Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
US 52 Ramp B Over Ohio River Road and US 52
SCI-823-0.00 Portsmouth Bypass (PID 77366)
Scioto County, Ohio

Prepared for:

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

Ohio Department of Transportation
District 9

DLZ Ohio, Inc.
6121 Huntley Road
Columbus, OH 43229
Phone: (614) 888-0040
Fax: (614) 436-0161

DLZ Job No. 0121-3070.03

June 19, 2007

Prepared by:

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

US 52 RAMP B OVER OHIO RIVER ROAD AND US 52

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

SCIOTO COUNTY, OHIO

For:

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

By:

DLZ OHIO, INC.
6121 Huntley Road
Columbus, OH 43229

DLZ Job. No. 0121-3070.03

June 19, 2007

TABLE OF CONTENTS

REPORT
OF
SUBSURFACE EXPLORATION
FOR
BRIDGE AND MSE RETAINING WALLS
US 52 RAMP B OVER OHIO RIVER ROAD AND US 52
PROJECT SCI-823-0.00 PORTSMOUTH BYPASS (PID 77366)
SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of the subsurface exploration and the engineering evaluation of the foundations and mechanically stabilized earth (MSE) retaining walls for US 52 Ramp B bridge over Ohio River Road and US 52 of the Portsmouth bypass project. Subsurface explorations were performed for the other features of the project but the results are presented in separate reports.

This project consists in part of constructing a bridge for proposed US 52 Ramp B over Ohio River Road (CR 503) and US 52. The structure as planned, is a five-span structure (Alternative 4) using MSE walls to hold back the roadway embankments and contain the abutments.

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

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

2.0 GENERAL PROJECT INFORMATION

Based upon the structure site plans, which are presented in Appendix I, it is assumed that the maximum height of the embankment/MSE wall at the rear and forward abutments will be approximately 32.8 and 38.1 feet, respectively. These heights are based upon the maximum difference between the proposed grade of US 52 Ramp B and the existing grade as per the revised profile for Ramp B, received May 15, 2007. In addition, it is understood that the MSE walls will be placed at approximate stations 35+42 and 42+80 for the rear and forward abutments, respectively.

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 eleven structure borings for the proposed bridge and MSE walls. Borings B-48 through B-53 were drilled for the currently proposed structure, as indicated on the structure site plan. These borings were drilled between May 15 and May 24, 2007. Borings TR-62, TR-64, TR-66, TR-71A and TR-73A were drilled for a previous design configuration. Borings TR-62, TR-64 and TR-66 were drilled between March 18 and 30, 2005 and Borings TR-71A and TR-73A were drilled between July 27 and 31, 2006. The subsurface conditions encountered in Boring TR-70A, drilled for MSE walls along Ramp B, were also considered in the analyses of MSE walls at the rear abutment. The boring logs for all borings are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were planned and staked in the field by representatives of DLZ. The surveyed locations and ground surface elevations of the borings were determined by representatives of Lockwood, Lanier, Mathias & Noland, Inc. (2LMN). The surveyed locations of the borings are reflected on the structure site plan presented in Appendix I.

4.0 FINDINGS

4.1 Geology of the Site

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

The genesis of the soils varies across the site. Soils in the floodplain consist primarily of alluvium and alluvial terraces, generally composed of silty clay, coarse sand, gravel, and cobbles. Below approximately elevation 700, the soils on the hillsides are generally lacustrine deposits. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. Bedrock within the structure area is primarily sandstone of the Logan Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to typically above approximately elevation 770.

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. The logs for Borings B-48 through B-51 were considered draft because additional rock core will be obtained at some of these boring locations to confirm the quality of rock below the completion depths of these borings (see Section 5.2 of this report). In addition, the log for Boring B-53 was also considered draft because the composition of fill material encountered in the upper 12 inches of the boring has yet been verified. Laboratory test results are presented on the boring logs and also in Appendix III.

4.2.1 Soil Conditions

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

Boring TR-62 was drilled for the forward abutment and Borings TR-71A and TR-73A were drilled in the vicinity of the rear abutment. Borings B-48 through B-53 were drilled for the piers of the currently proposed structure. Borings TR-64 and TR-66 were drilled near the proposed pier locations.

At the ground surface, Borings B-48, B-51, B-52, TR-64, TR-70A, TR-71A and TR-73A encountered approximately 1 to 10 inches of topsoil and the remainder of the borings, except Boring B-53, encountered approximately 10 to 12 inches of asphalt pavement and/or 4 to 6 inches of aggregate base. In Boring B-53, fill material was encountered between the ground surface and a depth of 3.5 feet.

Below the topsoil, asphalt pavement, aggregate base or fill material, the borings generally encountered natural cohesive soils interbedded with granular soils, except Borings B-48 through B-50, where 6.0 to 7.2 feet of possible fill consisting of primarily of sandy silt (A-4a) and silt (A-4b) were encountered. Generally, the natural cohesive soils consisted of stiff to very stiff sandy silt, soft to very stiff silt (A-4b), and stiff to very stiff silt and clay (A-6a) while the natural granular soils consisted of medium dense gravel with sand and silt (A-2-4) and medium dense to very dense coarse and fine sand (A-3a). Occasionally, medium dense gravel with sand (A-1-b) and gravel with sand, silt and clay (A-2-6) were also encountered. The native soil extended to depths ranging between approximately 3.5 and 17.0 feet below the ground surface, where bedrock was encountered. Note that bedrock was encountered below fills at a depth of 3.5 feet in Boring B-53.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. Severely weathered, argillaceous sandstone was encountered in all borings above the competent sandstone. The bedrock generally consisted of soft to hard, slightly to highly weathered, argillaceous sandstone. The amount of rock recovered in each core run varied between 87 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 0 and 100 percent with an average of 56 percent, indicating fair rock quality. Generally, the RQD values were lower in the upper portion of the bedrock and increased at greater depths.

Unconfined compressive strength of tested rock cores ranged between 5,450 and 11,036 pounds per square inch (psi). The tested rock cores were obtained at depths between 9.3 feet and 21.3 feet below the ground surface. A summary of the unconfined compressive strengths of the tested cores is shown in Table 1. Anticipating the need to design the foundations 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-48	18.2-18.6	150.6	13,427
B-48	26.4-26.8	153.7	10,833
B-49	21.3-21.7	156.2	11,938
B-49	27.6-28.0	149.0	12,784
B-50	25.9-26.3	144.4	7,575
B-51	18.1-18.5	150.1	12,065
B-51	21.3-21.7	145.6	7,605
B-52	19.5-19.9	141.8	11,770
B-52	24.2-24.6	139.6	11,230
B-53	13.4-13.6	156.0	10,290
B-53	17.7-18.1	154.5	14,321
TR-62	9.3-9.7	154.4	10,794
TR-64	19.5-19.8	147.7	12,706
TR-66	22.8-23.3	144.6	11,463
TR-71A	21.6-22.0	143.1	10,209
TR-73A	19.2-19.6	142.6	11,260

4.2.3 Groundwater Conditions

Seepage was observed in Borings B-48 through B-51, TR-66, TR-70A, TR-71A, and TR-73A. There were no measurable water levels in any of the borings prior to rock coring except Boring TR-66, where groundwater was encountered at a depth of 14.0 feet. Measurable water levels were present in all borings upon the completion of coring, at depths between the ground surface of the borings and 8.1 feet. Final water levels include water that was used during rock coring operations. Consequently, any seepage zones that might exist in the rock were masked.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is understood through comments from ODOT's Office of Structural Engineering (OSE) that driven steel H-piles were preferred for supporting the abutments. Prebored CIP piles and drilled shafts could also be considered for the support of the abutments. Spread footings, prebored CIP piles, driven piles and drilled shafts could be considered for supporting the piers. Additionally, it is understood that MSE walls will be used to contain the abutments and hold back the roadway embankment. Recommendations for the foundations, and MSE walls are presented in the following sections.

5.1 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls will be used to construct the approach embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. Given the subsurface conditions, deep foundations were assumed to be used to support the proposed MSE walls; stability analyses and settlement calculations for the proposed MSE walls were performed based on this assumption.

5.1.1 MSE Walls: General Information

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

A global stability analysis and bearing capacity analysis were performed for the MSE walls at this bridge location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding and overturning.

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability and settlement analyses are presented in Appendix IV. Other internal stability analyses (i.e. strap design) are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented in Table 2. In accordance with ODOT guidelines, a unit weight of 120 pounds per cubic foot (pcf) and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. Similarly, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 2- Soil Parameters Used in The MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear Abutment) (Boring TR-73A)	Stiff Silt (A-4b)	120	1667	0	0	29
Foundation Soil (Assumes Undercut) *Bearing on Bedrock*	Compacted Granular Fill	120	0	34*	0	34*

*Sliding analyses for MSE walls on compacted granular fill bearing on Rock use φ'=34°, otherwise use friction angle for compacted granular fill or existing soil below undercut, whichever is less.

5.1.2 MSE Wall Evaluations and Recommendations – Rear Abutment

The rear abutment was considered more critical at this structure due to the 12.0-foot thick soil layer at this location. In contrast, the top of bedrock was encountered at a depth of 3.5 feet at the forward abutment.

An embankment height of 32.8 feet, as shown on the updated cross-section drawings received from TranSystems Corporation on May 15, 2007, was assumed for the analyses of the rear abutment MSE wall. Including the additional embedment depth to the top of the leveling pad, a total wall height of 35.8 feet was assumed for the analyses. Additionally, the soil profile and properties encountered by Boring TR-73A were assumed the analyses of the rear abutment MSE wall.

Boring TR-73A encountered stiff silt and clay (A-6a) and silt (A-4b) to an approximate depth of 10.5 feet below the ground surface. Below this layer, medium dense gravel with sand and silt (A-2-4), and severely weathered sandstone was encountered to an approximate depth of 13.9 feet below the ground surface, at the top of cored bedrock.

Initially, analyses were performed to determine the global stability, bearing capacity and stability (sliding and overturning) of the MSE walls bearing on the existing soils. The results of the analyses indicated that the factors of safety for global stability, sliding, overturning, and drained bearing capacity were adequate. However, bearing capacity calculations indicated that the factor of safety for the undrained bearing capacity was 1.9, which is less than the recommended minimum value of 2.5.

Additional analyses indicated that an adequate factor of safety can be achieved if some of the some of the existing foundation soils are removed and replaced with compacted granular fill. Considering the subsurface conditions at Borings TR-70A, TR-71A and TR-73A, it is recommended that the existing foundations soils be overexcavated to an approximate depth of 5.0 feet below the bottom of the proposed leveling pad, corresponding to an approximate elevation of 539.8 (based on Boring TR-73A). The compacted granular fill below the leveling pad should conform to ODOT Supplemental Specification 840. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by a distance equal to the depth of the aggregate base.

For stability, sliding calculations indicated that a minimum reinforcement length of 0.7 times the full height (H+D), or 25.1 feet is required for stability of the proposed MSE wall at the rear abutment location.

The total maximum settlement (without overexcavation) at the face of the proposed rear abutment MSE wall was estimated to be approximately 5 inches at the centerline of the ramp. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankment loading. Differential settlement at this location was estimated to be approximately 0.9 percent, which is slightly less than the typically cited maximum value. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1.0 percent). The settlement calculations assumed no overexcavation within the MSE wall footprint area. However, overexcavation is recommended to increase the bearing capacity of the MSE foundation soils. If the recommended overexcavation is preformed, the settlement at the face of the proposed rear abutment MSE wall was estimated to be approximately 2 inches at the centerline of the ramp. Differential settlement at this location was estimated to be approximately 0.3 percent.

Table 3 presents the MSE retaining wall parameters and results of analyses at the rear abutment.

Table 3 - MSE Retaining Wall Parameters and Analyses Results (Rear Abutment)

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.00* (Based on $\Phi' = 30°$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 30°(0.67) = 0.39**
Allowable Bearing Capacity – Undrained Condition (Without overexcavation) $q_{all} = 3{,}496$ *psf*
Allowable Bearing Capacity – Drained Condition (With overexcavation) $q_{all} = 7{,}738$ psf
Global Stability *(Without Overexcavation)* Factor of Safety – Undrained Condition = 1.8 Factor of Safety – Drained Condition = 1.6 Factor of Safety – Drained Seismic Condition = 1.5
Estimated Settlement of MSE volume Maximum Total Settlement = 5 inches (Without Overexcavation) Differential Settlement = 0.9% (maximum allowable is 1.0% ODOT BDM 204.6.2.1) Maximum Total Settlement (With Overexcavation) – 2 inches
Full Height of MSE Wall = 35.8 feet (including embedment depth) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability, 0.7(H+D) = 25.1 feet

*For external stability Ka=0.0, back to back wall analyses. Ref: FHWA-NHI-00-043

**Sliding analyses for MSE walls on compacted granular fill bearing on Rock use φ'=34°, otherwise use friction angle for compacted granular fill or existing soil below undercut, whichever is less. In this case, use friction angle of soil below undercut, φ'=30°.

5.1.3 MSE Wall Evaluations and Recommendations – Forward Abutment

An embankment height of 38.1 feet, as shown on the structure site plan, was assumed for the analyses of the forward abutment MSE wall. Including the additional embedment depth to the top of the leveling pad, a total wall height of 41.1 feet was assumed for the analyses. Additionally, the soil profile and properties encountered by Boring TR-62 were assumed for the analyses of the forward abutment MSE wall.

Boring TR-62 encountered very stiff sandy silt (A-4a) to a depth of 3.5 feet below the ground surface. Below the thin soil layer, approximately 2.5 feet of severely weathered sandstone underlain by 10 feet of slightly weathered sandstone were encountered to the completion depth of 16.0 feet.

Given the presence of bedrock at shallow depths, it is recommended that the MSE wall be constructed on bedrock. It is anticipated that significant variations in the elevation of the top of rock will be encountered along the leveling pad. Significant rock excavation may be required on the right side of the forward abutment to construct the leveling pad and the MSE fill. On the left side of the forward abutment, where the top of rock may be below the bottom of the leveling pad elevation, it is recommended that the existing soils be overexcavated to the top of bedrock and replaced with compacted granular fill to the leveling pad elevation. If the leveling pad is founded on bedrock, no embedment into the rock is required. The compacted granular fill below the leveling pad should conform to ODOT Supplemental Specification 840. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by a distance equal to the depth of the aggregate base.

A stability (overturning and sliding) analysis was performed for the proposed MSE wall at the forward abutment location. However, due to the shallow nature of the existing soils at the forward abutment, global stability and settlement analyses were not required, and were assumed to be adequate. For compacted granular fill bearing on bedrock, a friction angle of 34 degrees may be used for internal stability and sliding calculations.

For stability, sliding calculations have indicated that a minimum reinforcement length of 0.7 times the full height (H+D) or 28.8 feet is required for stability of the proposed MSE wall at the forward abutment location.

5.2 Bridge Foundation Recommendations

Table 4 summarizes the foundation recommendations. It should be noted that the bedrock surface varies widely across the project area. The approximate bearing elevations presented in Table 4 indicate the elevations at the boring locations only. Variations in the elevations at which competent bedrock is encountered should be anticipated.

The analyses for drilled shaft foundations at Pier 1 and Pier 2 resulted in bearing elevations below the bottom of the borings drilled at these locations. For the analyses, it was assumed that the quality of the rock below the bearing elevations was at least as good as the rock encountered in the borings. This assumption will be confirmed by obtaining additional rock core at greater depths.

Table 4-Summary of Foundation Recommendations

Structural Element	Structure Borings	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	TR-73A	544.8	HP 12x53-driven	530.9	70 tons
			CIP Piles-prebored	525.9*	Pile Capacity++
			Drilled Shafts	525.9*	40 ksf+++
Pier 1	Left/B-48	542.5	HP 12x53-driven	531.0	70 tons
			Spread Footings	531.0	40 ksf
			CIP Piles-prebored	523.4*	Pile Capacity++
			Drilled Shaft	511.0**	40 ksf+++
	Right/B-49	549.1	HP 12x53-driven	534.1	70 tons
			Spread Footings	534.1	40 ksf
			CIP Piles-prebored	527.1*	Pile Capacity++
			Drilled Shaft	514.1**	40 ksf+++
Pier 2	Left/B-50	549.9	HP 12x53-driven	534.9	70 tons
			Spread Footings	534.9	40 ksf
			CIP Piles-prebored	527.9*	Pile Capacity++
			Drilled Shaft	514.9**	40 ksf+++
	Right/B-51	547.5	HP 12x53-driven	535.5	70 tons
			Spread Footings	535.5	40 ksf
			CIP Piles-prebored	528.0*	Pile Capacity++
			Drilled Shaft	515.5**	40 ksf+++
Pier 3	B-52	551.6	HP 12x53-driven	537.6	70 tons
			Spread Footings	537.6	40 ksf
			CIP Piles-prebored	532.6*	Pile Capacity++
			Drilled Shafts	532.6*	40 ksf+++
Pier 4	B-53	558.7	Spread Footings	552.2	40 ksf
			Drilled Shafts	547.2*	40 ksf+++
Forward Abutment	TR-62	559.1	CIP Piles-prebored	548.1*	Pile Capacity++
			Drilled Shafts	548.1*	40 ksf+++

* A minimum of 5-foot socket into competent rock, assumes no significant lateral loads.
** Drilled shaft tip elevation reflects 20-foot rock socket, design based upon lateral loading.
++ Pile capacity should conform to ODOT BDM 202.2.3.2
+++ End bearing capacity only, refer to section 5.3 for more information

5.2.1 Rear Abutment

If driven H-piles are used at the rear abutment, it is anticipated that HP 12x53 piles would be used and they would be driven to refusal at the top of bedrock.

