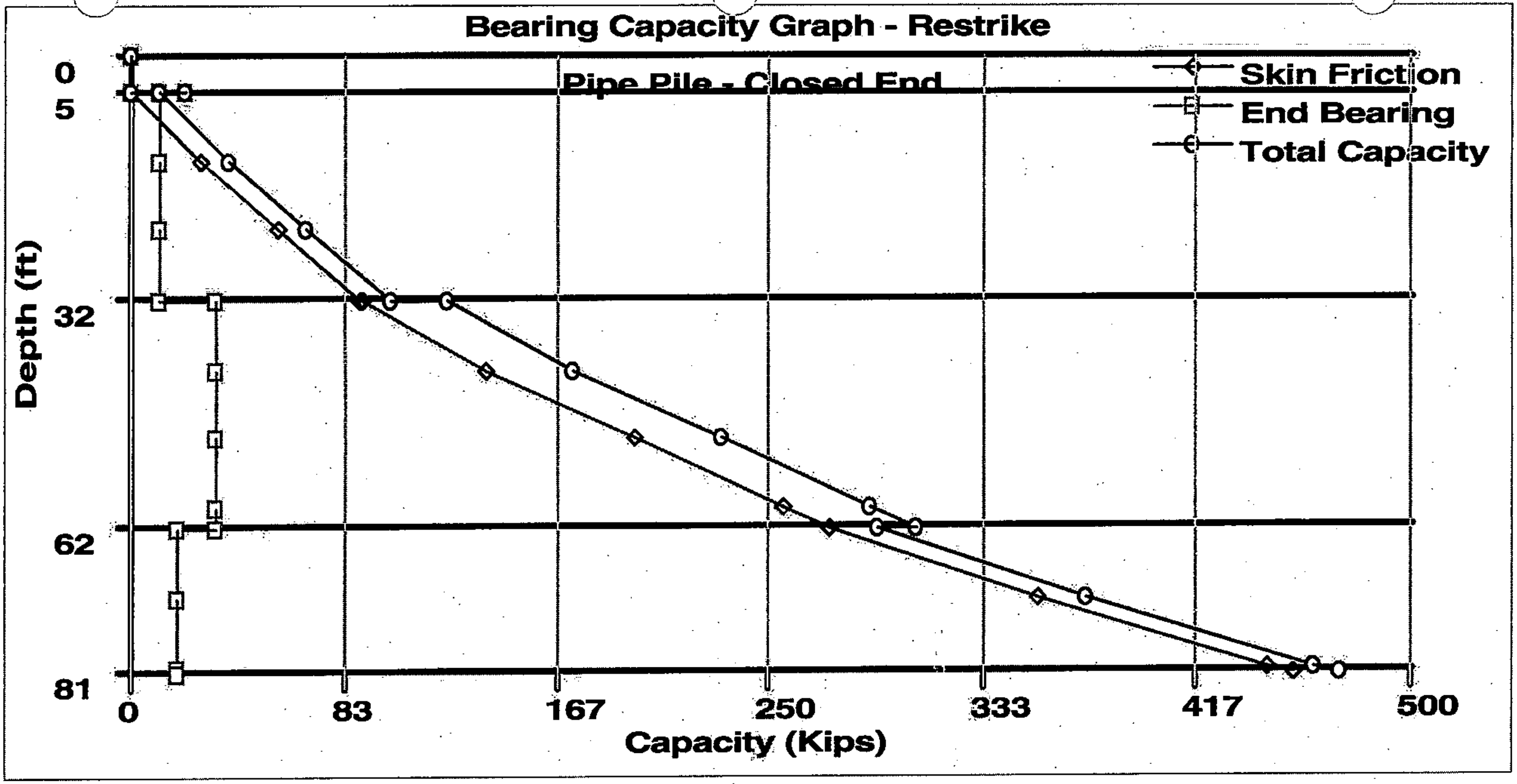

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\LSR1.DVN
Project Name: SCI-823 Project Date: 11/26/2007
Project Client: Transystems
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

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

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	24.50 ft
	- Driving/Restrike	24.50 ft
	- Ultimate:	24.50 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	5.00 ft	0.00%	120.00 pcf	1700.00 psf	T-79 Steel
2	Cohesive	27.00 ft	0.00%	120.00 pcf	900.00 psf	T-79 Steel
3	Cohesive	30.00 ft	0.00%	120.00 pcf	2700.00 psf	T-79 Steel
4	Cohesionless	19.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund

HP 12x53

Q_{ULT} = 140 tons = 280 kips

D = 65 ft.

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1120.00 psf	0.00 Kips
5.01 ft	Cohesive	N/A	N/A	732.50 psf	0.03 Kips
14.01 ft	Cohesive	N/A	N/A	751.30 psf	26.88 Kips
23.01 ft	Cohesive	N/A	N/A	793.89 psf	56.77 Kips
31.99 ft	Cohesive	N/A	N/A	836.38 psf	89.64 Kips
32.01 ft	Cohesive	N/A	N/A	1308.62 psf	89.73 Kips
41.01 ft	Cohesive	N/A	N/A	1460.00 psf	141.91 Kips
50.01 ft	Cohesive	N/A	N/A	1460.00 psf	194.09 Kips
59.01 ft	Cohesive	N/A	N/A	1460.00 psf	246.26 Kips
61.99 ft	Cohesive	N/A	N/A	1460.00 psf	263.54 Kips
62.01 ft	Cohesionless	5100.29 psf	22.59	N/A	263.66 Kips
71.01 ft	Cohesionless	5359.49 psf	22.59	N/A	321.34 Kips
80.01 ft	Cohesionless	5618.69 psf	22.59	N/A	384.60 Kips
80.99 ft	Cohesionless	5646.91 psf	22.59	N/A	391.83 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	1.65 Kips
5.01 ft	Cohesive	N/A	N/A	N/A	0.87 Kips
14.01 ft	Cohesive	N/A	N/A	N/A	0.87 Kips
23.01 ft	Cohesive	N/A	N/A	N/A	0.87 Kips
31.99 ft	Cohesive	N/A	N/A	N/A	0.87 Kips
32.01 ft	Cohesive	N/A	N/A	N/A	2.62 Kips
41.01 ft	Cohesive	N/A	N/A	N/A	2.62 Kips
50.01 ft	Cohesive	N/A	N/A	N/A	2.62 Kips
59.01 ft	Cohesive	N/A	N/A	N/A	2.62 Kips
61.99 ft	Cohesive	N/A	N/A	N/A	2.62 Kips
62.01 ft	Cohesionless	5100.58 psf	30.00	1.43 Kips	1.43 Kips
71.01 ft	Cohesionless	5618.98 psf	30.00	1.43 Kips	1.43 Kips
80.01 ft	Cohesionless	6137.38 psf	30.00	1.43 Kips	1.43 Kips
80.99 ft	Cohesionless	6193.82 psf	30.00	1.43 Kips	1.43 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	1.65 Kips	1.65 Kips
5.01 ft	0.03 Kips	0.87 Kips	0.90 Kips
14.01 ft	26.88 Kips	0.87 Kips	27.75 Kips
23.01 ft	56.77 Kips	0.87 Kips	57.65 Kips
31.99 ft	89.64 Kips	0.87 Kips	90.51 Kips
32.01 ft	89.73 Kips	2.62 Kips	92.34 Kips
41.01 ft	141.91 Kips	2.62 Kips	144.53 Kips
50.01 ft	194.09 Kips	2.62 Kips	196.70 Kips
59.01 ft	246.26 Kips	2.62 Kips	248.88 Kips
61.99 ft	263.54 Kips	2.62 Kips	266.16 Kips
62.01 ft	263.66 Kips	1.43 Kips	265.09 Kips
71.01 ft	321.34 Kips	1.43 Kips	322.77 Kips
80.01 ft	384.60 Kips	1.43 Kips	386.04 Kips
80.99 ft	391.83 Kips	1.43 Kips	393.26 Kips

$\Delta = 57.68^{k}$

$\Delta/ft = 6.41\ ^{k}/ft.$

Depth = 65 ft.