According to subsurface conditions at Borings TR-71A and TR-73A, the piles would be driven approximately 12 to 14 feet below the existing ground surface. If driven to refusal, the allowable capacity of the pile can be used. Because the piles will be driven to, or very near bedrock, it is recommended that reinforced pile points be used to prevent the piles from being damaged. Piles sleeves should be placed from the bottom of the leveling pad to the pile cap elevation, through the soil reinforced zone of the MSE wall. Piles should be driven through the sleeves after the MSE wall has been constructed up to the pile cap elevation.

As an alternative to driven H-piles, the rear abutment can be supported by cast-in-place (CIP), reinforced concrete piles. The CIP piles would be placed in prebored holes 12 inches larger than the diameter of the pile and a minimum of 5 feet deep into bedrock. After installing the CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment (per OSE). Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment construction. The allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration. Recommended bearing elevations for the CIP pile foundations are presented in Table 4. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

The contractor should anticipate the need for significant bracing of the prebored CIP piles to provide stability and ensure proper alignment of the abutment piles. The contractor should be prepared to perform hand-compaction near the abutment piles as necessary during the construction of the approach embankment.

Due to the relatively low rigidity of the piles compared to drilled shafts, it is anticipated that the piles will provide low resistance to lateral forces. Therefore, the prebored and socketed CIP pile or driven pile foundation systems may not provide sufficient lateral support if significant lateral loads are present.

As an alternative to pile foundations, drilled shafts could also be considered for the support of the rear abutment. It is recommended that the drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 4. For additional recommendations on drilled shafts, refer to Section 5.3.

At this time, it is understood that the use of spread footings may not be feasible at the abutment locations due to the proposed bridge configuration. Consequently, recommendations for spread footings at the rear abutment (bearing in MSE fill) are not presented.

5.2.2 Piers

The currently proposed structure utilizes an integral straddle bent for Piers 1 and 2 and T-type piers for Piers 3 and 4. The elevations of the proposed bottom of footing / pile cap are shown on the structure site plan presented in Appendix I.

5.2.2.1 Pier 1

Considering the subsurface conditions at Borings B-48 and B-49, Boring B-49 was considered more critical and therefore used for the foundation analysis for the pier. Boring B-49 was drilled for the currently proposed right pier foundation. Boring B-34 encountered approximately 13.0 feet of possible fills primarily consisting of silt and clay (A-6a) and silt (A-4b) followed by approximately 12 inches of coarse and fine sand (A-3a) overlying bedrock. Highly weathered, argillaceous sandstone was encountered in this boring below the soil to a depth of 19.9 feet, where more competent bedrock was encountered.

At the proposed foundation elevation of 544.6, Boring B-49 encountered very stiff sandy silt (A-4a). This material would provide considerably less bearing capacity than the underlying bedrock and the footing loads may induce undesirable settlement. Consequently, it is recommended that spread footings, founded on rock be considered to support the pier. These footings should be founded at or below the elevations presented in Table 4, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf).

If the depth of excavation required for the use of spread footings is excessive, pile foundations could be considered for the support of the left pier. Driven HP 12x53, 70-ton piles driven to refusal on bedrock could be considered to support the pier. It is anticipated that piles could be driven to approximate depths of 14 to 17 feet, corresponding to elevations of 528.4 (as per Boring B-48) and 532.1 (as per Boring B-49).

Alternatively, the use of prebored CIP piles could be considered. CIP piles could be prebored into bedrock to provide lateral support. Recommended bearing elevations for the CIP pile foundations are presented in Table 4. Additional recommendations for prebored CIP piles are presented in Section 5.2.1. However, lateral loading and uplift may be a concern for this type of foundation, which may require longer socket lengths.

Given the column arrangement being utilized for the straddle bent pier, a single drilled shaft has been considered for the support of the left pier column. It is understood that preliminary structural designs utilized a 54-

inch reinforced concrete column to support the proposed bent. Preliminary lateral and axial service loads have been provided by TranSystems for the purposes of preliminary design of the laterally loaded drilled shaft. Analyses indicate that a 60-inch diameter column supported by a 60-inch diameter drilled shaft would be required to limit deflections at the top of the column to approximately 0.95 inches. Analyses also indicate that a minimum of 20-foot deep rock socket will be required to resist the lateral loading. This corresponds to a bearing elevation of 511.0 and 514.1 based upon Borings B-48 and B-49, respectively. The drilled shafts should be straight (not belled) and may utilize an allowable bearing pressure of 40 ksf (20 tsf). If additional capacity is required, the drilled shaft may be designed as a friction-type drilled shaft. Recommendations for the design of friction-type drilled shafts are presented in Section 5.3. The minimum 20-foot deep rock socket is required for stability under lateral loading. It may be increased if necessary for axial loading capacity.

If the structural configuration or loading changes, DLZ should be notified so that we may revise our recommendations as required to ensure adequate geotechnical design of the drilled shaft. Calculations for the preliminary design of the drilled shaft are presented in Appendix IV.

5.2.2.2 Pier 2

Considering the subsurface conditions at Borings B-50 and B-51, Boring B-50 was considered more critical and therefore used for the foundation analysis for the pier. Boring B-50 was drilled for the currently proposed left pier foundation. Boring B-50 encountered approximately 7.0 feet of possible fills primarily consisting of sandy silt (A-4a) followed by gravel with sand and silt (A-2-4), silt (A-4b) and coarse and fine sand (A-3a) overlying bedrock. Highly weathered, argillaceous sandstone was encountered in this boring below the soil to a depth of 17.0 feet, where more competent bedrock was encountered.

At the proposed foundation elevation of 545.4, Boring B-50 encountered very stiff sandy silt (A-4a). This material would provide considerably less bearing capacity than the underlying bedrock and the footing loads may induce undesirable settlement. Consequently, it is recommended that spread footings, founded on rock be considered to support the pier. These footings should be founded at or below the elevations presented in Table 4, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf).

If the depth of excavation required for the use of spread footings is excessive, pile foundations could be considered for the support of the left pier. Driven HP 12x53, 70-ton piles driven to refusal on bedrock could be considered to support the pier. It is anticipated that piles could be driven to approximate depths of 15 to 17 feet, corresponding to elevations of 532.5 (as per Boring B-51) and 532.9 (as per Boring B-50).

Alternatively, the use of prebored CIP piles could be considered. CIP piles could be prebored into bedrock to provide lateral support. Recommended bearing elevations for the CIP pile foundations are presented in Table 4. Additional recommendations for prebored CIP piles are presented in Section 5.2.1. However, lateral loading and uplift may be a concern for this type of foundation, which may require longer socket lengths.

Given the column arrangement being utilized for the straddle bent pier, a single drilled shaft has been considered for the support of the left pier column. It is understood that preliminary structural designs utilized a 54-inch reinforced concrete column to support the proposed bent. Preliminary lateral and axial service loads have been provided by TranSystems for the purposes of preliminary design of the laterally loaded drilled shaft. Analyses indicate that a 66-inch diameter column supported by a 66-inch diameter drilled shaft would be required to limit deflections at the top of the column to approximately 0.96 inches. Analyses also indicate that a minimum of 20-foot deep rock socket will be required to resist the lateral loading. This corresponds to a bearing elevation of 514.9 and 515.5 based upon Borings B-50 and B-51, respectively. The drilled shafts should be straight (not belled) and may utilize an allowable bearing pressure of 40 ksf (20 tsf). If additional capacity is required, the drilled shaft may be designed as a friction-type drilled shaft. Recommendations for the design of friction-type drilled shafts are presented in Section 5.3. The minimum 20-foot deep rock socket is required for stability under lateral loading. It may be increased if necessary for axial loading capacity.

If the structural configuration or loading changes, DLZ should be notified so that we may revise our recommendations as required to ensure adequate geotechnical design of the drilled shaft. Calculations for the preliminary design of the drilled shaft are presented in Appendix IV.

5.2.2.3 Pier 3

Boring B-52 was drilled for the currently proposed Pier 3 location. Boring B-52 encountered approximately 5 feet of sandy silt (A-4a) and silt and clay (A-6a) followed by approximately 7.5 feet of gravel with sand (A-1-b) overlying bedrock. Highly weathered, argillaceous sandstone was encountered in this boring below the soil to a depth of 14.0 feet, where more competent bedrock was encountered.

At the proposed foundation elevation of 546.0, Boring B-52 encountered medium dense gravel with sand (A-1-b). This material would provide considerably less bearing capacity than the underlying bedrock and the footing loads may induce undesirable settlement. Consequently, it is recommended that spread footings, founded on rock be considered to support the pier. This footing should be founded at or below elevation 537.6, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf).

If the depth of excavation required for the use of spread footings is excessive, pile foundations could be considered for the support of the left pier. Driven HP 12x53, 70-ton piles driven to refusal on bedrock could be considered to support the pier. It is anticipated that piles could be driven to an approximate depth of 14 feet, corresponding to elevations of 537.6 (as per Boring B-52).

Alternatively, the use of prebored CIP piles could be considered. CIP piles could be prebored into bedrock to provide lateral support. Recommended bearing elevations for the CIP pile foundations are presented in Table 4. Additional recommendations for prebored CIP piles are presented in Section 5.2.1. However, lateral loading and uplift may be a concern for this type of foundation, which may require longer socket lengths.

Due to the relatively low rigidity of the piles compared to drilled shafts, it is anticipated that the piles will provide low resistance to lateral forces. Therefore, the prebored and socketed CIP pile or driven pile foundation systems may not provide sufficient lateral support if significant lateral loads are present.

As an alternative to pile foundations, drilled shafts could also be considered for the support of the rear abutment. It is recommended that the drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 4. For additional recommendations on drilled shafts, refer to Section 5.3.

5.2.2.4 Pier 4

Boring B-53 was drilled for the currently proposed Pier 4 foundation. Boring B-53 encountered only 3.5 feet of fill overlying bedrock. Moderately to highly weathered sandstone argillaceous sandstone was encountered below the soil to the completion depth of 20.3 feet.

Based upon the conditions encountered in boring B-53, it is recommended that spread footings, founded on rock be considered to support the pier. This footing should be founded at or below elevation 552.2, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf). This bearing elevation is slightly below the proposed elevation of the bottom of the pile cap (Elevation 552.9) as shown on the structure site plan.

Consideration should be given to the means and extent of the excavation, which would be required for the use of spread footings. Shoring may be required to avoid the closure of the adjacent Ohio River Road and disruption of existing utilities.

As an alternative to spread footings, drilled shafts could also be considered for the support of the pier. It is recommended that the drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 4. For additional recommendations on drilled shafts, refer to Section 5.3.

5.2.3 Forward Abutment

Boring TR-62, drilled for the currently proposed forward abutment, indicates that approximately 3.0 feet of overburden is present at forward abutment location. Below the soil, approximately 2.5 feet of severely weathered sandstone overlying slightly weathered argillaceous sandstone was encountered to the completion depth of 16.0 feet.

The forward abutment can be supported by cast-in-place (CIP), reinforced concrete piles. Due to the shallow overburden encountered by the boring, the piles should be prebored into bedrock to provide lateral support. The CIP piles would be placed in prebored holes 12 inches larger than the diameter of the pile and a minimum of 5 feet deep into bedrock. After installing the CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment. Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment construction. The allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration. Recommended bearing elevations for the CIP pile foundations are presented in Table 4. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

The contractor should anticipate the need for significant bracing of the prebored CIP piles to provide stability and ensure proper alignment of the abutment piles. The contractor should be prepared to perform hand-compaction near the abutment piles as necessary during the construction of the approach embankment.

Due to the relatively low rigidity of the piles compared to drilled shafts, it is anticipated that the piles will provide low resistance to lateral forces. Therefore, the prebored and socketed CIP pile foundation system may not provide sufficient lateral support if significant lateral loads are present.

As an alternative to a pile foundation, drilled shafts could also be considered for the support of the rear abutment. It is recommended that drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 4. For additional recommendations on drilled shafts, refer to section 5.3.

At this time, it is understood that the use of spread footings may not be feasible at the abutment locations due to the proposed bridge configuration. Consequently, recommendations for spread footings at the forward abutment (bearing in MSE fill) are not presented.

5.3 General Drilled Shaft Recommendations

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

If adequate capacity cannot be developed with reasonable shaft diameter, drilled shafts should be designed as friction-type shafts. Neglecting the overburden, upper two feet and bottom length equal to one diameter of the socket, allowable sidewall shear stress/adhesion of 3,750 pounds per square foot (psf) may be used for the rock socket. If designed as friction-type shafts, the shafts should be designed such that design loads are carried entirely by the rock socket resistance ignoring any end bearing.

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 quickly when exposed to water or left to desiccate, losing its strength quickly. 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.

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. Although no significant seepage was encountered by any of the borings drilled for this project, water could flow into the drilled shaft excavations at other locations during installation particularly within wet zones that may be present in the rock. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

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

5.4 General Earthwork Recommendations

The proposed alignment of US 52 Ramp B over Ohio River Road and US 52 traverses a gently to moderately sloping area. Consequently, fill placement will be required to construct the approach embankments at the abutments. The maximum fill anticipated is approximately 37 feet, near the proposed forward abutment. In addition, excavations up to 16 feet deep may be required for the pier and abutment foundations.

The proposed MSE wall at the forward abutment is located at the base of an existing rock cut. Consequently, it is anticipated that significant excavation into bedrock will be required to accommodate the soil reinforcing straps of the MSE wall. The contractor should be prepared to excavate hard, durable sandstone by blasting or other appropriate means. In places where fill is to be placed on bedrock, a level bench should be cut into the bedrock prior to the placement of fill for stability purposes.

Between 1 to 10 inches of topsoil were encountered at the ground surface. 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 3 feet of subgrade level should also be removed prior to placing fill or pavement materials.

Organic soils were not encountered in any of the borings. However, if organic soils are encountered, it is recommended that at least the top 3 feet of subgrade soil be removed prior to the construction of the new embankment. Overexcavation may need to be deeper if organic soils are encountered at depths greater than three feet.

The embankments should be constructed in accordance with ODOT Items 203. It is anticipated that the embankments will be constructed with side slopes of 2H:1V or flatter. Based on the materials encountered by the borings, the foundation soils are considered adequately stable under the proposed embankment loads.

Excavations 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.

It is recommended that earthwork be performed under continuous observation and testing by a soils technician with the general guidance of a geotechnical engineer.

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.

While excavating for the footings, unsuitable soils may be encountered deeper than indicated by the borings. These unsuitable materials will need to be overexcavated until suitable bearing material is encountered. Overexcavations should be backfilled with compacted engineered fill.

5.5 Groundwater Considerations

Water seepage was encountered in more than 50 percent of the borings. However, a measurable groundwater level was noted only in Boring TR-66 prior to adding drilling water. Representative final water levels could not be obtained due to the use of water during rock coring operations. Foundation construction on top of the rock is expected to encounter only minor seepage. Excavations or shafts extending below ground level may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage and water flow that may enter any excavations.

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.

Eric W. Tse, P.E.
Geotechnical Engineer

Dorothy A. Adams

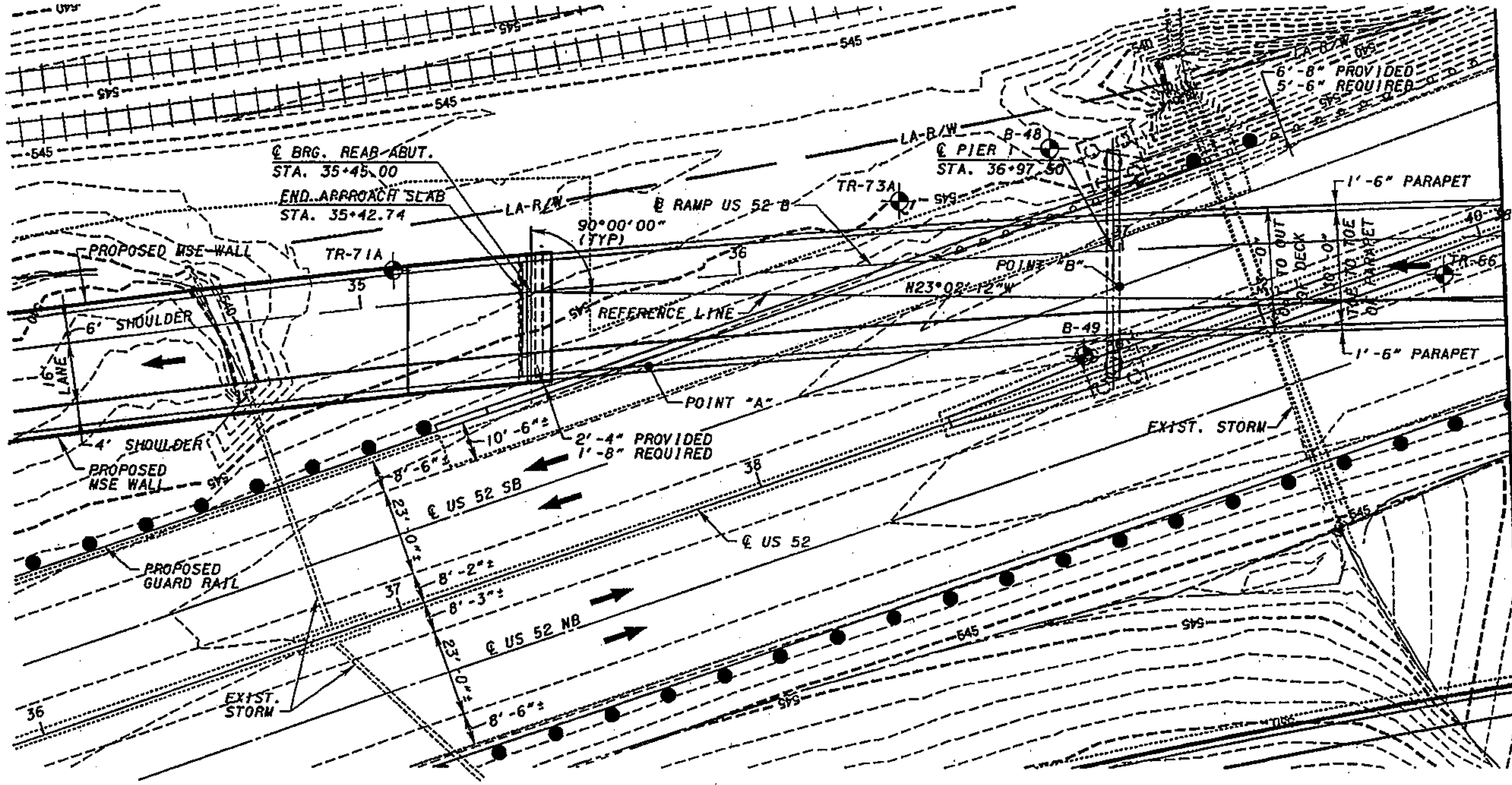
Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

ewt

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\US 52 Ramp B\US 52 Ohio River Road-Structure Report-RAMP B 6-19-07.doc

APPENDIX I

Structure Plan and Profile Drawings - 11"x17"



PLAN

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.