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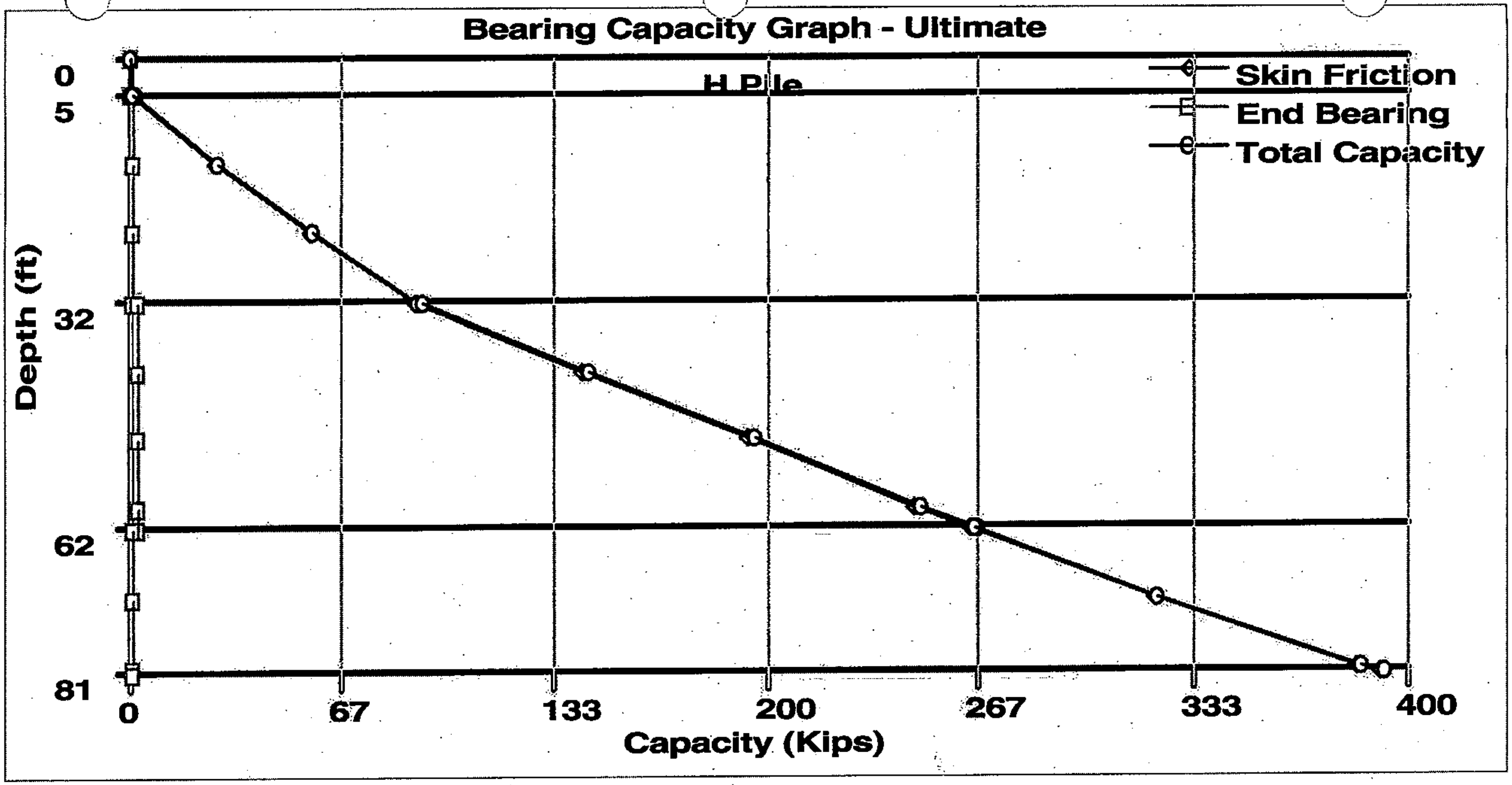

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\LSR1.DVN
Project Name: SCI-823 Project Date: 11/26/2007
Project Client: Transystems
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 5.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	24.50 ft
	- Driving/Restrike	24.50 ft
	- Ultimate:	24.50 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	5.00 ft	0.00%	120.00 pcf	1700.00 psf	T-79 Steel
2	Cohesive	27.00 ft	0.00%	120.00 pcf	900.00 psf	T-79 Steel
3	Cohesive	30.00 ft	0.00%	120.00 pcf	2700.00 psf	T-79 Steel
4	Cohesionless	19.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund

HP 14×73

Q_{ULT} = 190 tons = 380 Kips

D = 71 ft

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1120.00 psf	0.00 Kips
5.01 ft	Cohesive	N/A	N/A	732.50 psf	0.03 Kips
14.01 ft	Cohesive	N/A	N/A	739.75 psf	31.32 Kips
23.01 ft	Cohesive	N/A	N/A	774.93 psf	65.58 Kips
31.99 ft	Cohesive	N/A	N/A	810.02 psf	102.74 Kips
32.01 ft	Cohesive	N/A	N/A	1204.95 psf	102.83 Kips
41.01 ft	Cohesive	N/A	N/A	1343.17 psf	159.65 Kips
50.01 ft	Cohesive	N/A	N/A	1460.00 psf	226.34 Kips
59.01 ft	Cohesive	N/A	N/A	1460.00 psf	288.09 Kips
61.99 ft	Cohesive	N/A	N/A	1460.00 psf	308.53 Kips
62.01 ft	Cohesionless	5100.29 psf	23.58	N/A	308.68 Kips
71.01 ft	Cohesionless	5359.49 psf	23.58	N/A	384.83 Kips
80.01 ft	Cohesionless	5618.69 psf	23.58	N/A	468.35 Kips
80.99 ft	Cohesionless	5646.91 psf	23.58	N/A	477.88 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	2.27 Kips
5.01 ft	Cohesive	N/A	N/A	N/A	1.20 Kips
14.01 ft	Cohesive	N/A	N/A	N/A	1.20 Kips
23.01 ft	Cohesive	N/A	N/A	N/A	1.20 Kips
31.99 ft	Cohesive	N/A	N/A	N/A	1.20 Kips
32.01 ft	Cohesive	N/A	N/A	N/A	3.61 Kips
41.01 ft	Cohesive	N/A	N/A	N/A	3.61 Kips
50.01 ft	Cohesive	N/A	N/A	N/A	3.61 Kips
59.01 ft	Cohesive	N/A	N/A	N/A	3.61 Kips
61.99 ft	Cohesive	N/A	N/A	N/A	3.61 Kips
62.01 ft	Cohesionless	5100.58 psf	30.00	1.98 Kips	1.98 Kips
71.01 ft	Cohesionless	5618.98 psf	30.00	1.98 Kips	1.98 Kips
80.01 ft	Cohesionless	6137.38 psf	30.00	1.98 Kips	1.98 Kips
80.99 ft	Cohesionless	6193.82 psf	30.00	1.98 Kips	1.98 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	2.27 Kips	2.27 Kips
5.01 ft	0.03 Kips	1.20 Kips	1.24 Kips
14.01 ft	31.32 Kips	1.20 Kips	32.52 Kips
23.01 ft	65.58 Kips	1.20 Kips	66.79 Kips
31.99 ft	102.74 Kips	1.20 Kips	103.94 Kips
32.01 ft	102.83 Kips	3.61 Kips	106.45 Kips
41.01 ft	159.65 Kips	3.61 Kips	163.26 Kips
50.01 ft	226.34 Kips	3.61 Kips	229.95 Kips
59.01 ft	288.09 Kips	3.61 Kips	291.70 Kips
61.99 ft	308.53 Kips	3.61 Kips	312.14 Kips
62.01 ft	308.68 Kips	1.98 Kips	310.66 Kips
71.01 ft	384.83 Kips	1.98 Kips	386.81 Kips
80.01 ft	468.35 Kips	1.98 Kips	470.32 Kips
80.99 ft	477.88 Kips	1.98 Kips	479.86 Kips

$\Delta = 76.15^k$

$\Delta/ft = 8.46\ ^k/ft$

D = 71 ft.

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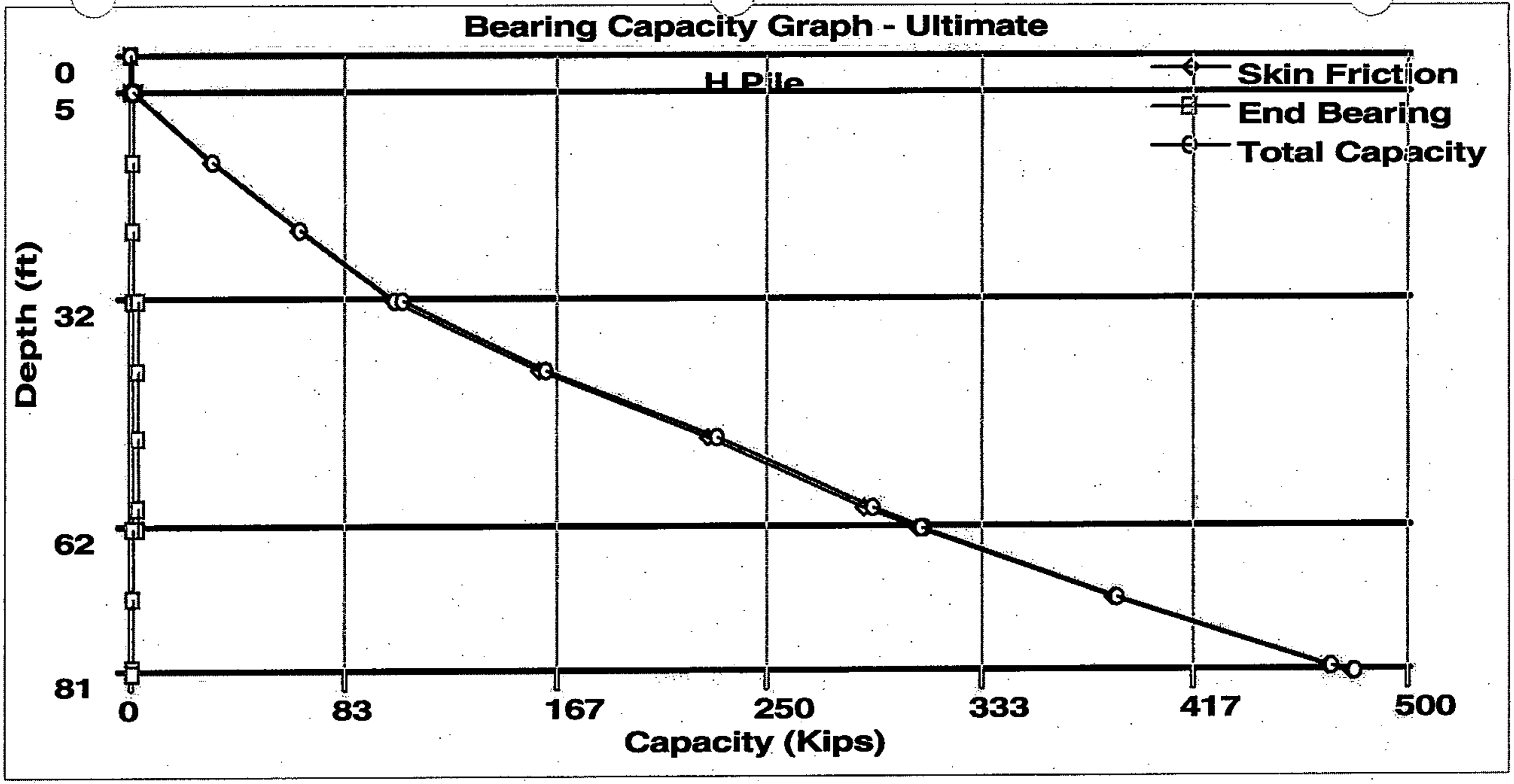

Reference: FHWA-IF-99-025, Drilled Shaft Construction Procedures and Design Methods

Assuming Smooth Rock Socket.

Based on boring B-42
Zone 1 from el 603.6 to 582.5,

$q_u = 1590$ psi (lower bound)
$E_r = 399$ ksi

Zone 1 - Side Resistance

$f_{max} = 0.65 \cdot p_a \left[q_u / p_a \right]^{0.5}$; where $q_u \leq f'_c$ {Eq. 11.24}

$f_{max} = 0.65 (14.7 \text{ psi}) \left[1590 \text{ psi} / 14.7 \text{ psi} \right]^{0.5} = 99.4$ psi

$f_{max} = 99.4$ psi $= 14{,}300$ psf

$f_{all} = \dfrac{f_{max}}{F.S.} = \dfrac{14{,}300 \text{ psf}}{3.0} = 4{,}770$ psf Use $f_{all} = 4500$ psf

Zone 1 - Uplift Side Resistance

$f_{max_{Uplift}} = \Psi \cdot f_{max_{compression}}$ {Eq. 11.30}

Use $\Psi = 0.7$

$f_{all_{uplift}} = 0.7 (4500 \text{ psf}) = 3150$ psf Use $f_{all_{uplift}} = 3000$ psf

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CLIENT	TranSystems Corp.	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 2
SUBJECT	Little Scioto River - Ret. Wall	COMP. BY	SJK DATE 11-19-07
	Drilled Shaft - Side Resistance	CHECKED BY	DWT DATE 11-20-07

Based on boring B-43

Zone 2 from el. 582.5 to 532.9;

$q_u = 7886$ psi (lower bound)

$E_r = 2.2 \times 10^6$ psi

Zone 2 - Side Resistance

$f_{max} = 0.65\ p_a \left[q_u / p_a \right]^{0.5}$ $\quad q_u \leq f_c' \quad$ Use $q_u = f_c' = 4500$ psi

$f_{max} = 0.65 (14.7\ psi) \left[(4500\ psi) / (14.7\ psi) \right]^{0.5} = 167$ psi

$f_{max} = 167$ psi $= 24{,}000$ psf

$f_{all} = \dfrac{f_{max}}{F.S.} = \dfrac{24{,}000\ psf}{3.0} = 8{,}000$ psf $\quad$ Use $f_{all} = 7500$ psf

Zone 2 - Uplift Side Resistance

$f_{max_{uplift}} = \psi \cdot f_{max_{compression}}$ $\quad$ Use $\psi = 0.7$

$f_{all_{uplift}} = 0.7 (7500\ psf) = 5250$ psf $\quad$ Use $f_{all_{uplift}} = 5{,}000$ psf

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CLIENT: Transystems Corp. — PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass — SHEET NO. 1 OF 14
SUBJECT: Forward Abutment Ret. Wall (Lt.) SR 823 over SR 335 and Little Scioto Rv — COMP. BY SJK DATE 2-8-08
CHECKED BY [illegible] DATE 11-Feb-08

*Abutment Wall – perpendicular to alignment

From Provided plans: At highest point; top of wall, el = 618
bot of Cap, el = 588

Assume H = 30.0' Length of wall = 45.1'

Traffic Loading: $q_{TR} = (2')(125\ pcf) = 250\ psf$ (verticle load)