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED IN THE TS&L SUBMITTAL

FOUNDATION DATA:

ALL NEW PILES SHALL BE HP14x73 PILES AND HAVE A MAXIMUM CAPACITY OF 95 TONS. DRILLED SHAFTS SHALL BE 3'-6" DIAMETER WITH 3'-0" DIAMETER ROCK SOCKET AND HAVE AN ALLOWABLE END BEARING CAPACITY OF 40 TSF.

CURVE DATA - US52B-2

P.I. = STA. 42+30.25
Δ = 32°16'24"
Dc = 1°36'00"
R = 3580.99'
T = 1036.08'
L = 2017.08'
E = 146.87'
e_{max} = -0.043
P.C. = 31+94.17
P.T. = 52+11.25

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"	"C"	"D"	"E"
PROPOSED	22.19'	23.25'	28.30'	21.35'	27.65'
PREFERRED	17.0'	17.0'	17.0'	17.0'	17.0'

800' VERT. CURVE DATA

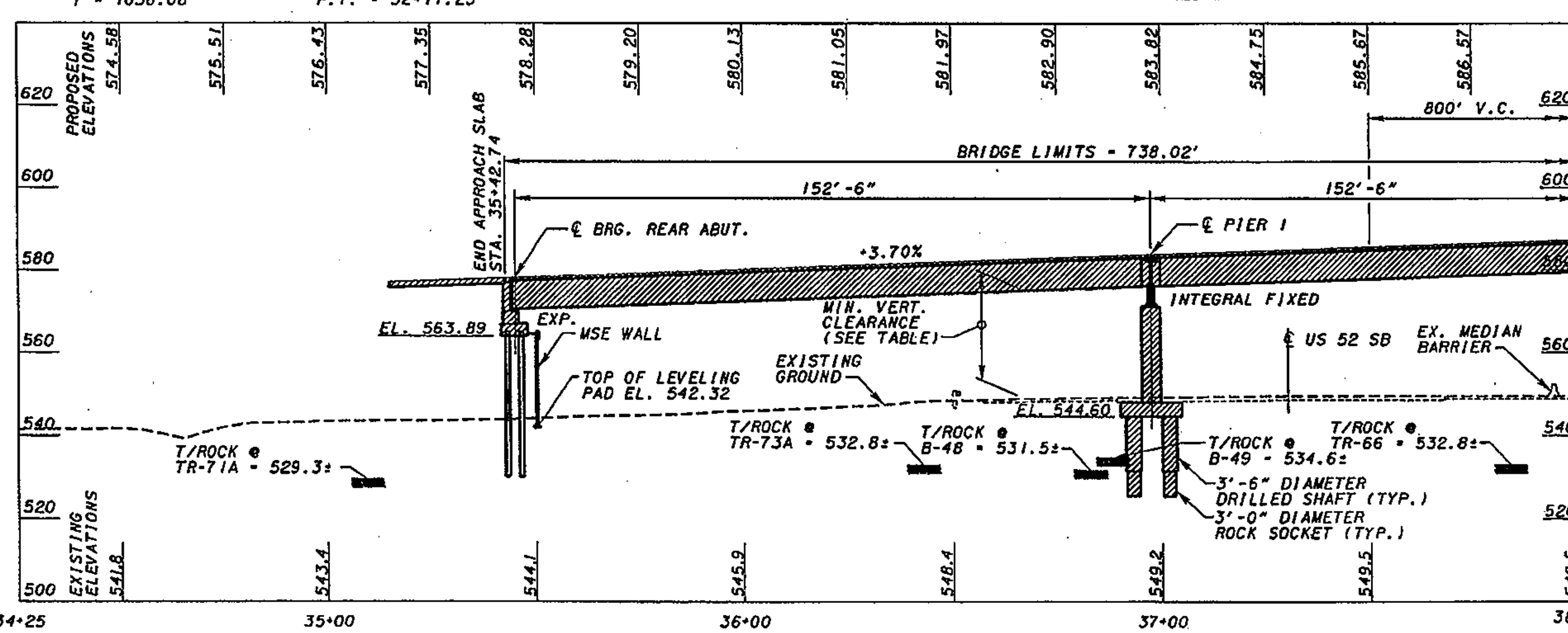
P.V.I. STA. = 41+50.00
P.V.I. ELEV. = 600.45
G_1 = +3.70%
G_2 = -0.87%

ESTIMATED PILE LENGTHS REAR ABUTMENT

ORDER LENGTH - 40'
ESTIMATED LENGTH - 35'

ESTIMATED PILE LENGTHS FORWARD ABUTMENT

ORDER LENGTH - 35'
ESTIMATED LENGTH - 30'



ELEVATION ALONG © RAMP US 52 B

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

TRAFFIC DATA

(RAMP US 52 B)
CURRENT YEAR ADT (2010) = 13400
DESIGN YEAR ADT (2030) = 21000
CURRENT YEAR ADTT (2010) = 1876
DESIGN YEAR ADTT (2030) = 2940

PROPOSED STRUCTURE

TYPE: 5 SPAN CONTINUOUS STEEL PLATE GIRDER A709 GRADE 50W, DOG LEGGED AT SPLICES, WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED ON INTEGRAL PIER AND STUB ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS

SPANS: 152'-6", 152'-6", 145'-0", 142'-0", 141'-6" (ALONG BASELINE RAMP B)

ROADWAY: 30'-0" TOE TO TOE OF PARAPETS

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

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

SUPERELEVATION: 0.043 FT/FT

CROWN:

ALIGNMENT: 1°36'00" CURVE TO THE RIGHT

WEARING SURFACE: MONOLITHIC CONCRETE

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

LATITUDE:

LONGITUDE:

DESIGN AGENCY: Systems
DESIGNED MSL | CHECKED PJP | DRAWN JDG | REVISED | REVIEWED JRC | DATE 3/05/07 | STRUCTURE FILE NUMBER

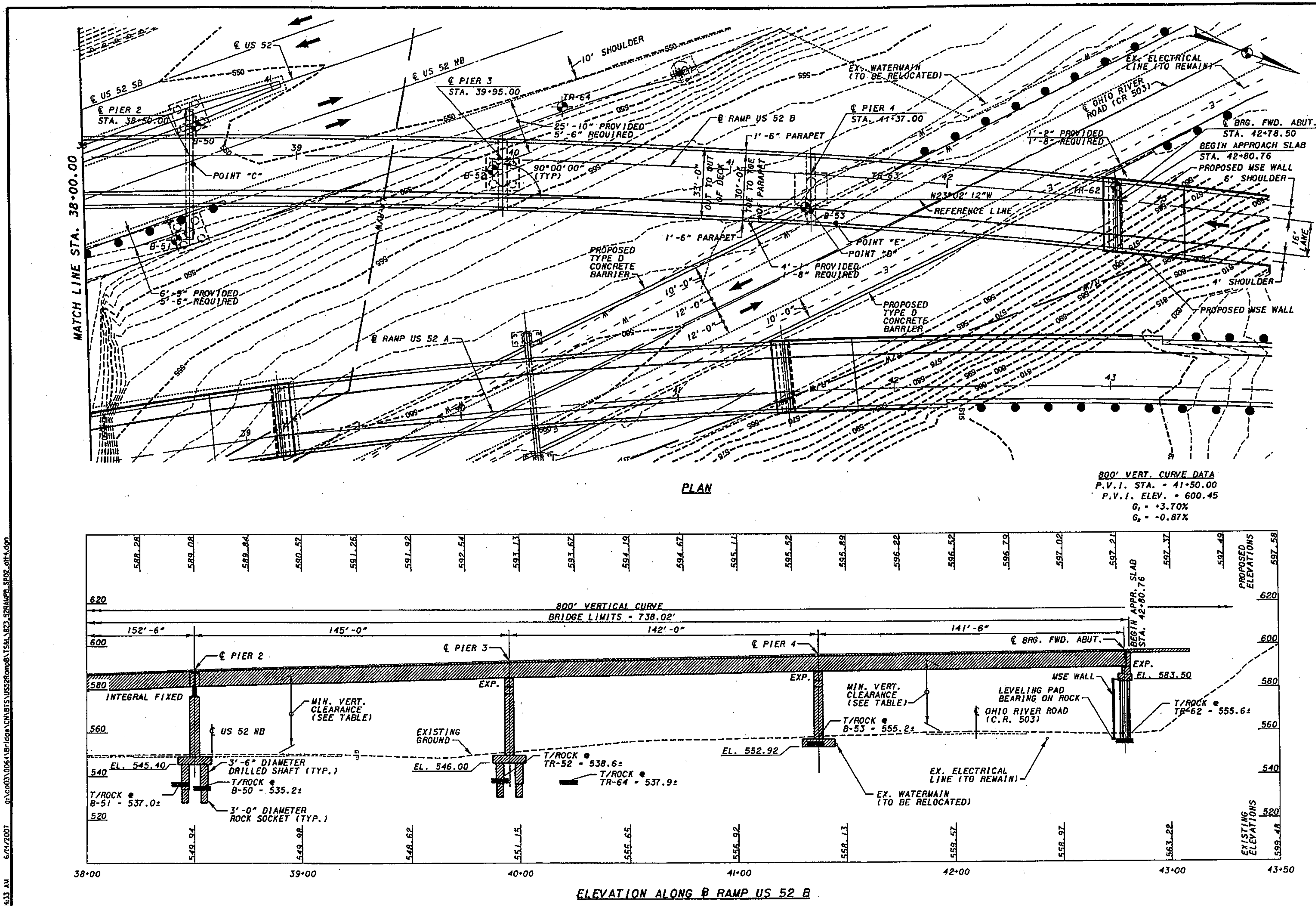
SCIOTO COUNTY
STA. 35+42.74
STA. 42+80.76

PRELIMINARY SITE PLAN - ALT 4
BRIDGE NO. US-52-XXXX
US-52 RAMP B TO SOUTHBOUND S.R. 823

SCI-823-0.00
PID 77366

1/5

9:03:31 AM 6/14/2007 q:\co03\0064\Bridge\CNB\TS\US52RampB\TS&L\823_52RAMPB_SPOLalt4.dgn



DESIGNED MSL | DRAWN JDG | REVIEWED JRC | DATE 3/05/07 | CHECKED PJP

SCIOTO COUNTY
STA. 35+42.74
STA. 42+80.76

PRELIMINARY SITE PLAN - ALT 4
BRIDGE NO. US-52-XXXX
US-52 RAMP B TO SOUTHBOUND S.R. 823

SCI-823-0.00
PID 77366

2/5

g:\co03\0064\Bridge\CN\BTS\US52RampB\TSL\823_52RAMPB_SP02_alt4.dgn 6/14/2007 9:14:33 AM

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Twelve (12) 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 to 2 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

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

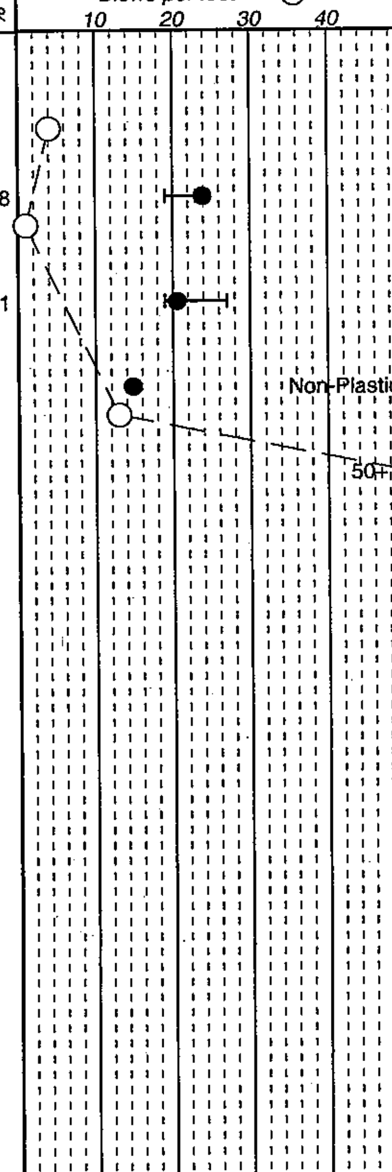
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-48 | Location: Sta. 36+82.8, 27.7 ft. LT of US 52 Ramp B BL | Date Drilled: 05/15/07

WATER OBSERVATIONS: Water seepage at: 8.5'-10.0'
Water level at completion: 3.8' (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
	542.5												
0.1	542.4						Topsoil - 1"						
		2 2 2	15	1		1.5	POSSIBLE FILL: Stiff brown and black SANDY SILT (A-4a), trace fine to coarse sand; slightly organic; contains roots; moist to wet.						
3.0	539.5	1 1 WOH	18	2		0.75	POSSIBLE FILL: Medium stiff brown SILT (A-4b), little to some clay, trace gravel; moist.	7	3	--	5	68	18
					ST-1			1	7	--	8	63	21
7.0	535.5	2 3 10	16	3			Medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); moist.	57	10	--	7	26	
11.0	531.5	50/5	5	4			Severely weathered gray SILTSTONE.						
11.5	531.0	Core 30"	Rec 30"	RQD 0%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, carbonaceous, argillaceous, broken.						
14.1	528.4	Core 60"	Rec 60"	RQD 74%	R-2		Soft to medium hard gray SANDSTONE; very fine to fine grained, highly weathered to decomposed, argillaceous, broken.						
15.4	527.1						Medium hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, pyritic, highly fractured. @ 16.0', 16.3', 16.6', clay filled fractures.						
16.8	525.7	Core 120"	Rec 120"	RQD 100%	R-3		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, highly fractured. @ 17.1', 17.6', 18.7', high angle fractures. @ 18.2', qu=13,427 psi. @ 19.0', moderately to highly fractured, slightly weathered. @ 26.4', qu=10,833 psi, Er=3,614,000 psi.						
29.0	513.5						Bottom of Boring - 29.0'						

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



DRAFT

FILE: 0121-3070-03 [6/15/2007 3:51 PM]

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

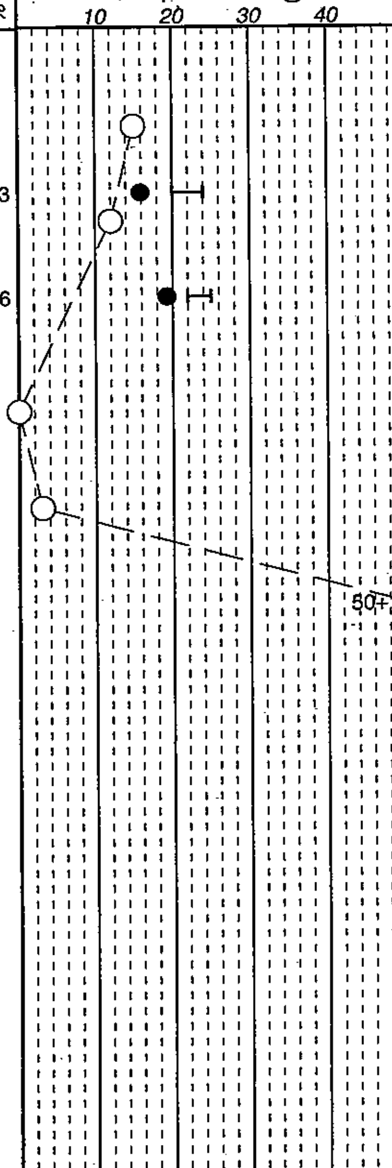
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-49 | Location: Sta. 36+88.1, 27.3 ft. RT of US 52 Ramp B BL | Date Drilled: 05/23/07