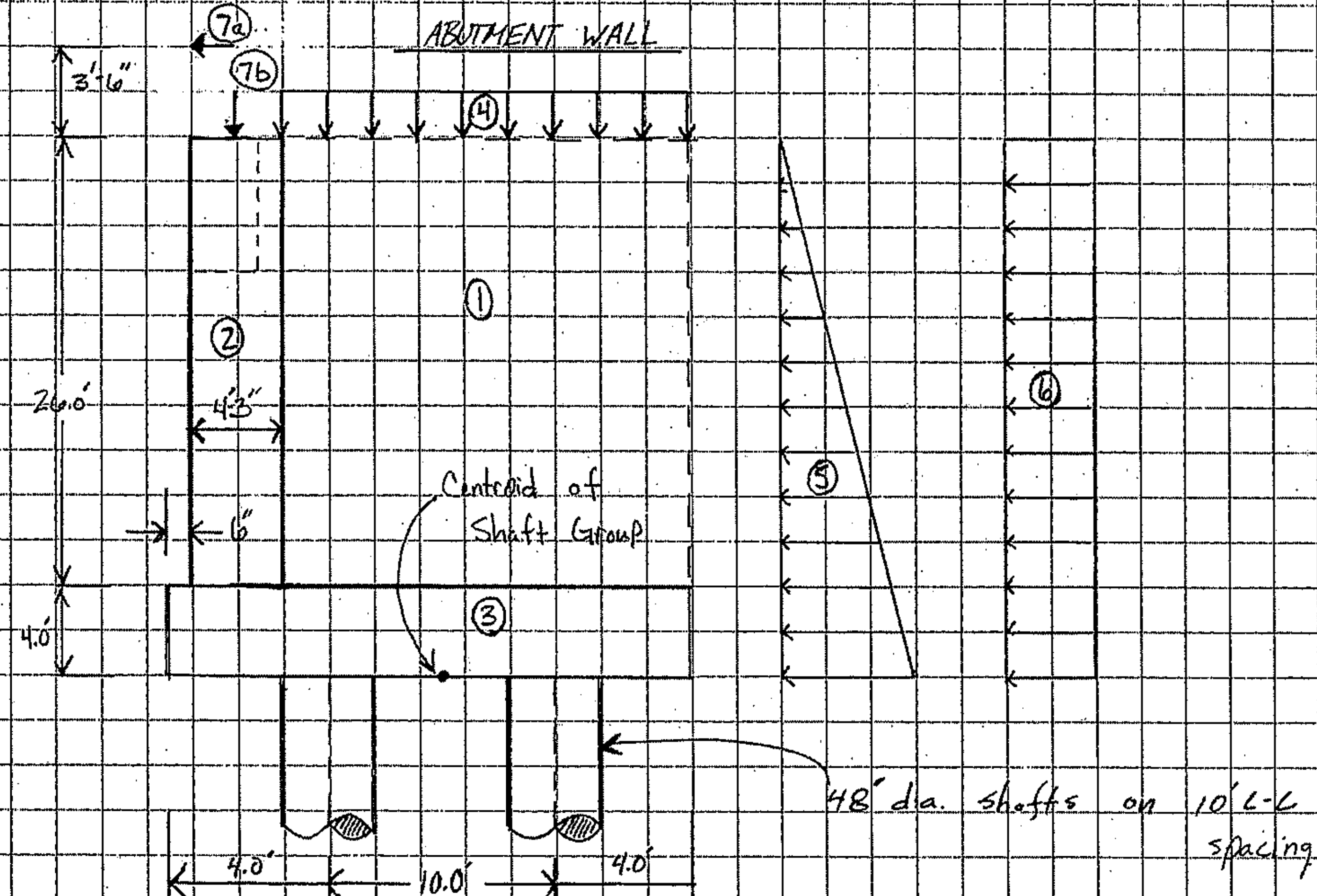
Assumed Fill Parameters: $\gamma = 125\ pcf$

$C = 0 \qquad \phi = \phi' = 30°$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.33$$

Assume: 48" dia. drilled shafts at 2.5 D spacing (c-c) min.
*Conservatively assume maximum wall height along length of wall.

ABUTMENT WALL



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CLIENT Transystems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Forward Abutment Ret. Wall (Lt.)

PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 14
COMP. BY SAK DATE 2-8-08
CHECKED BY TAH DATE 11-Feb-08

* Loads

① Vertical Soil dead weight
② Retaining Wall dead weight
③ Footing dead weight
④ Vehicle surcharge - vertical component (equal to 2' of fill)
⑤ Soil lateral load
⑥ Vehicle surcharge - horizontal component
(7a) Horizontal Impact Loading (Retaining Wall only)
(7b) Vertical Superstructure dead load and live load

*Example Calculations

① F_1 = (13.25')(26.0')(125 pcf) = 43,063 lb/ft of width
② F_2 = (4.25')(26.0')(150 pcf) = 16,575 lb/ft width
③ F_3 = (18')(4.0')(150 pcf) = 10,800 lb/ft width
④ F_4 = (18' - 0.5 - 4.25')(250 psf) = 3,313 lb/ft width
⑤ $F_5 = \frac{1}{2}(0.33)(125\ pcf)(30.0')^2$ = 18,563 lb/ft width
⑥ F_6 = (0.33)(250 psf)(30.0') = 2,475 lb/ft width
(7a) Parapet horizontal load (equal to 10 kips per AASHTO Fig. 2.7.4B
Spread load over length of 5-feet per AASHTO 2.7.1.3.6
F_{7a} = 2 kips per linear foot. Assume acting at 3.5' above wall.
F_{7a} only for Retaining Wall (parallel to alignment)
(7b) Superstructure load (Abutment wall only)
As per TranSystems, DL per beam is 189.84 kips
LL per lane is 106.36 kips

5-beam Arrangement
for entire wall: DL = (5)(189.84 k) = 949.2 k
LL = (3 lanes)(106.36 k/lane)(0.9) = 287.17 k

F_{7b} = (949.2 k) + (287.2 k) = 1236.4 k * For entire length of wall.
= 27.4 k/ft width

Force and Moment Summary Sheet 3 of 14 S/K 2-8-08

SR 823 over Little Scioto River, Abutment Retaining Wall (AW) TAA 11-Feb-08

Spacing, Shaft to Shaft in same row 10 feet
Spacing, Between Rows 10 feet
No of Shafts per Row 5 Shaft Dia. (in) 48 Shaft Area (in^2) 1809

Load ID	Vert. Or Hor.?	Unfactored load (kips)	Arm (ft)	Unfactored Moment (kip-ft)	
1	V	1942.1	-2.375	-4613	Vertical Soil Dead Weight
2	V	748	6.375	4766	Retaining Wall Dead Weight
3	V	487.1	0	0	Footing Dead Weight
4	V	149.4	-2.375	-355	Vehicle Surcharge - Vertical
5	H	837	10	8372	Soil Lateral Load
6	H	112	15	1674	Vehicle Surcharge - Horizontal
7a	H	0	33.5	0	Impact Loading
7b	V	1236	6.375	7880	Axial load from structure
		5510.9		17724 kip-feet	212684430 lb-in Mz 425368860

17,724 kip-ft

c/c spacing between shaft rows = 10 feet

Footing over hang = 4 feet In Y-direction
Footing thickness = 4 feet

Wall Height = 30 feet

Wall Length = 45.1 feet

Summation Vertical Loads = 4562.1 kips 4562125 lbs X-Direction 9124250
Summation Horizontal Loads = 948.8 kips 948791 lbs Y-Direction 1897582.5

Check: V= 4562.1 kips
M= 17724 k-ft N= 10 x_bar = 0

e = $\Sigma M/\Sigma V$ = 3.88 feet $\Sigma Nd^2 = 250$

$$P = \frac{V}{N} \pm \frac{Vec}{\Sigma Nd^2}$$

P= 456 +/- 354

P_{max} = 811 kips
P_{min} = 102 kips

σ_{max} =	64.5	ksf
σ_{min} =	8.1	ksf

Does not Include self-weight of drilled shafts.

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CLIENT Transystems Corp
PROJECT SCI-823 Portsmouth Bypass
SUBJECT SR823 over Little Scioto River.
Forward Abut Retaining Wall

PROJECT NO. 0121-3070.03
SHEET NO. 4 OF 14
COMP. BY SJK DATE 2-8-08
CHECKED BY TRW DATE 11-Feb-08

* Abutment Wall

From GROUP Analyses:

per shaft (2 Rows of 5 shafts each)

$V_{max} = 131$ kips $\quad M_{max} = 204$ k-ft (UNFACTORED)

Check Shear

Assume a "generic" load factor of 2.0

$V_u = (2.0)(131 \text{ kips}) = 262 \text{ kips}$

$\phi V_c = (0.85)(2)\sqrt{4500 \text{ psi}}\,(48'')\left(\frac{48''}{2}\right)\left(\frac{1}{1000}\right) = 131^k$

Deficiency, $\phi V_s = 262^k - 131^k = 131^k$

$\phi V_s = 131^k = (0.85)\dfrac{A_v \cdot (60 \text{ ksi})(48''/2)}{4''}$ Solve for A_v

$A_v = 0.43 \text{ in}^2 \qquad \dfrac{.43 \text{ in}^2}{2} = 0.215 \text{ in}^2$

Use #5 bar @ 4" pitch

OR Use #4 bar @ 3" pitch

* Shear and Moment Capacity are considered adequate with a reasonable amount of reinforcing.

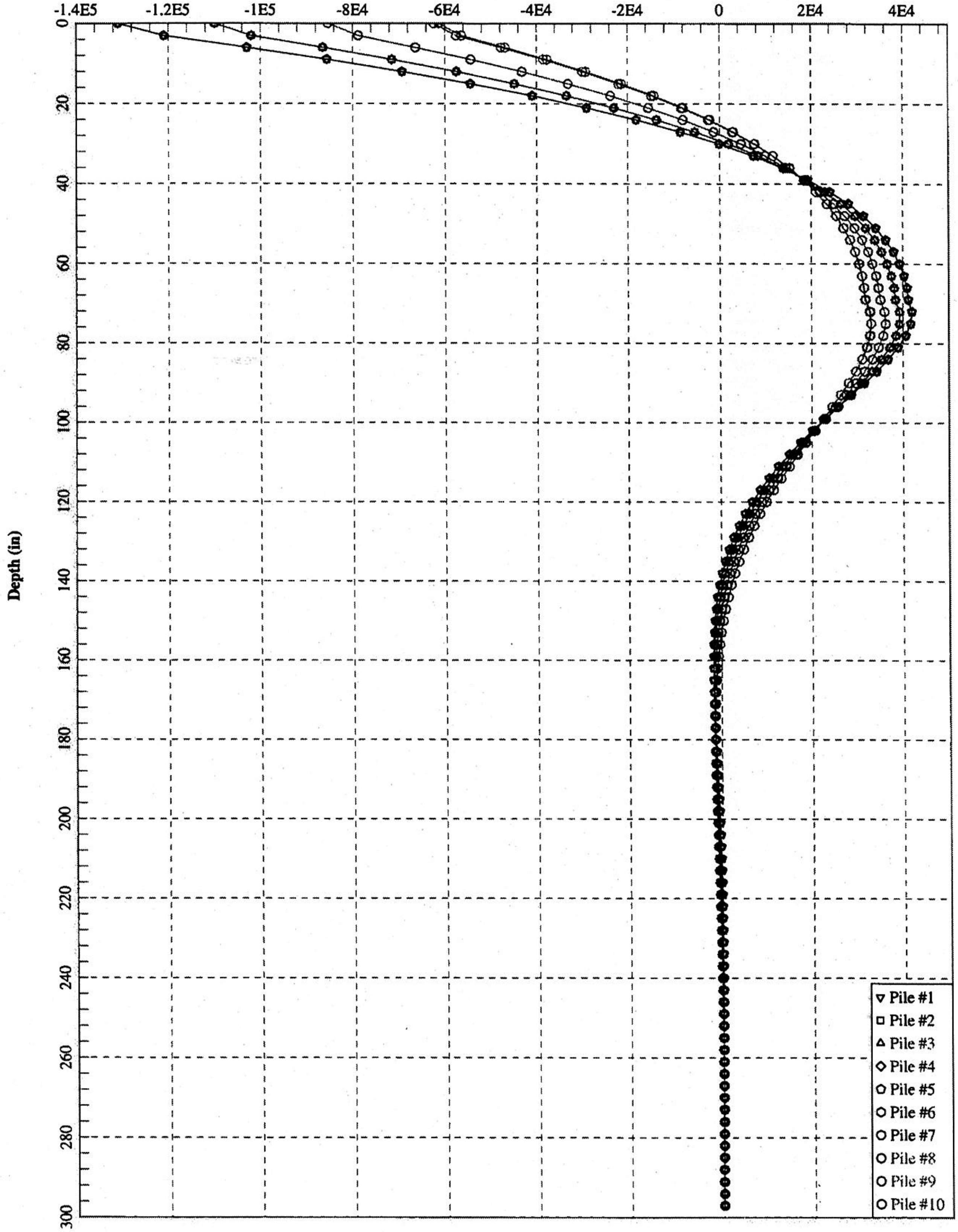
Using a load factor of 2.0, the minimum required rock socket length, $L_{min} = 216'' = 18'$.

Although shaft tip pressures exceed 80 ksf, axial capacities may be achieved through side friction along the length of the rock socket.

Sheet 5 of 14
S7K 2-8-08
TRW 11-Feb-08

Abutment Wall (AW)

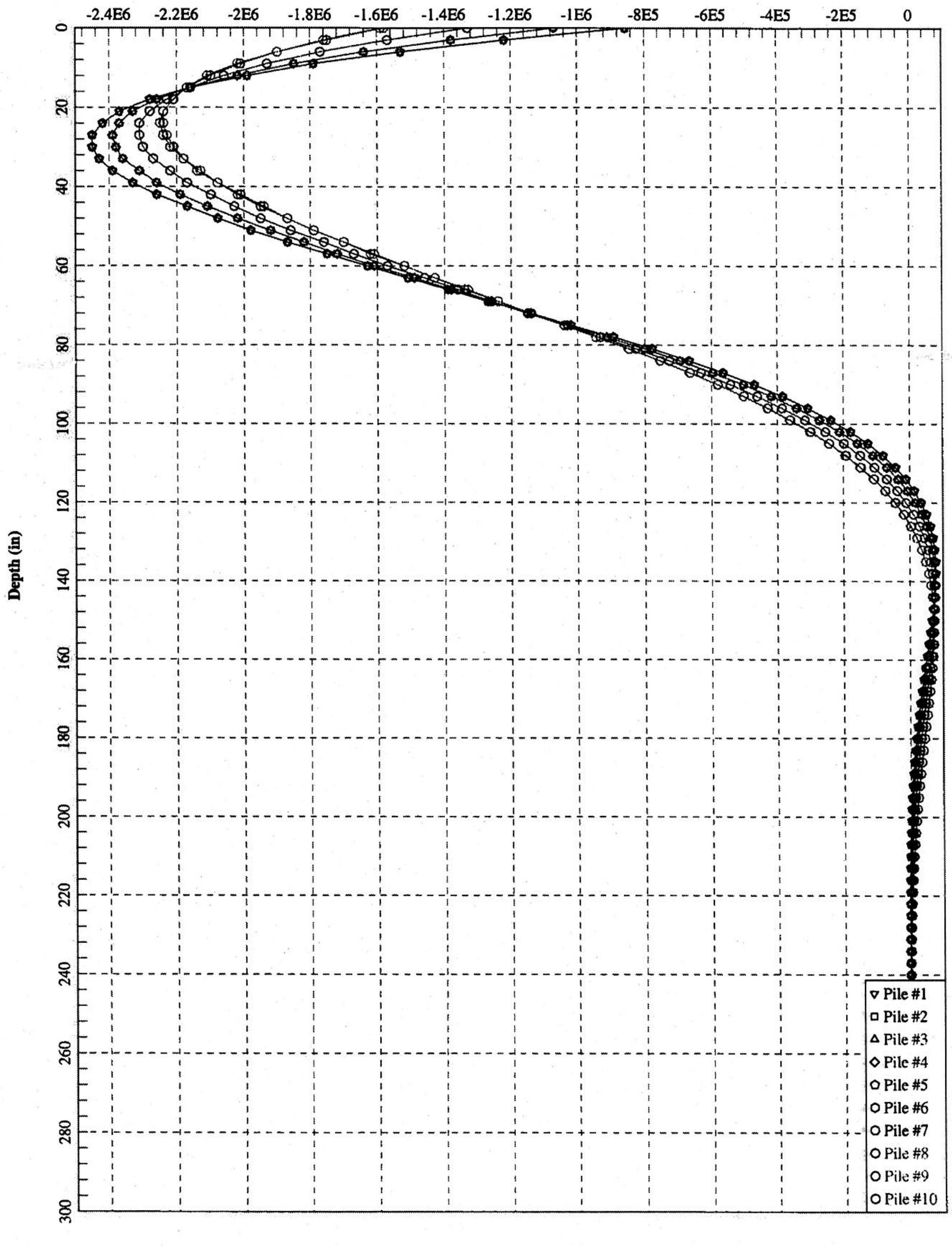
Shear Y dir (lbs)