WATER OBSERVATIONS: Water seepage at: 13.9'-14.1'
Water level at completion: 3.8' (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)
0	549.1						Asphalt Concrete Pavement - 12" Aggregate Base - 4"							
1.3	547.8	9 6 9	18	1A 1B		4.0	POSSIBLE FILL: Medium dense brown COARSE AND FINE SAND (A-3a), trace to little gravel; damp.							
2.0	547.1	5 5 7	18	2		4.0	POSSIBLE FILL: Very stiff to hard gray SANDY SILT (A-4a), some fine to coarse sand, trace gravel; contains rock fragments; damp .	3	3	--	27	44	23	
6.0	543.1				ST-1	4.0	POSSIBLE FILL: Very stiff to hard gray SILT (A-4b), little fine to coarse sand, trace gravel; damp.	4	7	--	9	64	16	
8.5	540.6	W O H	14	3		0.5	Soft to medium stiff brown and gray SILT (A-4b), little to some fine sand, trace gravel; little clay; contains roots; damp to moist.							
		WOH 1 2	16	4		0.5	**DRAFT**							
13.5	535.6	6 19 50/3	15	5			Very dense brown COARSE AND FINE SAND (A-3a), little silty clay, little gravel; contains rock fragments; damp.							50+
14.5	534.6						Severely weathered brown SANDSTONE.							
15.0	534.1	Core 24"	Rec 23"	RQD 0%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, highly weathered, argillaceous, highly fractured to broken; contains clay filled low angle fractures.							
		Core 60"	Rec 60"	RQD 26%	R-2									
19.9	529.2						Medium hard gray SANDSTONE; very fine to fine grained, slightly weathered, slightly argillaceous, moderately to highly fractured. @ 20.2'-20.8', decomposed argillaceous zone, broken. @ 21.3', qu=11,938 psi. @ 23.5', moderately fractured.							
25		Core 60"	Rec 60"	RQD 80%	R-3		@ 25.5'-26.2', high angle fractures.							
		Core 36"	Rec 36"	RQD 100%	R-4		@ 27.6', qu=12,784 psi, Er=4,772,258 psi.							
30.0	519.1						Bottom of Boring - 30.0'							

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



FILE: 0121-3070-03 [6/15/2007 3:51 PM]

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

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

LOG OF: Boring B-49 | Location: Sta. 36+88.1, 27.3 ft. RT of US 52 Ramp B BL | Date Drilled: 05/23/07

WATER OBSERVATIONS: Water seepage at: 13.9'-14.1'
Water level at completion: 3.8' (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) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	519.1													
35														
40														
45														
50														
55														
60														

DRAFT

FILE: 0121-3070-03 [6/15/2007 3:51 PM]

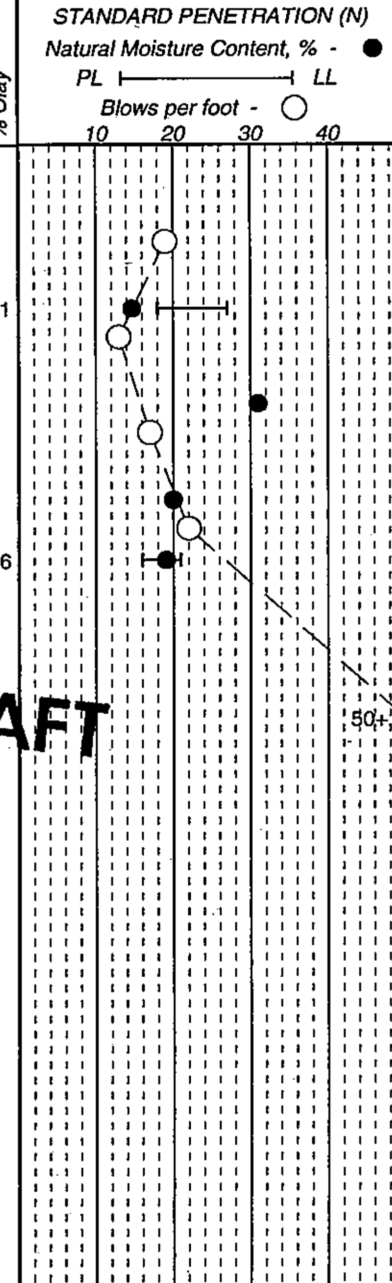
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-50 | Location: Sta. 38+53.3, 13.5 ft. LT of US 52 Ramp B BL | Date Drilled: 05/23/07

WATER OBSERVATIONS: Water seepage at: 1.9'-2.0', 13.5'-14.7'
Water level at completion: 8.1' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	549.9						Asphalt Concrete Pavement - 12" Aggregate Base - 5"						
1.4	548.5	11 9 10	18	1			POSSIBLE FILL: Medium dense brown (A-3a), little gravel; damp.						
2.0	547.9	4 5 8	18	2		4.0	POSSIBLE FILL: Very stiff to hard mottled brown and gray SANDY SILT (A-4a), little fine to coarse sand, trace to little gravel; damp. @ 3.5'-6.0', gray. @ 6.0'-8.5', brown, contains rock fragments.	9	17	--	10	43	21
		2 8 9	18	3		4.5							
8.5	541.4	6 10 12	18	4			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), little fine to coarse sand, little silt; damp.						
10.0	539.9				ST-1	1.0	Medium stiff to stiff brown SILT (A-4b), some fine to coarse sand, little clay; moist.	0	4	--	29	51	16
13.0	536.9	5 10 50/6	18	5A 5B			Very dense brown COARSE AND FINE SAND (A-3a), little silt, little gravel; moist to wet.						
14.7	535.2						Severely weathered brown SANDSTONE.						
15.0	534.9	Core 24"	Rec 22"	RQD 0%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, broken, contains iron staining.						
18.5	531.4	Core 60"	Rec 60"	RQD 15%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, slightly argillaceous, highly fractured. @ 19.9'-20.3', argillaceous zone, highly weathered. @ 20.6', 21.2', argillaceous zones, low angle fractures.						
		Core 60"	Rec 60"	RQD 93%	R-3		@ 22.9', moderately fractured. @25.9', qu=7,575 psi, Er=2,524,923 psi.						
30.0	519.9	Core 36"	Rec 35"	RQD 97%	R-4		Bottom of Boring - 30.0'						



DRAFT

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-50 | *Location:* Sta. 38+53.3, 13.5 ft. LT of US 52 Ramp B BL | *Date Drilled:* 05/23/07

WATER OBSERVATIONS: Water seepage at: 1.9'-2.0', 13.5'-14.7'
Water level at completion: 8.1' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	519.9													
35							DRAFT							
40														
45														
50														
55														
60														

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

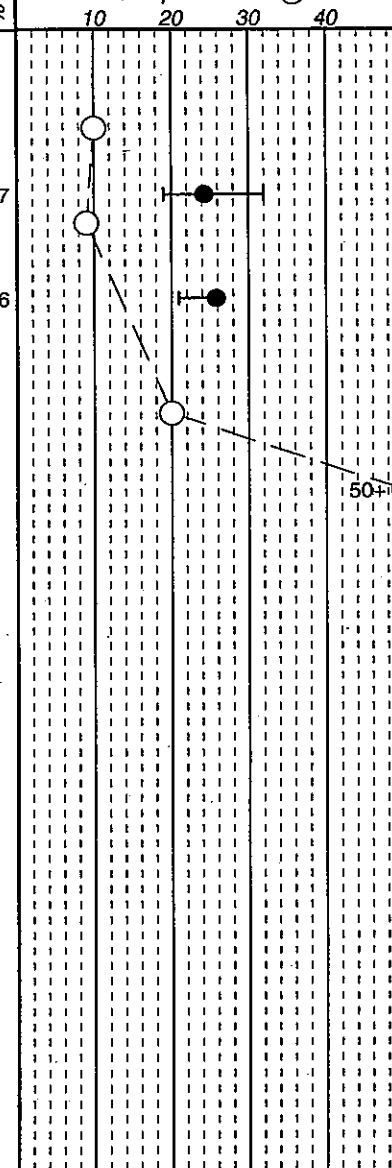
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-51 | Location: Sta. 38+43.3, 38.7 ft. RT of US 52 Ramp B BL | Date Drilled: 05/16/07

WATER OBSERVATIONS: Water seepage at: 9.2'
Water level at completion: 7.5' (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	547.5												
0.3	547.2						Topsoil - 3"						
		7 5 5	16	1		2.5	Very stiff brown SANDY SILT (A-4a), little clay, trace gravel; damp.						
3.0	544.5	3 4 5	18	2		2.5	Very stiff brown SILT AND CLAY (A-6a), trace fine sand; moist.	0	0	--	3	70	27
6.0	541.5				ST-1	2.5	Very stiff brown SILT (A-4b), little clay, trace fine sand; moist to wet.	0	0	--	8	76	16
8.0	539.5	7 10 10	18	3			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), little clay; damp.						
10.5	537.0	42 50/3	9	4			Severely weathered brown SANDSTONE.						
12.0	535.5	Core 30"	Rec 30"	RQD 0%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, highly weathered, micaceous, broken. @ 12.7'-15.6', iron staining, gray, highly fractured.						
15.6	531.9	Core 60"	Rec 60"	RQD 81%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, highly fractured. @ 18.1', qu=12,065 psi. @ 18.2'-18.8', contains argillaceous fractures. @ 19.5', moderately to highly fractured, slightly weathered. @ 21.3', qu=7,605 psi, Er=3,710,318 psi.						
		Core 90"	Rec 90"	RQD 96%	R-3								
27.0	520.5						Bottom of Boring - 27.0'						

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



DRAFT

FILE: 0121-3070-03 [6/15/2007 3:51 PM]

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

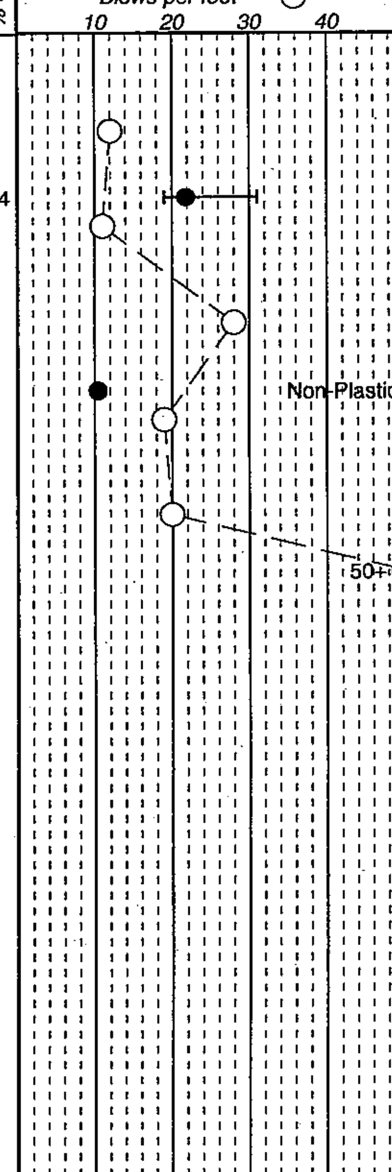
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-52 | *Location:* Sta. 39+90.5, 5.0 ft. RT of US 52 Ramp B BL | *Date Drilled:* 05/16/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 5.5' (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	551.6												
0.3	551.3						Topsoil - 4"						
		4 6 6	12	1		4.5+	Hard brown SANDY SILT (A-4a), little to some clay; contains rock fragments; dry to damp.						
3.0	548.6	3 4 7	18	2		4.25	Hard brown SILT AND CLAY (A-6a), trace fine to coarse sand; moist.	0	1	--	4	62	34
5.5	546.1	10 13 15	17	3			Medium dense brown GRAVEL WITH SAND (A-1-b), trace to little clay; damp.						
		7 10 9	18	4				57	12	--	18	14	
		7 10 10	18	5			@ 10.5', little silt.						
13.0	538.6	50/3	3	6			Severely weathered brownish gray SANDSTONE.						
14.0	537.6	Core 60"	Rec 60"	RQD 30%	R-1		Medium hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, highly fractured to broken, contains iron stained, fractured.						
17.9	533.7	Core 120"	Rec 120"	RQD 90%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, moderately to highly fractured. @ 19.5', qu=11,770 psi, Er=4,965,071 psi. @ 20.3'-20.8', argillaceous zone. @ 24.2', qu=11,230 psi. @ 24.2', moderately fractured.						
29.0	522.6						Bottom of Boring - 29.0'						

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



FILE: 0121-3070-03 [6/15/2007 3:51 PM]

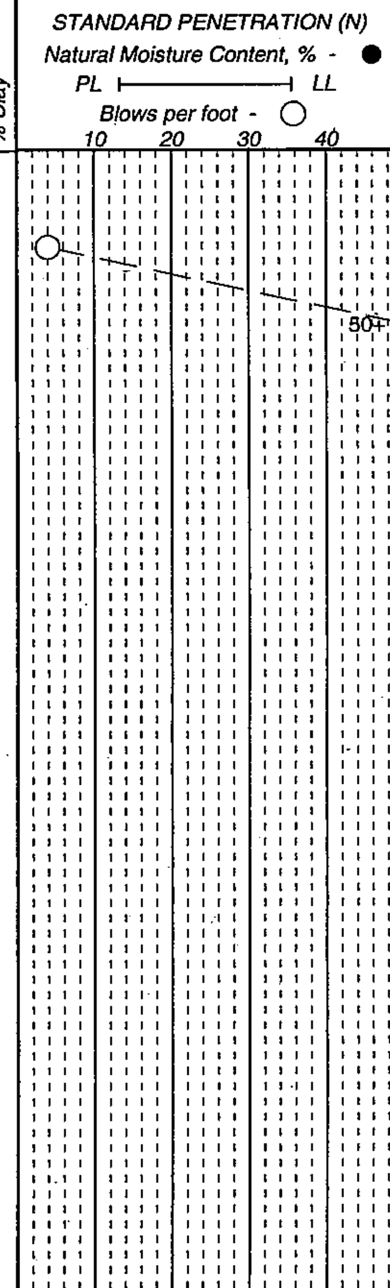
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-53 | *Location:* Sta. 41+35.4, 15.5 ft. RT of US 52 Ramp B BL | *Date Drilled:* 05/24/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 3.9' (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	558.7						Fill: 12"						
1.0	557.7	3 2 2	14	1			FILL: Very loose to loose brown GRAVEL WITH SAND AND SILT (A-2-4), little fine to coarse sand, little silt; contains roots; damp.						
3.5	555.2	40 50/3	8	2			Severely weathered brown SANDSTONE; contains iron staining.						
5.0	553.7	Core 51"	Rec 49"	RQD 54%	R-1		Soft to medium hard brown SANDSTONE; very fine to fine grained, highly weathered, argillaceous, highly fractured to broken, contains abundant low angle fractures. @ 5.0'-6.5', decomposed, very argillaceous (soil like).						
9.9	548.8	Core 60"	Rec 60"	RQD 68%	R-2		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, slightly argillaceous, highly fractured. @ 10.0', 10.5', argillaceous low angle fractures. @ 10.8-11.4', iron stained, high angle fractures. @ 12.1'-12.7', iron stained, high angle fractures. @ 13.4', qu=10,290 psi.						
15		Core 72"	Rec 72"	RQD 91%	R-3		@ 15.5', moderately to highly fractured. @ 17.7', qu=14,321 psi.						
20.3	538.5						Bottom of Boring - 20.3'						
25													
30													



DRAFT

FILE: 0121-3070-03 [6/15/2007 3:51 PM]

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

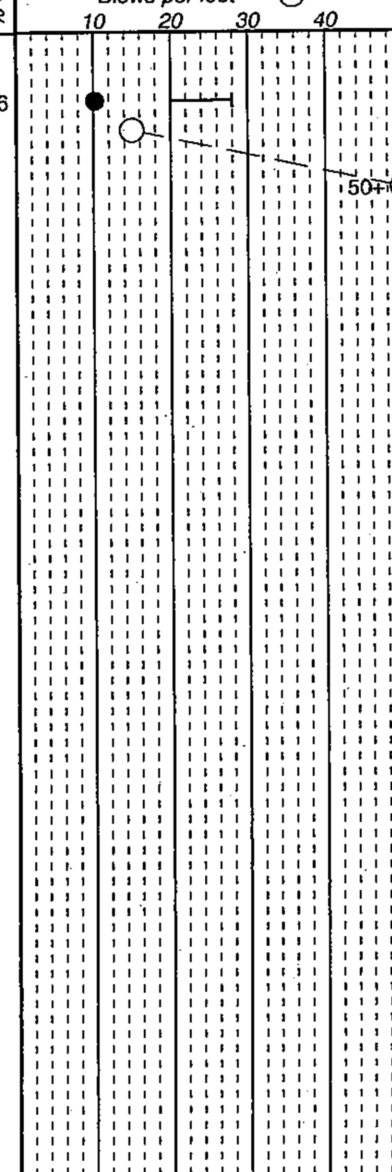
Client: TranSystems, Inc. *Project:* SCI-823-0.00 *Job No.* 0121-3070.03

LOG OF: Boring TR-62 *Location:* Sta. 42+78.1, 6.5 ft. LT of US 52 Ramp B BL *Date Drilled:* 3/18/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
1.9' (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	559.1												
0.5	558.6						Aggregate base - 6"						
		8 9 6	14	1		3.5	Very stiff gray SANDY SILT (A-4a), little gravel, little clay; contains sandstone fragments; damp.	17	15	--	6	46	16
3.5	555.6	50/2	2	2			Severely weathered gray SANDSTONE.						
6.0	553.1	Core 120"	Rec 120"	RQD 78%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thinly bedded, slightly fractured. @ 9.3', qu = 10,794 psi. @ 11.2'-11.3', high angle fracture.						
16.0	543.1						Bottom of Boring - 16.0'						