Sheet 6 of 14
SJK 2-8-08
THA 11-Feb-08

Abutment Wall (AW)

Moment Z dir (lbs-in)



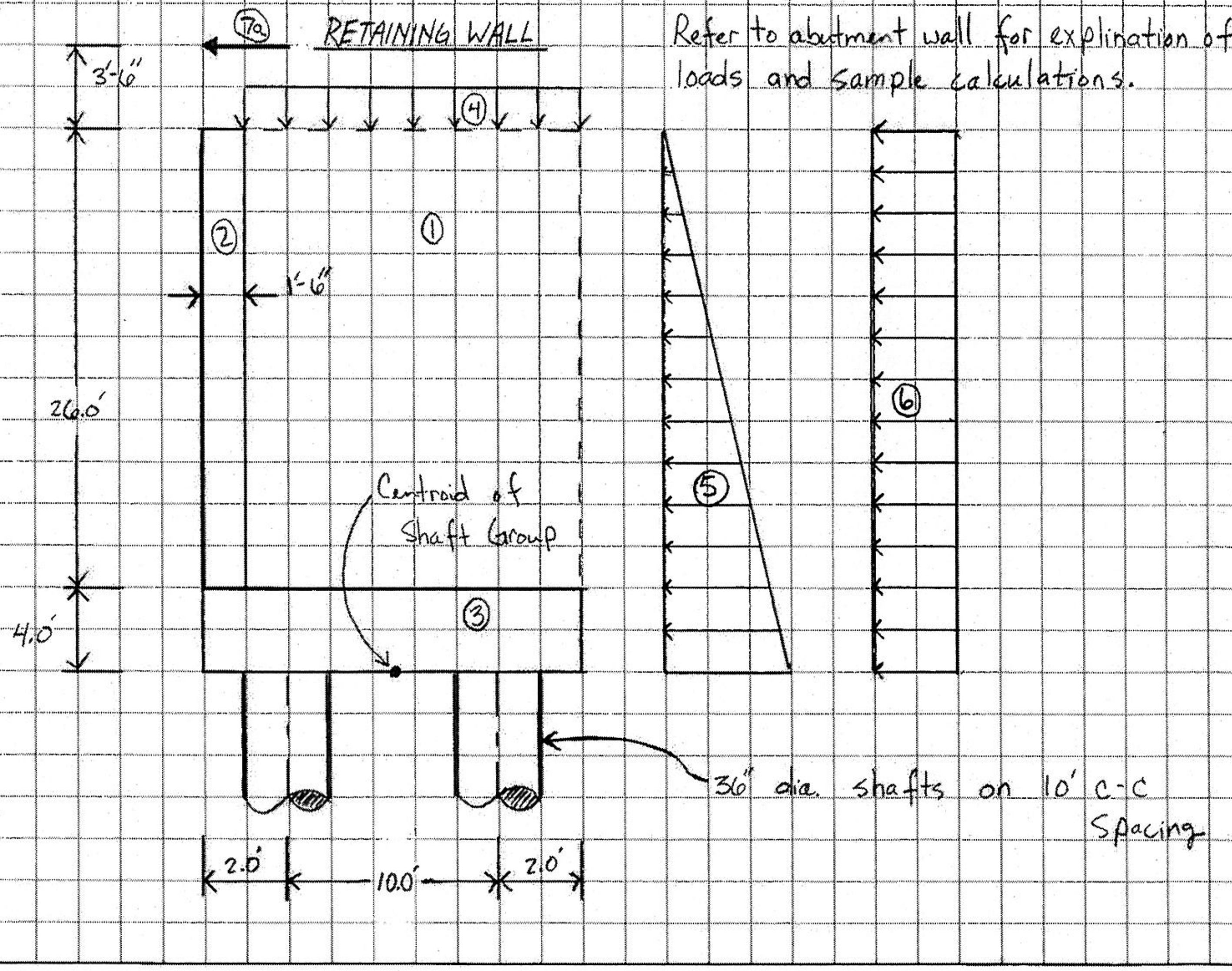
* Retaining Wall - parallel to alignment
From provided plans: At highest point; Top of wall, el = 618
bot of wall, el = 588

Assume H = 30.0'

* Because retaining wall (parallel to alignment) is supported by 48" diameter shafts at the corner of wall, assume wall section supported by 36" diameter shafts begins at station 139+15. Assume wall length supported by drilled shafts is 50 feet in length.

Assume: 36" diameter drilled shafts at 3.33 D spacing (c-c) min.
Conservatively assume maximum wall height along length of wall.

Refer to abutment wall for explination of loads and sample calculations.



Force and Moment Summary
SR 823 over Little Scioto River, Retaining Wall (RW)

Sheet 8 of 14
SJK 2-8-08
TAW 11-Feb-08

Spacing, Shaft to Shaft in same row 10 feet
Spacing, Between Rows 10 feet
No of Shafts per Row 5

Shaft Dia. (in) 36   Shaft Area (in^2) 1017

Load ID	Vert. Or Hor.?	Unfactored load (kips)	Arm (ft)	Unfactored Moment (kip-ft)	
1	V	2031.3	-0.75	-1523	Vertical Soil Dead Weight
2	V	293	6.25	1828	Retaining Wall Dead Weight
3	V	420.0	0	0	Footing Dead Weight
4	V	156.3	-0.75	-117	Vehicle Surcharge - Vertical
5	H	928	10	9281	Soil Lateral Load
6	H	124	15	1856	Vehicle Surcharge - Horizontal
7a	H	100	33.5	3350	Impact Loading
7b	V	0	0	0	Axial load from structure
		4051.9		14675 kip-feet	

176100000 lb-in   Mz   352200000
14,675 kip-ft

c/c spacing between shaft rows = 10 feet

Footing over hang = 2 feet   In Y-direction
Footing thickness = 4 feet

Wall Height = 30 feet

Wall Length = 50 feet

Summation Vertical Loads = 2900.0 kips   2900000 lbs   X-Direction   5800000
Summation Horizontal Loads = 1151.9 kips   1151875 lbs   Y-Direction   2303750

Check: V= 2900.0 kips
M= 14675 k-ft   N= 10   x_bar = 0

$e = \Sigma M / \Sigma V =$ 5.06 feet $\Sigma Nd^2 = 250$

$$P = \frac{V}{N} \pm \frac{Vec}{\Sigma Nd^2}$$

P= 290 +/- 294

$P_{max} =$ 584 kips
$P_{min} =$ -4 kips

$\sigma_{max} =$	82.6 *	ksf
$\sigma_{min} =$	-0.5	ksf

- Does not Include shaft self weight.
- Develop axial resistance via side-friction,

Retaining Wall

From GROUP Analyses:

per shaft (2 Rows of 5 shafts each)

$V_{max} = 137^k$ $M_{max} = 111$ k-ft (UNFACTORED)