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



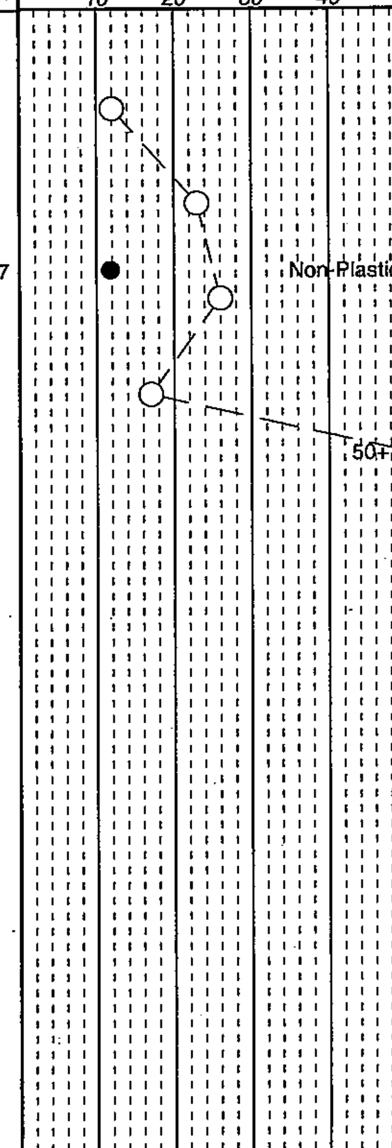
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-64 | Location: Sta. 40+21.8, 25.3 ft. LT of US 52 Ramp B BL | Date Drilled: 3/30/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 0.0' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	548.4												
0.1	548.3						Topsoil - 1"						
		6 7 5	14	1			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; contains sandstone fragments; damp to moist.						
		7 11 12	16	2									
		7 11 15	17	3				21	19	--	25	28	7
		4 7 10	16	4									
10.5	537.9	50/4	2	5			Severely weathered brown SANDSTONE.						
11.5	536.9	Core 120"	Rec 118"	RQD 11%	R1		Hard brown and gray SANDSTONE; very fine to fine grained, moderately to highly weathered, thickly bedded to massive, highly fractured. @ 15.3', clay seam @ 15.7', gray. @ 19.5', qu = 12,706 psi.						
21.5	526.9						Bottom of Boring - 21.5'						



FILE: 0121-3070-03 (6/15/2007 3:51 PM)

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

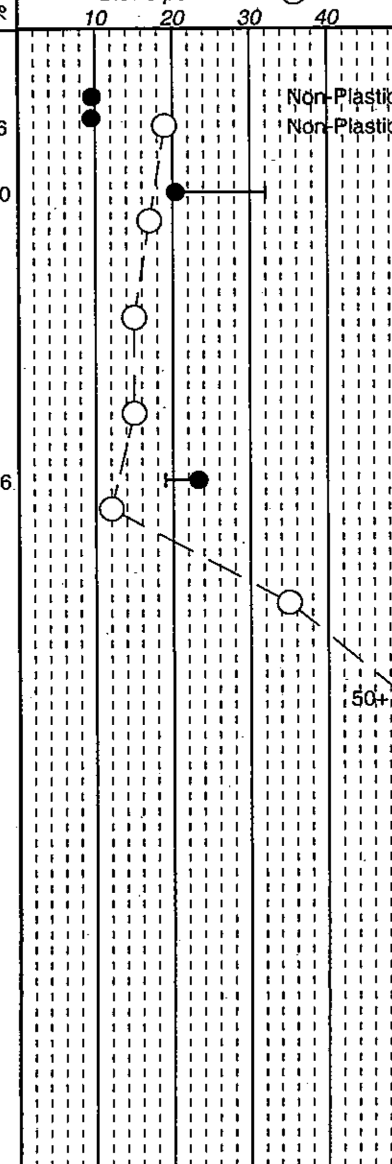
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-66 | Location: Sta. 37+84.0, 9.7 ft. RT of US 52 Ramp B BL | Date Drilled: 3/30/05

WATER OBSERVATIONS: Water seepage at: 16.0'-17.0'
Water level at completion: 14.0' (prior to coring)
8.0' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	549.8						Asphalt Concrete Pavement - 10"						
0.8	549.0												
1.3	548.5	11 10 9	12	1A			Medium dense gray GRAVEL WITH SAND (A-1-b), trace silty clay; damp.	44	28	--	19	9	
				1B				19	27	--	27	21	6
3.0	546.8						Medium dense brown COARSE AND FINE SAND (A-3a), some silt, little gravel, trace clay; damp.						
		5 7 10	13	2		3.5	Very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand; damp to moist.	0	1	--	12	57	30
5.5	544.3						Stiff to very stiff brown SILT (A-4b), little clay, little fine sand, trace coarse sand; contains sandstone fragments; damp.						
		10 7 8	10	3		2.0							
		7 10 5	9	4		2.0							
10													
		8 5 7	17	5		2.5	@ 11.0'-12.5', damp to moist.	0	1	--	19	64	16
		3 6 29	14	6									
15													
17.0	532.8	11 18 50/3	13	7			Severely weathered brown SANDSTONE.						
17.5	532.3						Hard brown SANDSTONE; very fine to fine grained, slightly to highly weathered, argillaceous, micaceous, thickly bedded to massive, moderately to highly fractured. @ 17.5'-20.0', broken. @ 19.1', gray.						
20		Core 120"	Rec 120"	RQD 58%	R1		@ 22.8', qu = 11,463 psi.						
25													
27.5	522.3						Bottom of Boring - 27.5'						
30													

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



FILE: 0121-3070-03 [6/15/2007 3:51 PM]

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

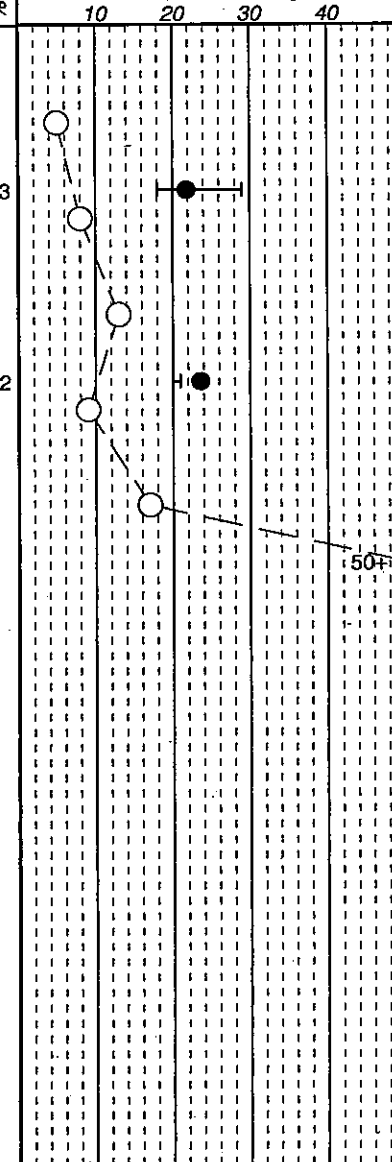
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-70A | Location: Sta. 33+52.3, 5.3 ft. LT of US 52 Ramp B BL | Date Drilled: 07/31/06 to 08/01/06

WATER OBSERVATIONS: Water seepage at: 9.5' - 12.5'
Water level at completion: None (prior to coring)
5.9' (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	540.6						Topsoil - 10"						
0.8	539.8	1 2 3	6	1		1.5	Stiff brown SILT AND CLAY (A-6a), trace to little fine sand; damp to moist.						
							@ 3.0', trace organics.						
		3 4 4	18	2		1.25		2	4	--	14	57	23
							@ 5.5', little fine sand, trace coarse sand, moist.						
		5 8 5	18	3		1.0							
8.0	532.6	4 4 5	18	4			Loose to medium dense brown SANDY SILT (A-4a), trace to little clay; moist.	6	1	--	23	58	12
							@ 10.5', trace to little gravel.						
		4 8 9	18	5									
13.5	527.1	50/3	3	6			Severely weathered brownish gray SANDSTONE.						
14.5	526.1	Core 120"	Rec 104"	RQD 52%	R-1		Medium hard brown SANDSTONE; fine grained, moderately to highly weathered, micaceous, highly fractured. @ 14.5'-15.1', possible core loss. @ 16.1', high angle fracture. @ 18.6', gray. @ 20.2', high angle fracture.						
20.9	519.7						Hard gray SANDSTONE; fine grained, slightly weathered, micaceous, moderately fractured.						
24.5	516.1						Bottom of Boring - 24.5'						

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



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

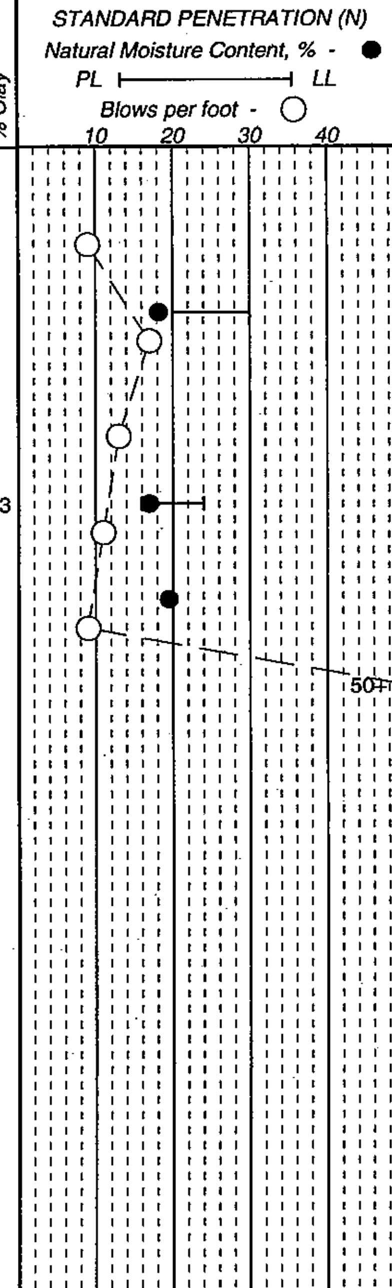
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-71A | Location: Sta. 35+09.8, 9.1 ft. LT of US 52 Ramp B BL | Date Drilled: 07/31/06

WATER OBSERVATIONS: Water seepage at: 9.5'-12.5'
Water level at completion: None (prior to coring)
3.3' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	542.8												
0.5	542.3						Topsoil - 6"						
		4 5 4	15	1		2.25	Stiff to very stiff brown SANDY SILT (A-4a); damp to moist.						
							@ 3.0', mottled brown and gray.						
		3 7 10	16	2		2.25		0	1	--	8	91	
		3 5 8	18	3		3.0							
		2 3 8	18	4		--		14	8	--	23	42	13
							@ 10.5', moist to wet.						
		1 4 5	18	5		--							
13.5	529.3	50/3	2	6			Severely weathered brown SANDSTONE.						
13.9	528.9	Core 120"	Rec 110"	RQD 49%	R-1		Medium hard brown SANDSTONE; fine to medium grained, moderately to highly weathered, broken, contains argillaceous seams. @ 16.7', high angle fracture. @ 16.9', highly fractured.						
19.8	523.0						Hard gray SANDSTONE; fine to medium grained, slightly to moderately weathered, pyritic (halos), micaceous, thickly bedded to massive, highly to moderately fractured. @ 21.6', qu = 10,209 psi.						
23.9	518.9						Bottom of Boring - 23.9'						

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



FILE: 0121-3070-03 (6/15/2007 3:51 PM)

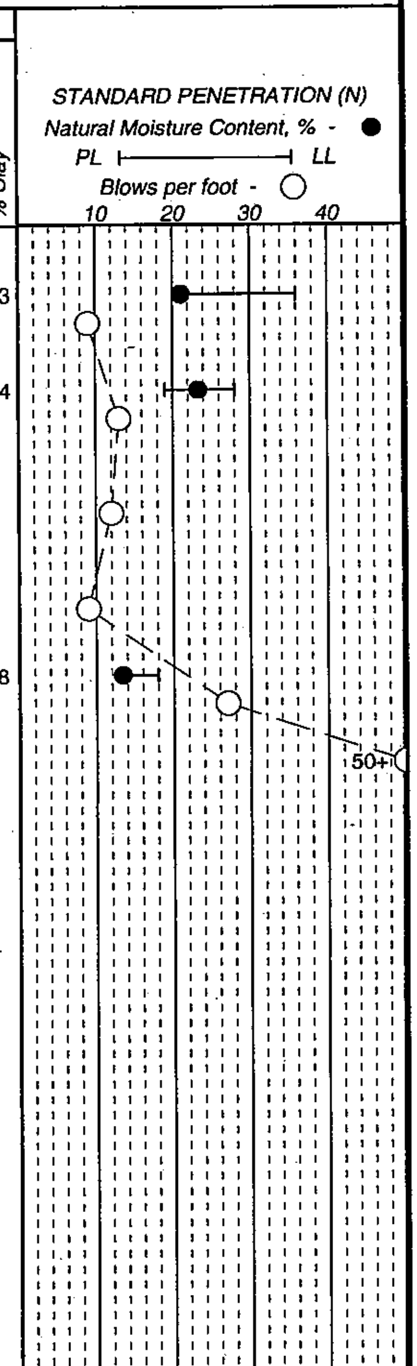
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-73A | Location: Sta. 36+42.9, 16.2 ft. LT of US 52 Ramp B BL | Date Drilled: 07/27/06

WATER OBSERVATIONS: Water seepage at: 7.3'-7.4', 11.0'-12.0'
Water level at completion: None (prior to coring)
1.6' (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
0	544.8												
0.5	544.3						Topsoil - 6"						
		4 4 5	16	1			Stiff to very stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp to moist.	1	1	--	4	61	33
3.0	541.8	4 7 6	18	2		2.0	Stiff brown SILT (A-4b), some clay, trace fine to coarse sand, trace gravel; moist.	1	1	--	7	67	24
		4 4 8	18	3		1.5							
		3 5 4	18	4		1.5							
10.5	534.3	5 8 19	18	5A 5B			Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4); damp to moist.	28	15	--	27	22	8
12.0	532.8						Severely weathered brown SANDSTONE.						
		50/3	3	6									
13.9	530.9	Core 120"	Rec 107"	RQD 55%	R-1		Medium hard brown SANDSTONE; fine grained, highly weathered, micaceous, thickly bedded, broken, contains clay filled seams. @ 16.3'-17.9', argillaceous.						
19.2	525.6						Hard gray SANDSTONE; fine grained, slightly weathered, thickly bedded to massive, slightly fractured. @ 19.2'-19.6', qu = 11,260 psi.						
23.9	520.9						Bottom of Boring - 23.9'						

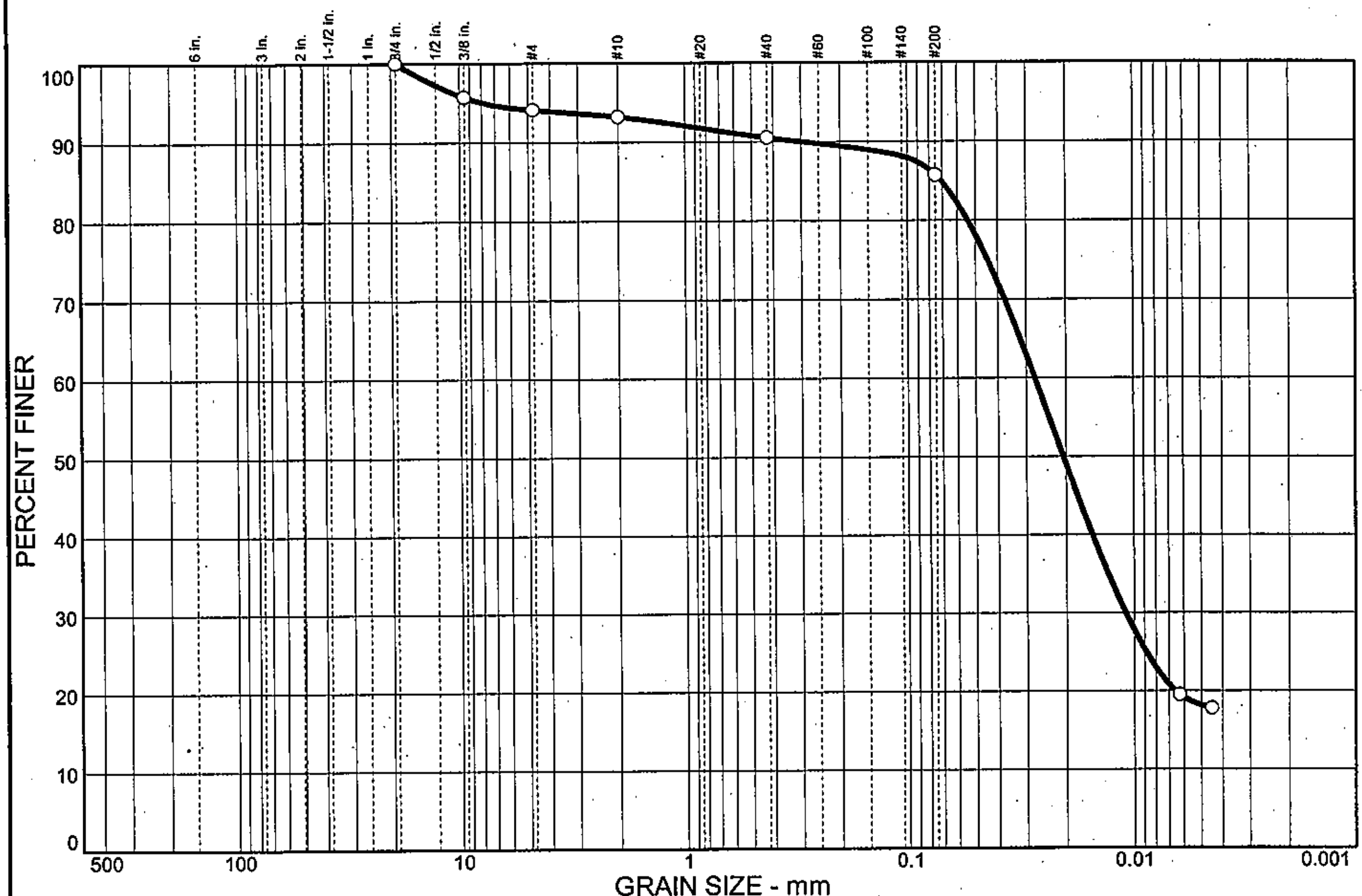


FILE: 0121-3070-03 [6/15/2007 3:51 PM]

APPENDIX III

Laboratory Test Results

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.9	0.9	2.7	4.8	67.6	18.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	95.7		
#4	94.1		
#10	93.2		
#40	90.5		
#200	85.7		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 19 LL= 23 PI= 4

Coefficients

D_{85}= 0.0711 D_{60}= 0.0275 D_{50}= 0.0206
D_{30}= 0.0109 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 23.9%

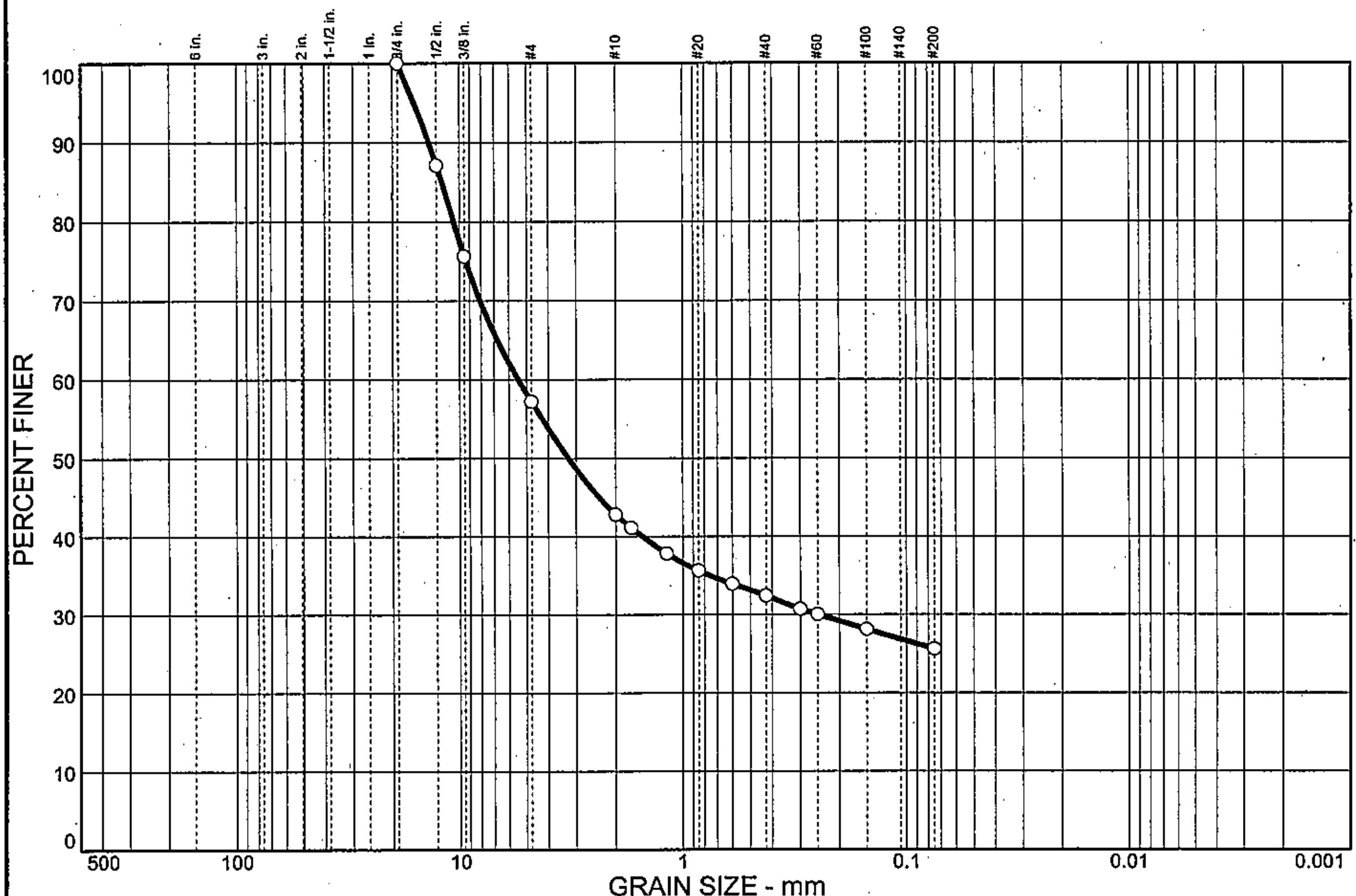
Sample No.: 2 **Source of Sample:** B-48 **Date:** 6/12/07
Location: **Elev./Depth:** 3.5

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	42.8	14.4	10.4	6.8	25.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.50 in.	87.1		
0.375 in.	75.6		
#4	57.2		
#10	42.8		
#12	41.1		
#16	37.8		
#20	35.6		
#30	33.9		
#40	32.4		
#50	30.7		
#60	30.0		
#100	28.1		
#200	25.6		

* (no specification provided)

Soil Description

Silty gravel with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 12.0 D_{60}= 5.44 D_{50}= 3.25
D_{30}= 0.250 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

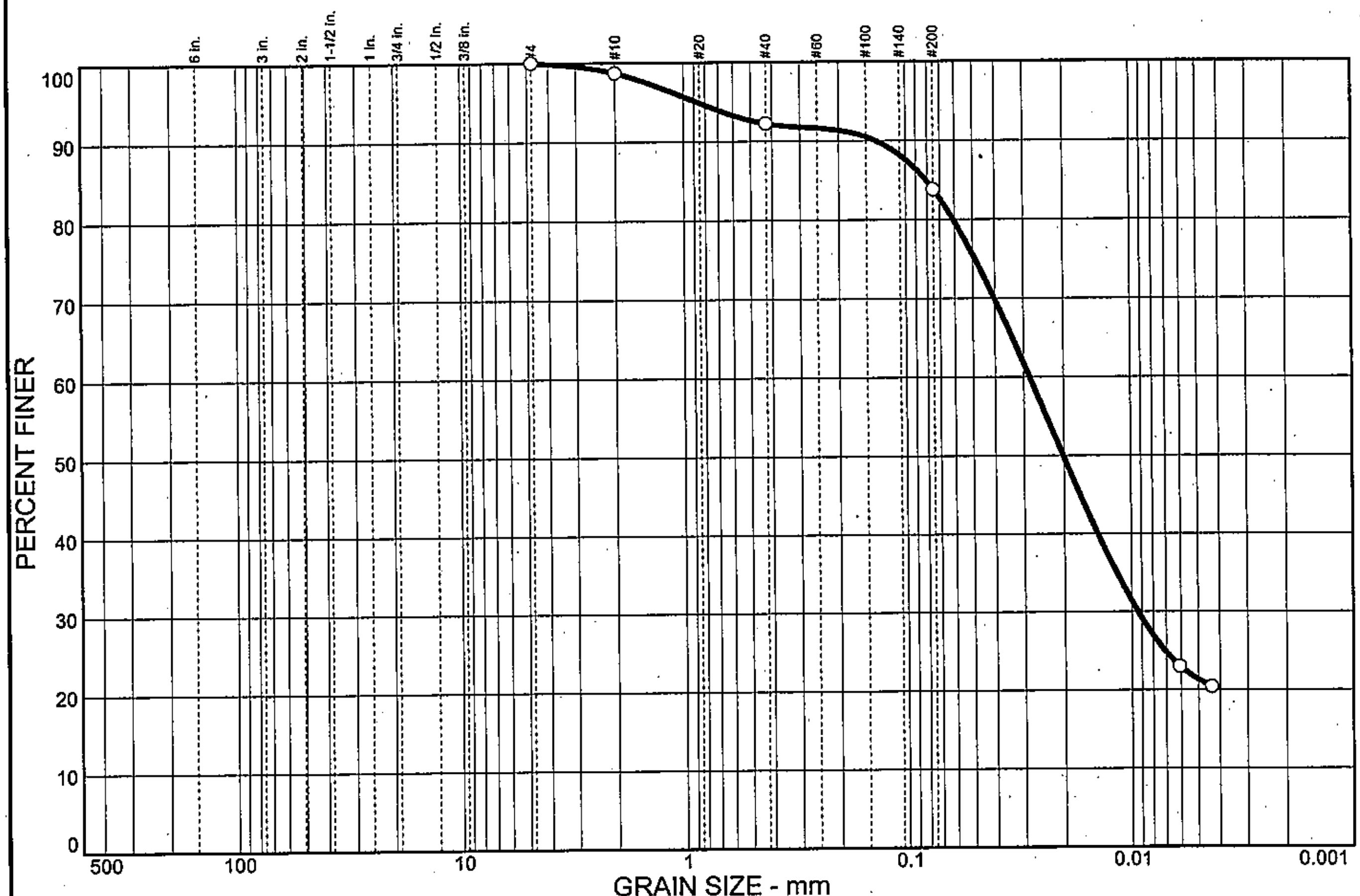
Moisture Content= 14.8%

Sample No.: 3 **Source of Sample:** B-48 **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	1.3	6.5	8.4	62.5	21.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.7		
#40	92.2		
#200	83.8		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 19 LL= 27 PI= 8

Coefficients

D_{85}= 0.0811 D_{60}= 0.0283 D_{50}= 0.0201
D_{30}= 0.0094 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

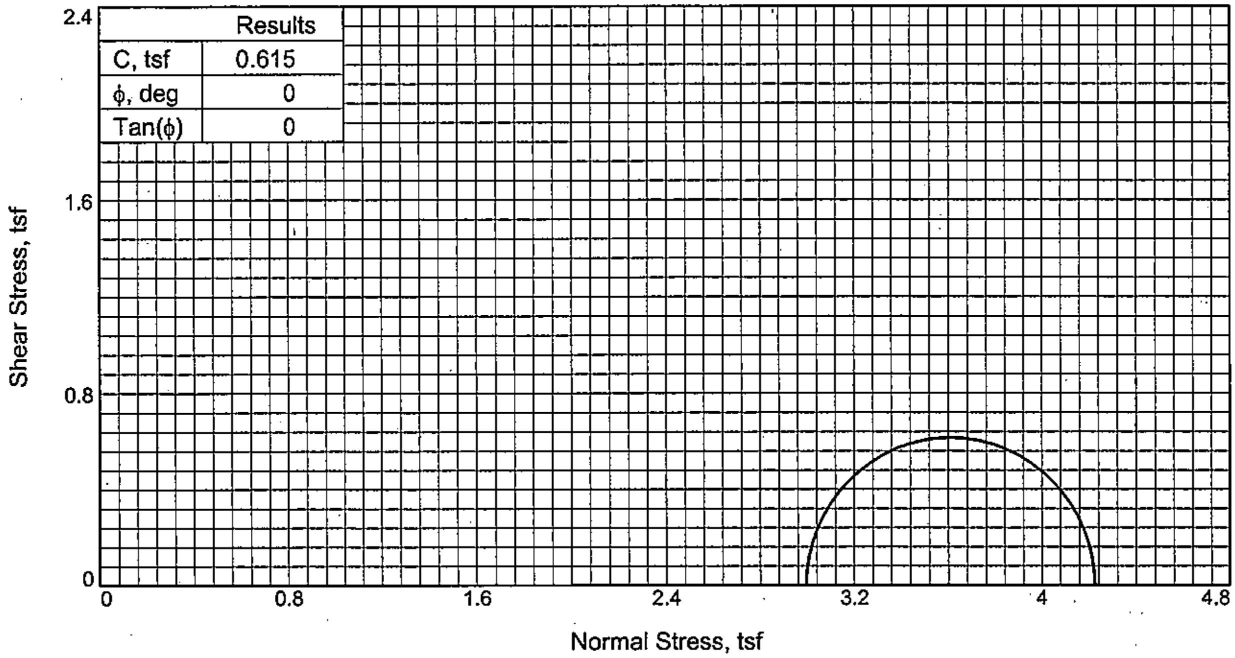
Remarks

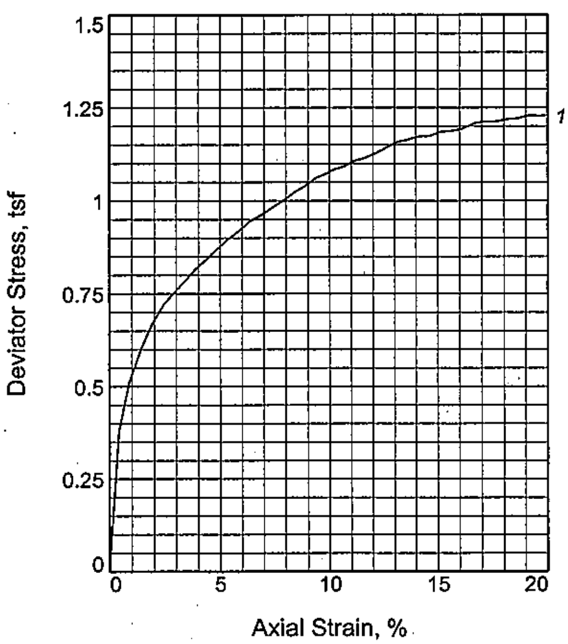
Moisture Content= 20.6%

Sample No.: ST-1 **Source of Sample:** B-48 **Date:** 6/8/07
Location: **Elev./Depth:** 6.0

DLZ

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





	Sample No.	1
Initial	Water Content,	21.4
	Dry Density, pcf	105.0
	Saturation,	95.3
	Void Ratio	0.6053
	Diameter, in.	2.85
	Height, in.	5.43
At Test	Water Content,	21.9
	Dry Density, pcf	105.0
	Saturation,	97.8
	Void Ratio	0.6053
	Diameter, in.	2.85
	Height, in.	5.43
	Strain rate, in./min.	0.06
	Back Pressure, tsf	0.00
	Cell Pressure, tsf	3.00
	Fail. Stress, tsf	1.23
	Ult. Stress, tsf	
	σ_1 Failure, tsf	4.22
	σ_3 Failure, tsf	3.00

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

LL= 27 **PL=** 19 **PI=** 8
Assumed Specific Gravity= 2.7
Remarks:

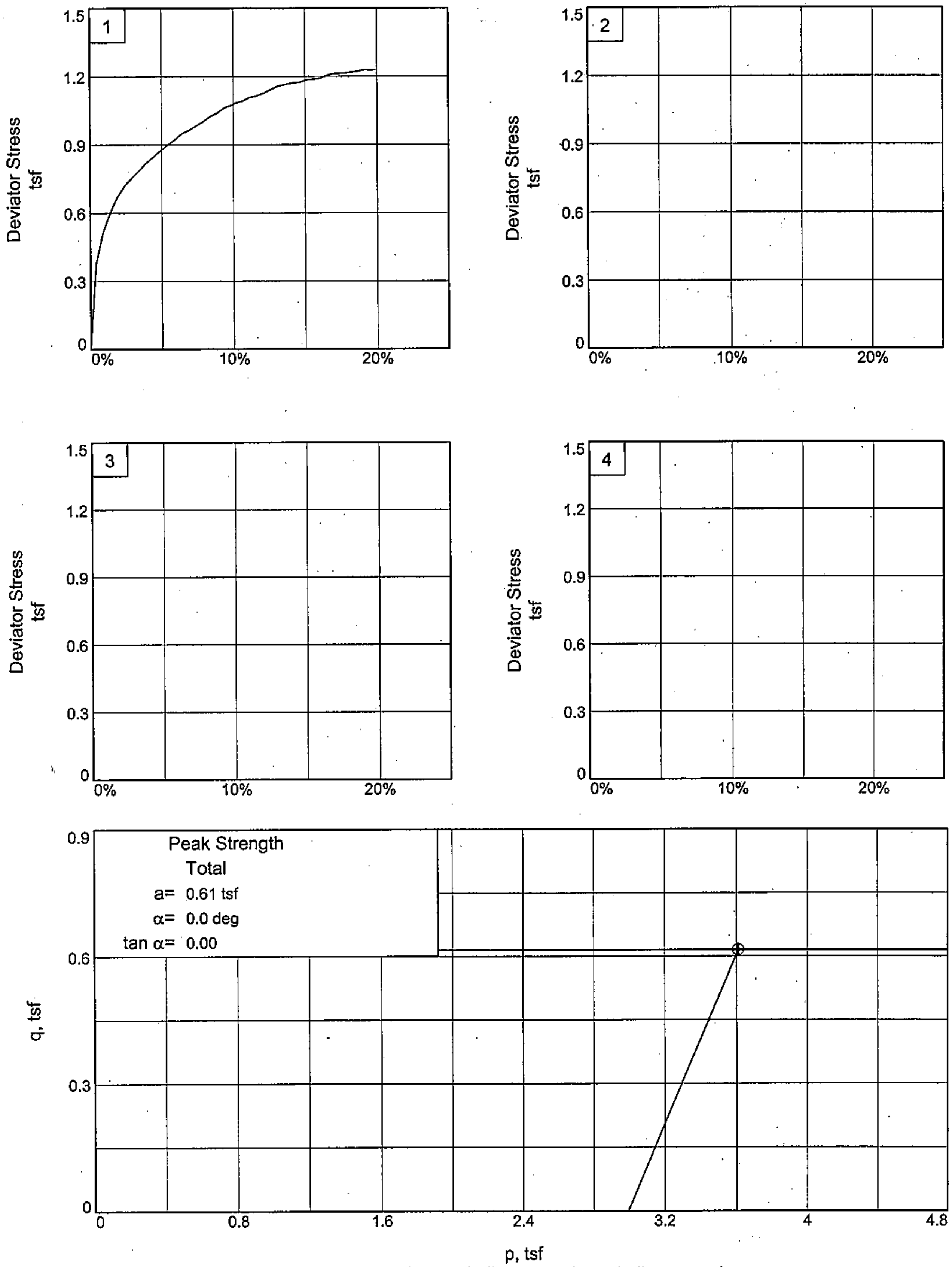
Figure _______

Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-48 **Depth:** 6.0
Sample Number: ST-1
Proj. No.: 0121-3070.03 **Date:** 6/8/07