Check Shear

Assume a "generic" load factor of 2.0

$V_u = 2.0(137^k) = 270^k$ Say $V_u = 270^k$

$\phi V_c = (0.85)(2)(\sqrt{4500})(36")(\frac{36"}{2})(1/1000) = 74^k$

Deficiency: $\phi V_s = 270 - 74 = 196^k$

$\phi V_s = 196^k = (0.85)\dfrac{A_v (60ksi)(36/2)}{3"}$ Solve for A_v

$A_v = 0.64 \text{ in}^2$ $0.64/2 = 0.32 \text{ in}^2$

Use #5 bar @ 3" pitch

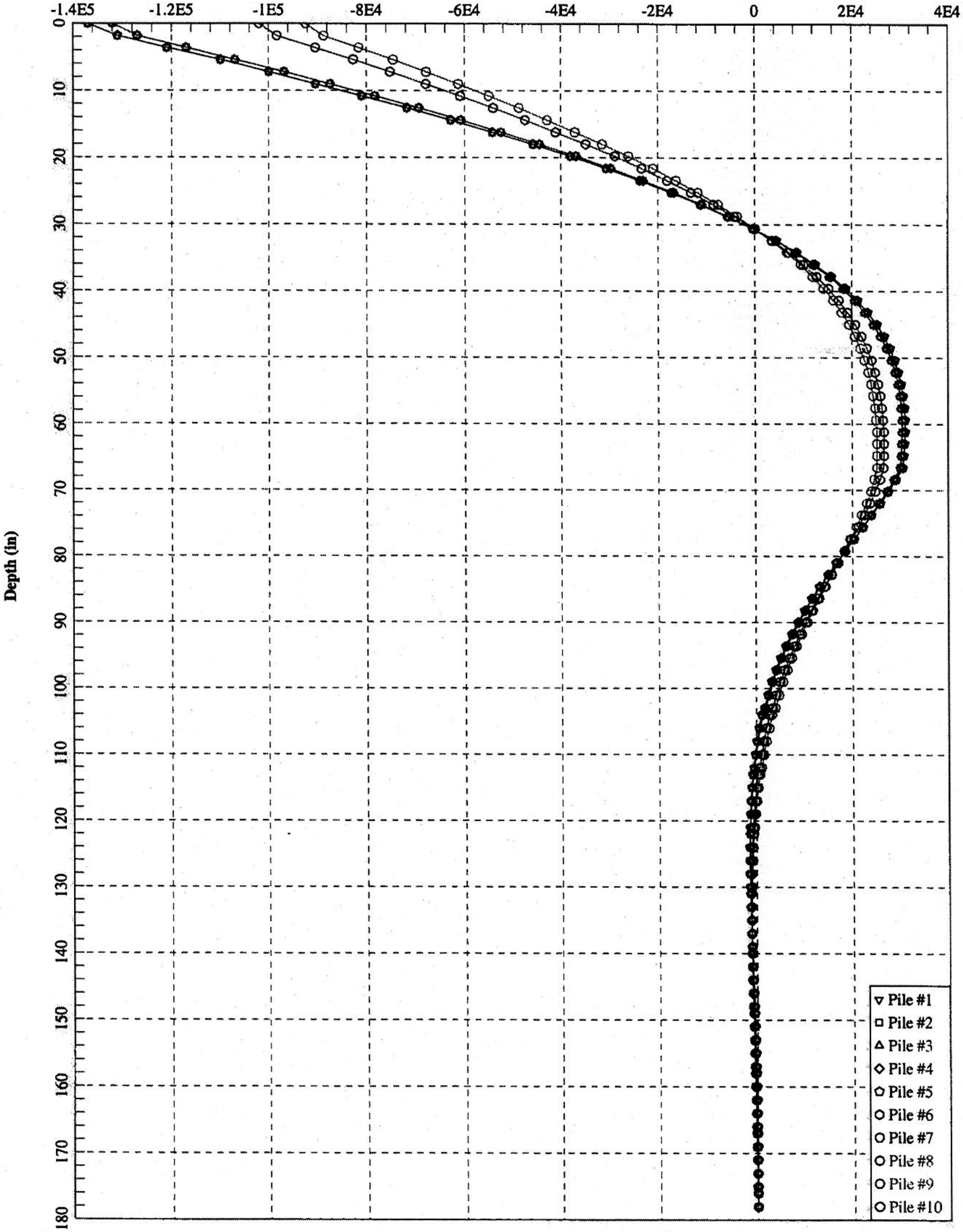
* Shear and Moment Capacity are considered adequate with a reasonable amount of reinforcement.

* Using a load factor of 2.0, the minimum required rock socket length, $L_{min} = 180" = 15'$

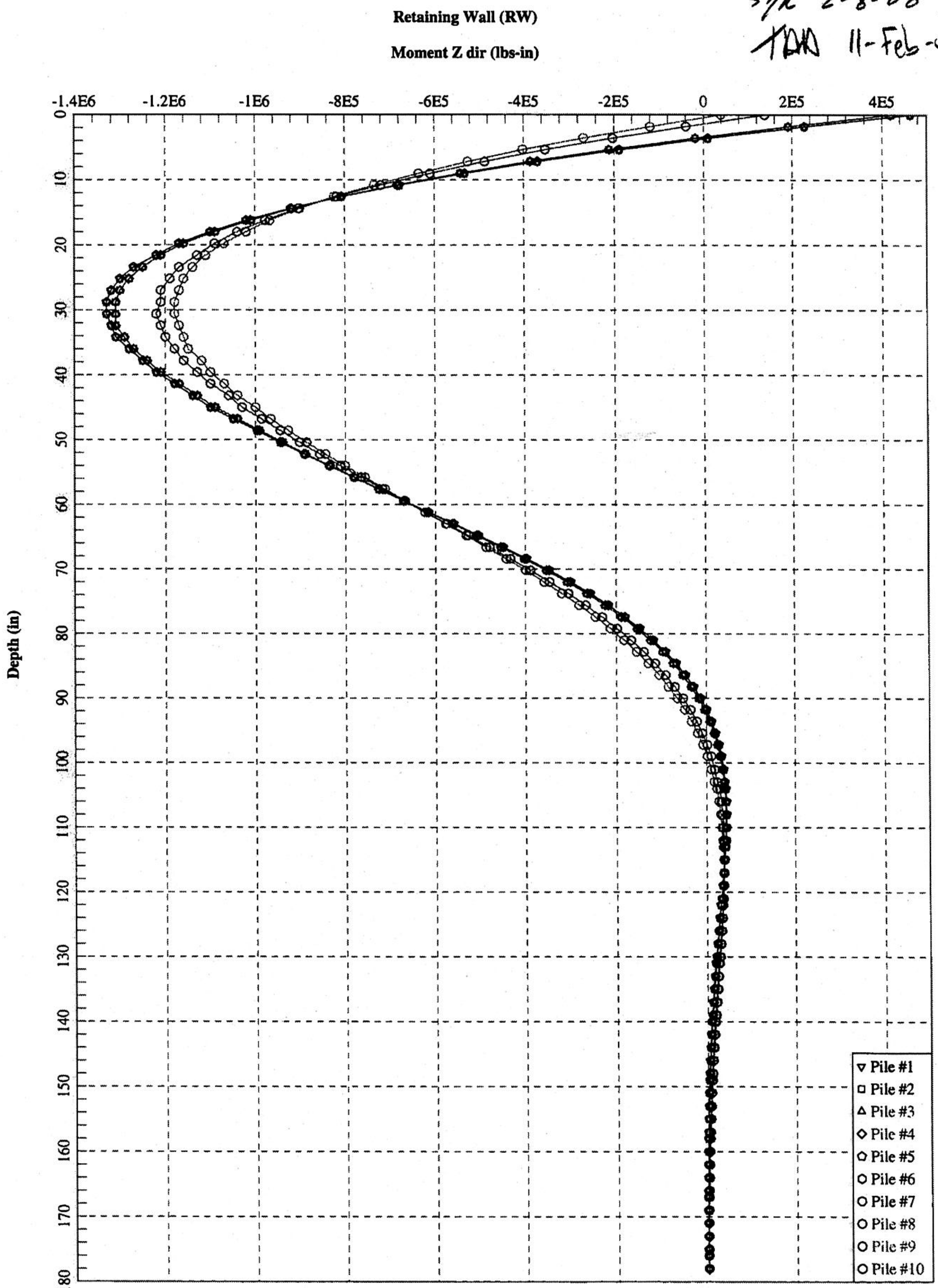
Sheet 10 of 14
S/K 2-8-08
TAD 11-Feb-08

Retaining Wall (RW)

Shear Y dir (lbs)



Sheet 11 of 14
S/K 2-8-08
TDM 11-Feb-08



DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT	Transystems Corp.	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	12 OF 14
SUBJECT	Forward Abutment Ret. Wall (Lt.)	COMP. BY	SJK DATE 2-8-08
	S.R. 823 over SR 335 and Little Scioto Rv.	CHECKED BY	[illegible] DATE 11-Feb-08

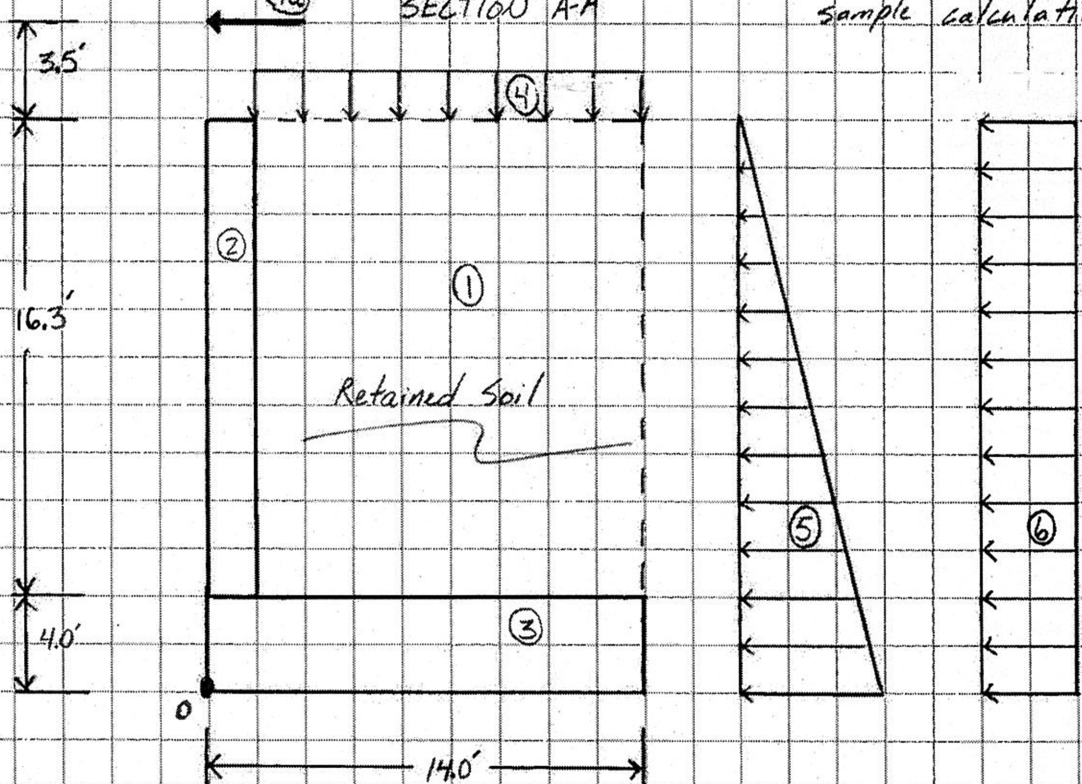
* Retaining Wall - parallel to alignment * SPREAD FOOTINGS *

* Section A-A ; Top of wall, el. 624.3
Bot. of cap, el. 604.0

Assume H = 20.3 ft. Footing Thickness = 4.0 ft.

Refer to abutment wall for explination of loads and sample calculations

RETAINING WALL
SECTION A-A



• Retained Soil: γ = 125 pcf ϕ = 30° K_a = 0.33
• Traffic loading = 250 psf
• Impact Loading = 2 kips per linear foot Acting 3.5' above wall
⟶ See page 2 for additional details

Sliding and Overturning are adequate OK
Refer to Force and Moment Summary on the following page.

Sheet 13 of 14
SJK 2-8-08
TAH 11-Feb-08

Force and Moment Summary
SR 823 over Little Scioto River, Retaining Wall (RW) Section A-A
Wall Supported on Spread Footings

Arm about toe of wall

Load ID	Vert. Or Hor.?	Unfactored Load (kips) per ft width	Arm (ft)	Unfactored Moment (kip-ft) per ft width	Resisting or Overturning	
1	V	25.5	7.75	197	R	Vertical Soil Dead Weight
2	V	3.7	0.75	28	R	Retaining Wall Dead Weight
3	V	8.4	7.00	65	R	Footing Dead Weight
4	V	3.1	7.75	24	R	Vehicle Surcharge - Vertical
5	H	8.5	6.77	66	O	Soil Lateral Load
6	H	1.7	10.15	13	O	Vehicle Surcharge - Horizontal
7a	H	2.0	23.80	16	O	Impact Loading
7b	V	0.0	0.00	0	O	Axial load from structure

Neglect traffic as resisting force

291 Sum of Resisting Moments
94 Sum of Overturning Moments

$$FS = \frac{M_{Resisting}}{M_{Overturning}}$$

FS overturning = 3.08

Footing Width = 14 feet
Footing thickness = 4 feet
Wall Height = 20.3 feet

In Y-direction

Summation Vertical Loads = 37.5 kips X-Direction
Summation Horizontal Loads = 12.2 kips Y-Direction

$\Phi =$ 30 degrees

$K_a =$ 0.33 Soil

$\mu =$ 0.55 on Rock

Thrust = 12.2 kips
Resistance = 20.6 kips

$$FS = \frac{\Sigma F_v \cdot \mu}{\Sigma F_H}$$

FS sliding = 1.70

Sheet 14 of 14
SJK 2-8-08
TKM 11-Feb-08

Force and Moment Summary

SR 823 over Little Scioto River, Retaining Wall (RW) Section B-B

Wall Supported on Spread Footings

Check this section to see if spread footings may be used

Arm about toe of wall

Load ID	Vert. Or Hor.?	Unfactored Load (kips) per ft width	Arm (ft)	Unfactored Moment (kip-ft) per ft width	Resisting or Overturning	
1	V	34.4	7.75	266	R	Vertical Soil Dead Weight
2	V	5.0	0.75	38	R	Retaining Wall Dead Weight
3	V	8.4	7.00	65	R	Footing Dead Weight
4	V	3.1	7.75	24	R	Vehicle Surcharge - Vertical
5	H	13.9	8.67	108	O	Soil Lateral Load
6	H	2.1	13.00	17	O	Vehicle Surcharge - Horizontal
7a	H	2.0	29.50	16	O	Impact Loading
7b	V	0.0	0.00	0	O	Axial load from structure

Neglect traffic as resisting force

370 Sum of Resisting Moments

140 Sum of Overturning Moments

$$FS = \frac{M_{Resisting}}{M_{Overturning}}$$

FS overturning = 2.64

Footing Width = 14 feet

Footing thickness = 4 feet

Wall Height = 26 feet

In Y-direction

Summation Vertical Loads = 47.7 kips X-Direction

Summation Horizontal Loads = 18.1 kips Y-Direction

Φ = 30 degrees K_a = 0.33 Soil

μ = 0.55 on Rock Thrust = 18.1 kips

Resistance = 26.2 kips

$$FS = \frac{\Sigma F_v \cdot \mu}{\Sigma F_H}$$

FS sliding = 1.45

No Good