DLZ

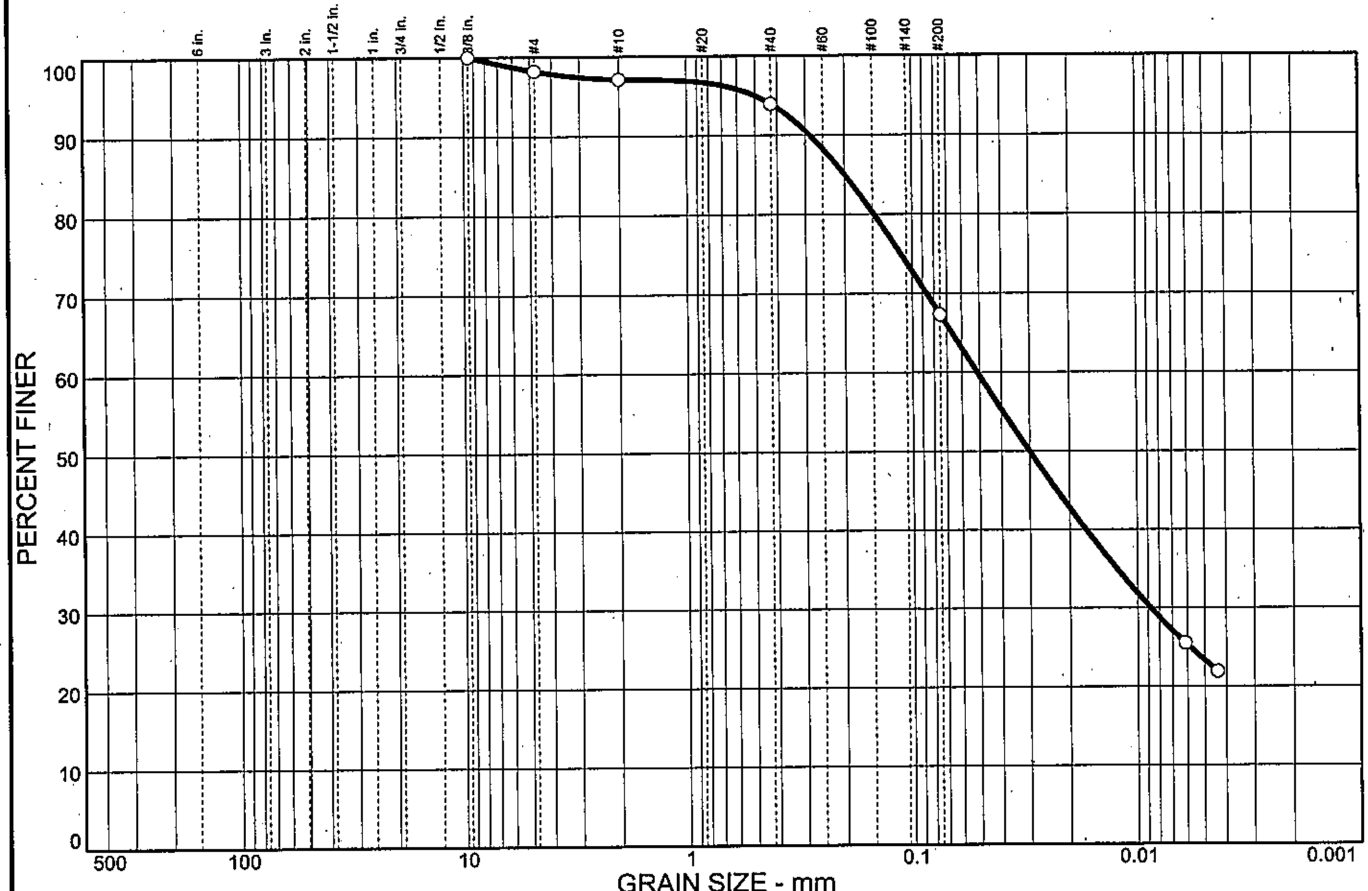


Stress Paths: o indicates peak + indicates end

Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-48 **Depth:** 6.0 **Sample Number:** ST-1
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.8	1.1	3.2	26.6	44.0	23.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.2		
#10	97.1		
#40	93.9		
#200	67.3		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 20 LL= 24 PI= 4

Coefficients

D_{85}= 0.201 D_{60}= 0.0513 D_{50}= 0.0301
D_{30}= 0.0088 D_{15}= D_{10}=
C_u= C_c=

Classification

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

Remarks

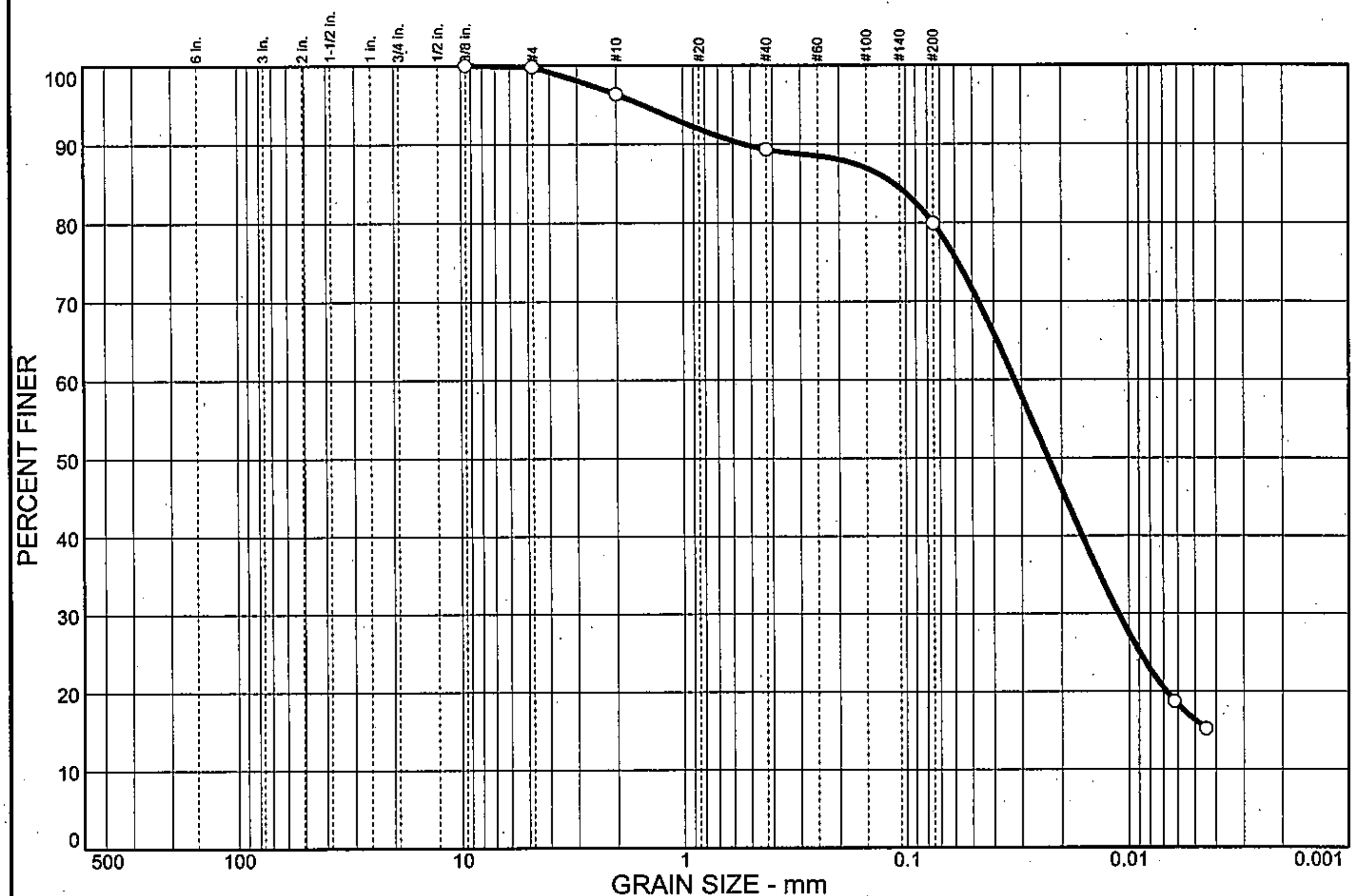
Moisture Content= 15.9%

Sample No.: 2 **Source of Sample:** B-49 **Date:** 6/11/07
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	0.2	3.5	7.0	9.4	63.6	16.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.8		
#10	96.3		
#40	89.3		
#200	79.9		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= 22 LL= 25 PI= 3

Coefficients

D_{85}= 0.113 D_{60}= 0.0323 D_{50}= 0.0229
D_{30}= 0.0111 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(1)

Remarks

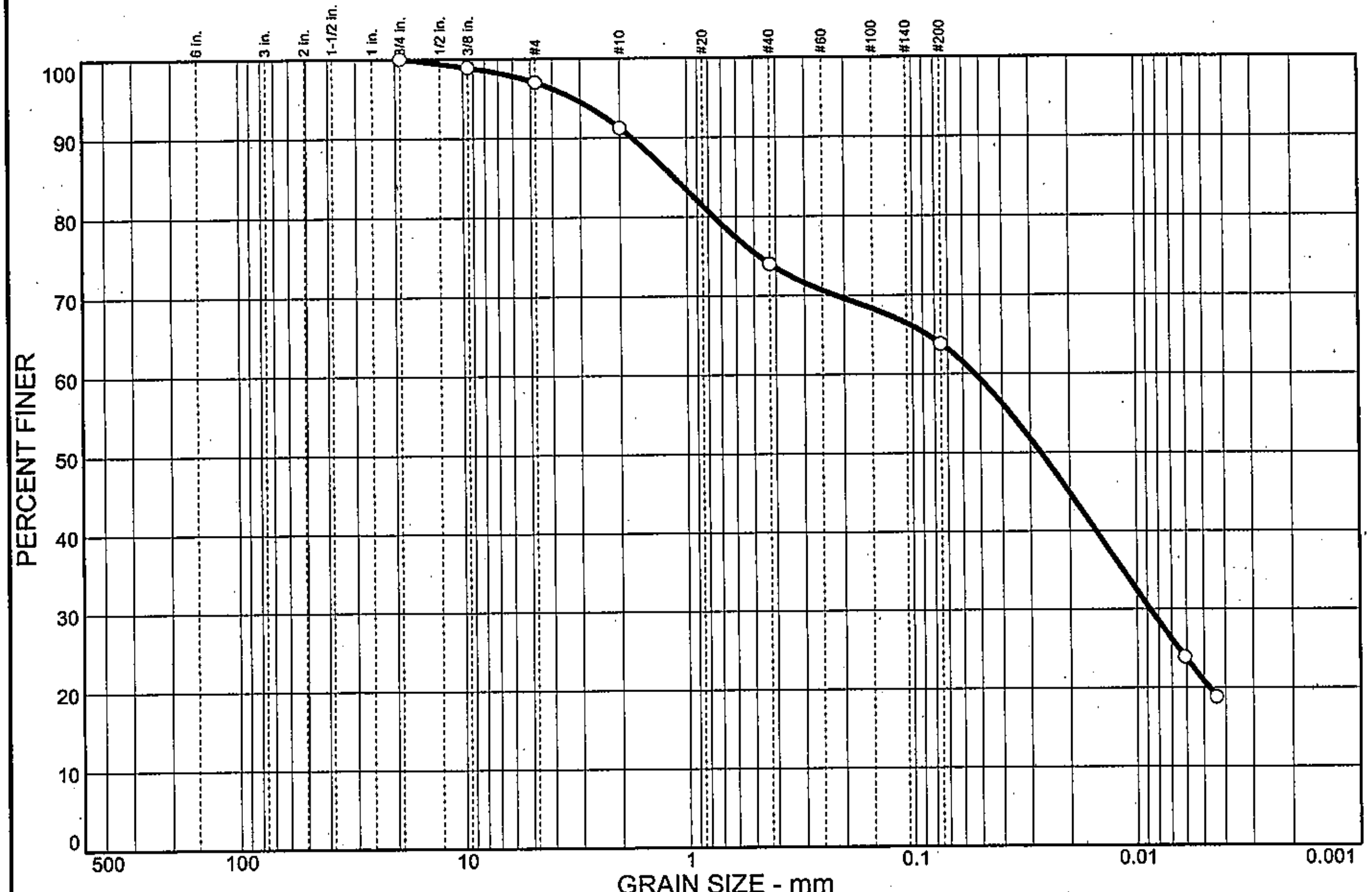
Moisture Content= 19.3%

Sample No.: ST-1 **Source of Sample:** B-49 **Date:** 6/11/07
Location: **Elev./Depth:** 6.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	3.0	5.8	17.2	10.2	43.0	20.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.9		
#4	97.0		
#10	91.2		
#40	74.0		
#200	63.8		

* (no specification provided)

Soil Description

Sandy lean clay

Atterberg Limits

PL= 18 LL= 27 PI= 9

Coefficients

D_{85}= 1.15 D_{60}= 0.0530 D_{50}= 0.0270
D_{30}= 0.0087 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(3)

Remarks

Moisture Content= 14.7%

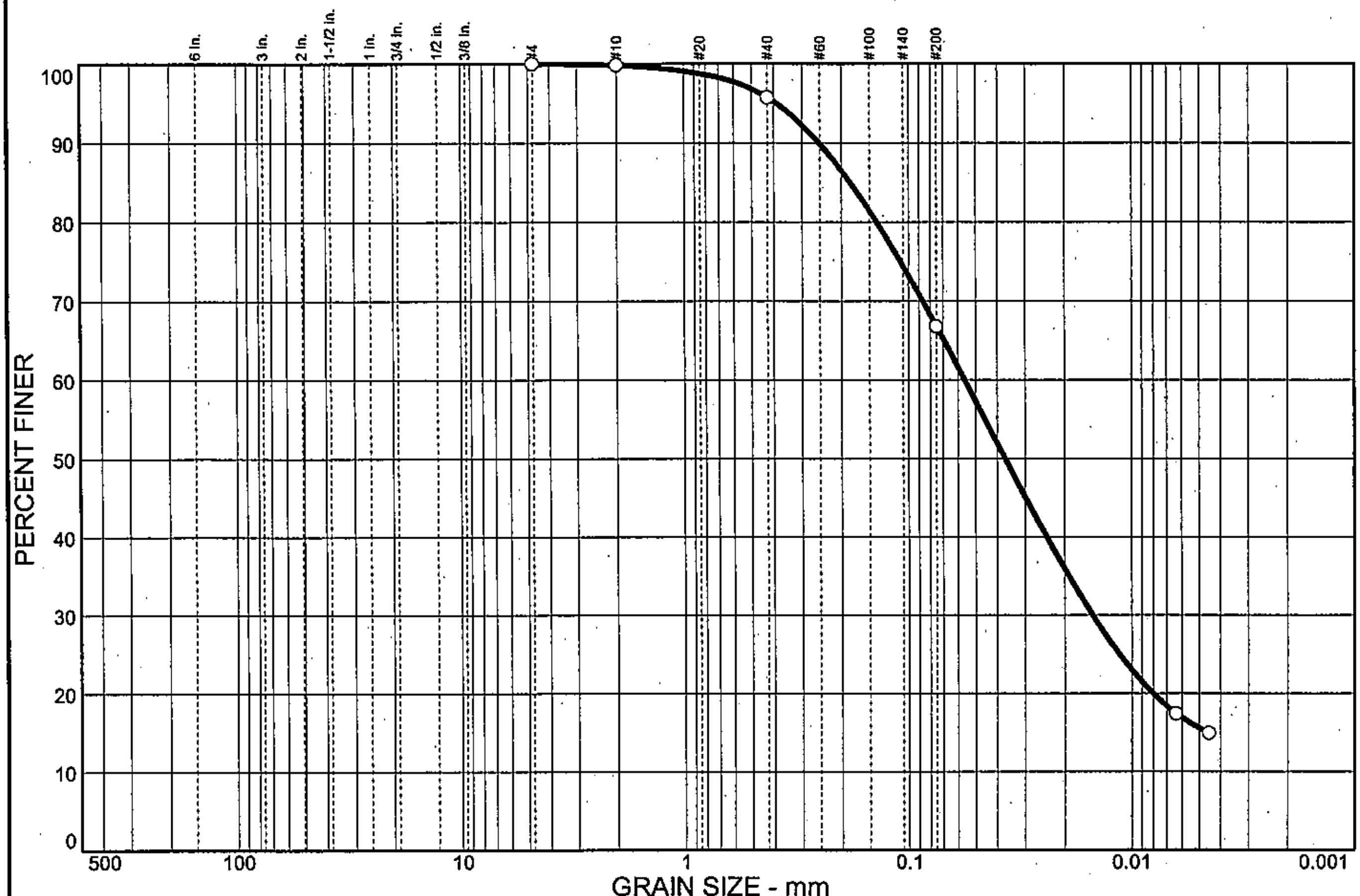
Sample No.: 2 **Source of Sample:** B-50 **Date:** 6/11/07
Location: **Elev./Depth:** 3.5

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	0.2	4.1	28.9	51.2	15.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#40	95.7		
#200	66.8		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 16 LL= 21 PI= 5

Coefficients

D_{85}= 0.184 D_{60}= 0.0561 D_{50}= 0.0369
D_{30}= 0.0150 D_{15}= 0.0046 D_{10}=
C_u= C_c=

Classification

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

Remarks

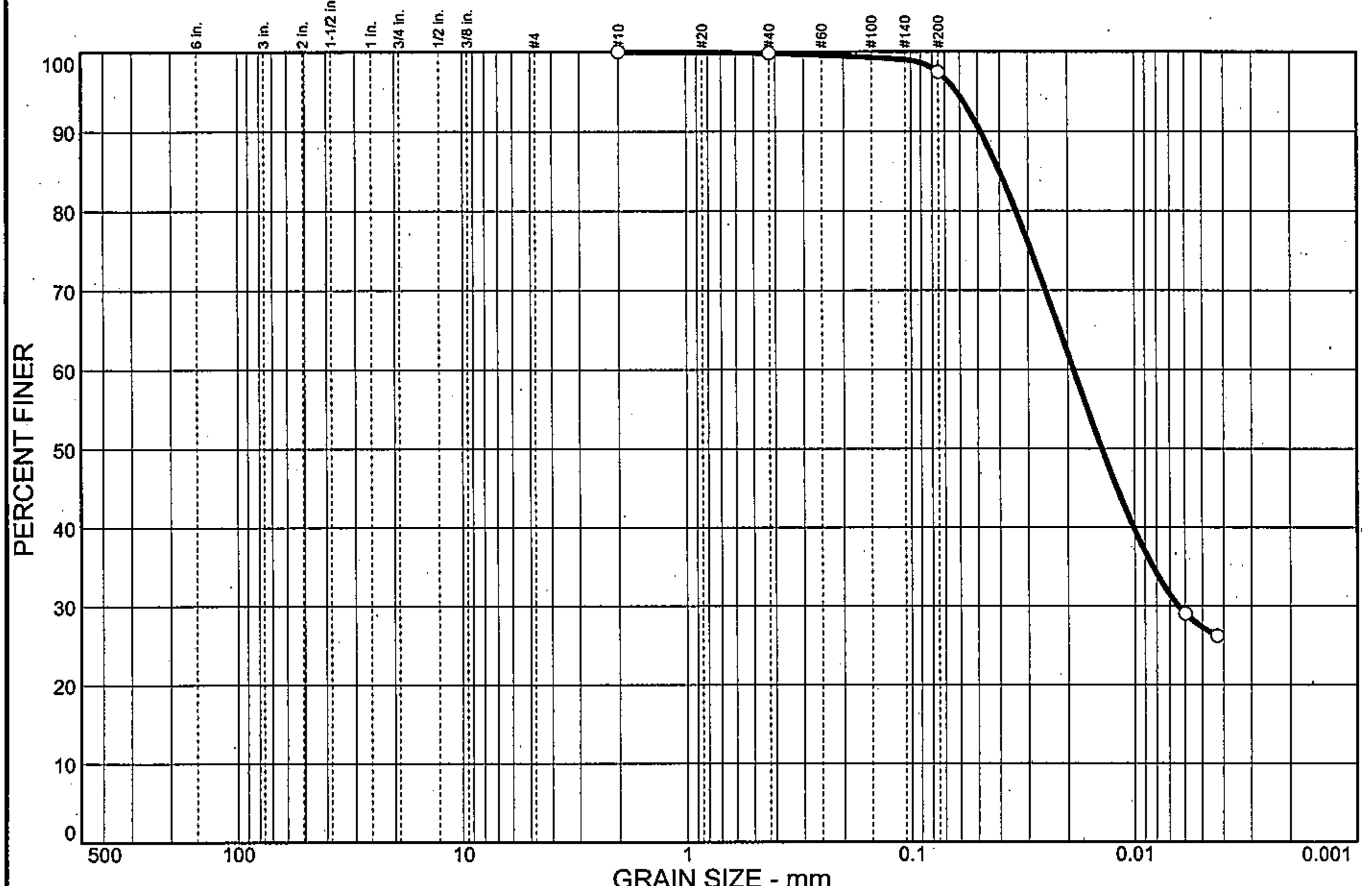
Moisture Content= 19.1%

Sample No.: ST-1 **Source of Sample:** B-50 **Date:** 6/11/07
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	0.0	0.0	0.2	2.4	70.1	27.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.8		
#200	97.4		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 32 PI= 13

Coefficients

D_{85}= 0.0402 D_{60}= 0.0188 D_{50}= 0.0140
D_{30}= 0.0064 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(12)

Remarks

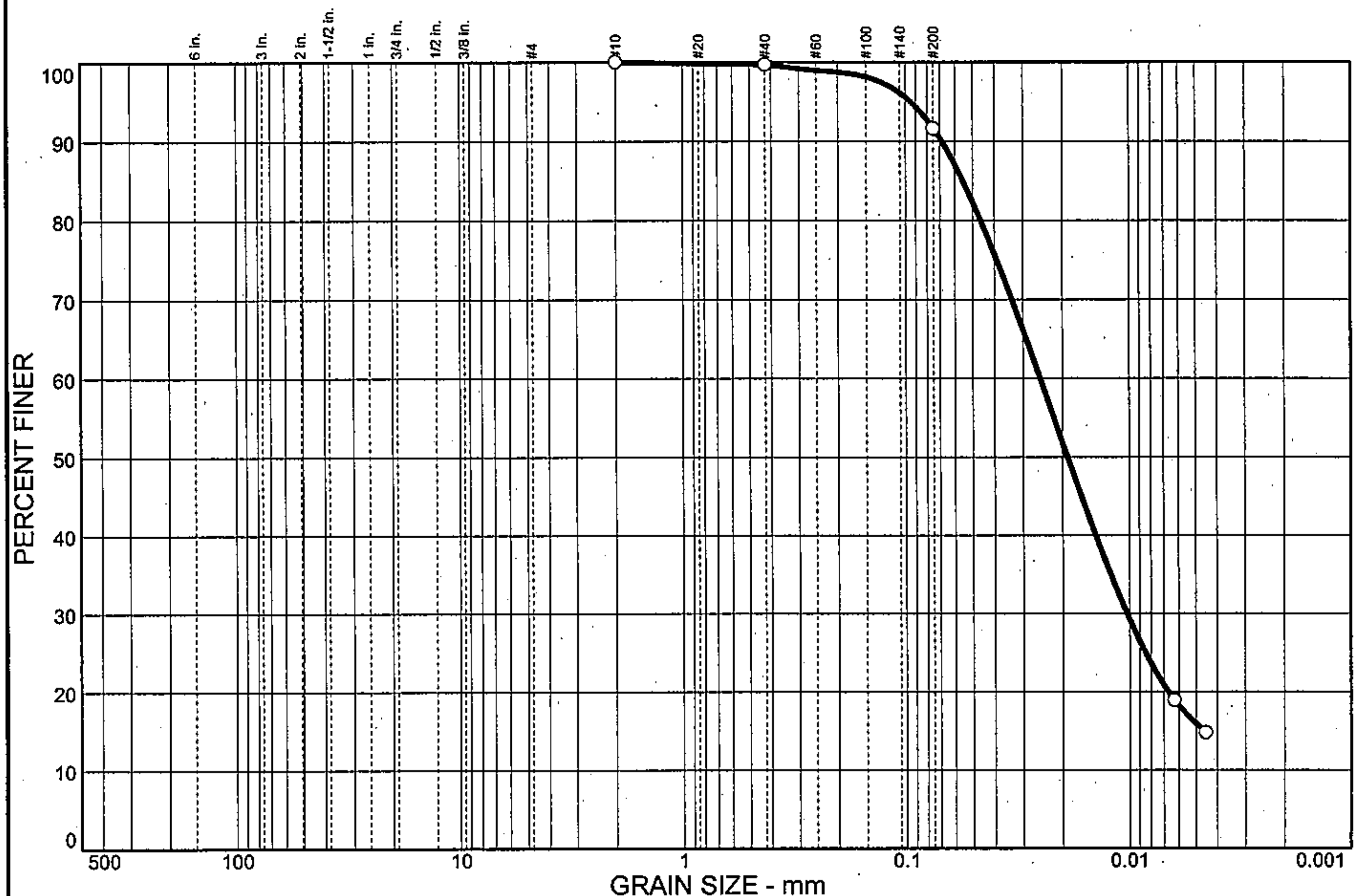
Moisture Content= 24.3%

Sample No.: 2 **Source of Sample:** B-51 **Date:** 6/8/07
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	8.1	75.7	15.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	91.6		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 21 LL= 26 PI= 5

Coefficients

D_{85}= 0.0554 D_{60}= 0.0252 D_{50}= 0.0190
D_{30}= 0.0103 D_{15}= 0.0046 D_{10}=
C_u= C_c=

Classification

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

Remarks

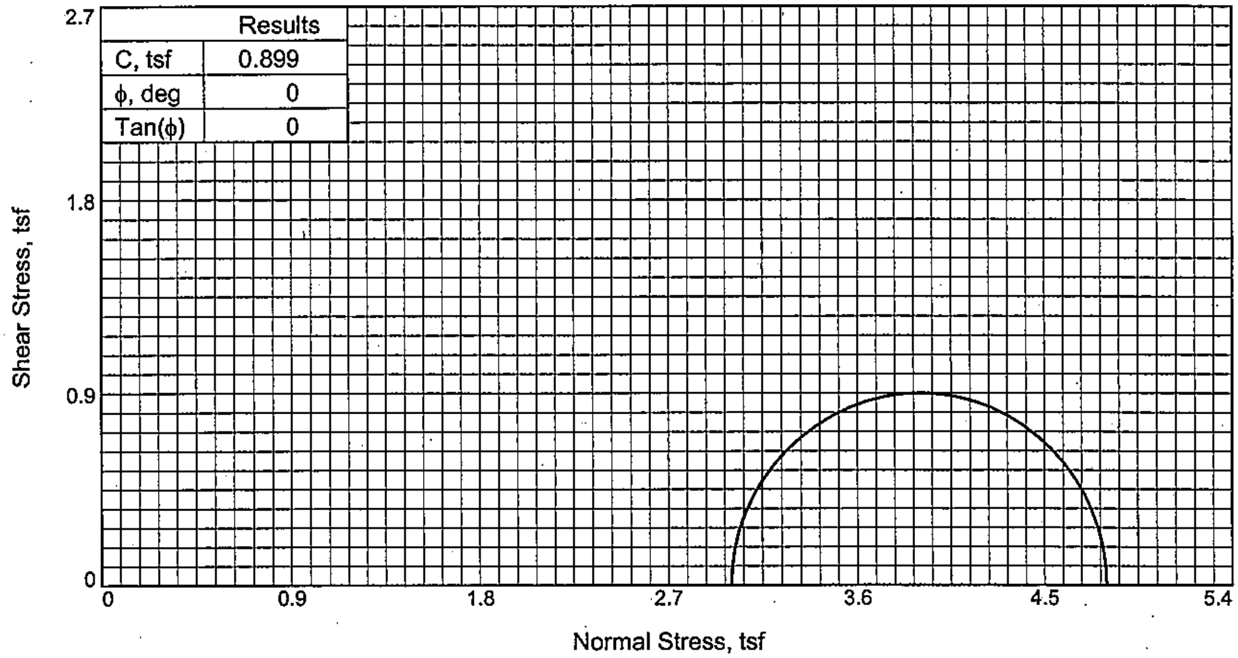
Moisture Content= 25.8%

Sample No.: ST-1 **Source of Sample:** B-51 **Date:** 6/12/07
Location: **Elev./Depth:** 6.0

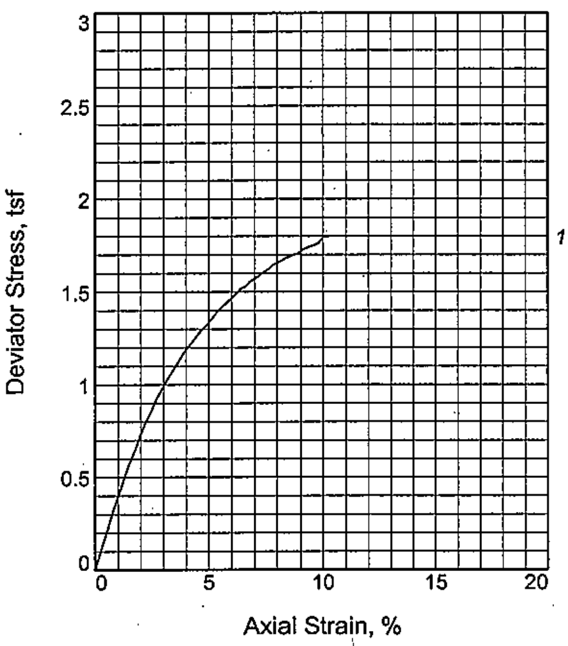
DLZ

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

Project No: 0121-3070.03 **Figure**



Results	
C, tsf	0.899
ϕ, deg	0
Tan(ϕ)	0



Sample No.		1
Initial	Water Content,	23.5
	Dry Density, pcf	102.2
	Saturation,	97.8
	Void Ratio	0.6485
	Diameter, in.	2.86
	Height, in.	5.53
At Test	Water Content,	25.3
	Dry Density, pcf	102.2
	Saturation,	105.2
	Void Ratio	0.6485
	Diameter, in.	2.86
	Height, in.	5.53
Strain rate, in./min.		0.06
Back Pressure, tsf		0.00
Cell Pressure, tsf		3.00
Fail. Stress, tsf		1.80
Ult. Stress, tsf		
σ_1 Failure, tsf		4.80
σ_3 Failure, tsf		3.00

Type of Test:
Unconsolidated Undrained
Sample Type: 3" press tube
Description:

LL= 26 **PL=** 21 **PI=** 5
Assumed Specific Gravity= 2.7
Remarks:

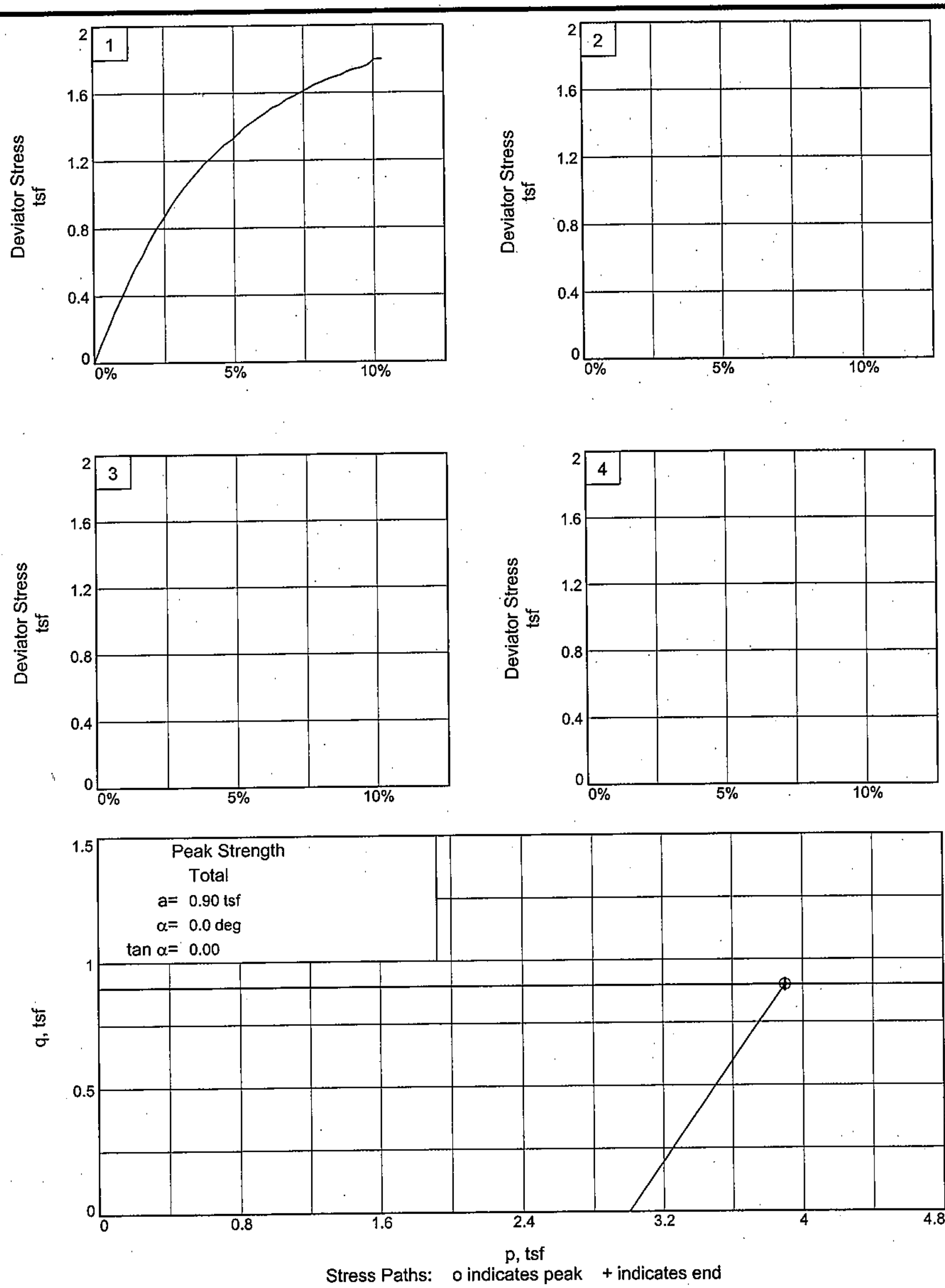
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-51 **Depth:** 6.0
Sample Number: ST-1
Proj. No.: 0121-3070.03 **Date:** 6/12/07

DLZ

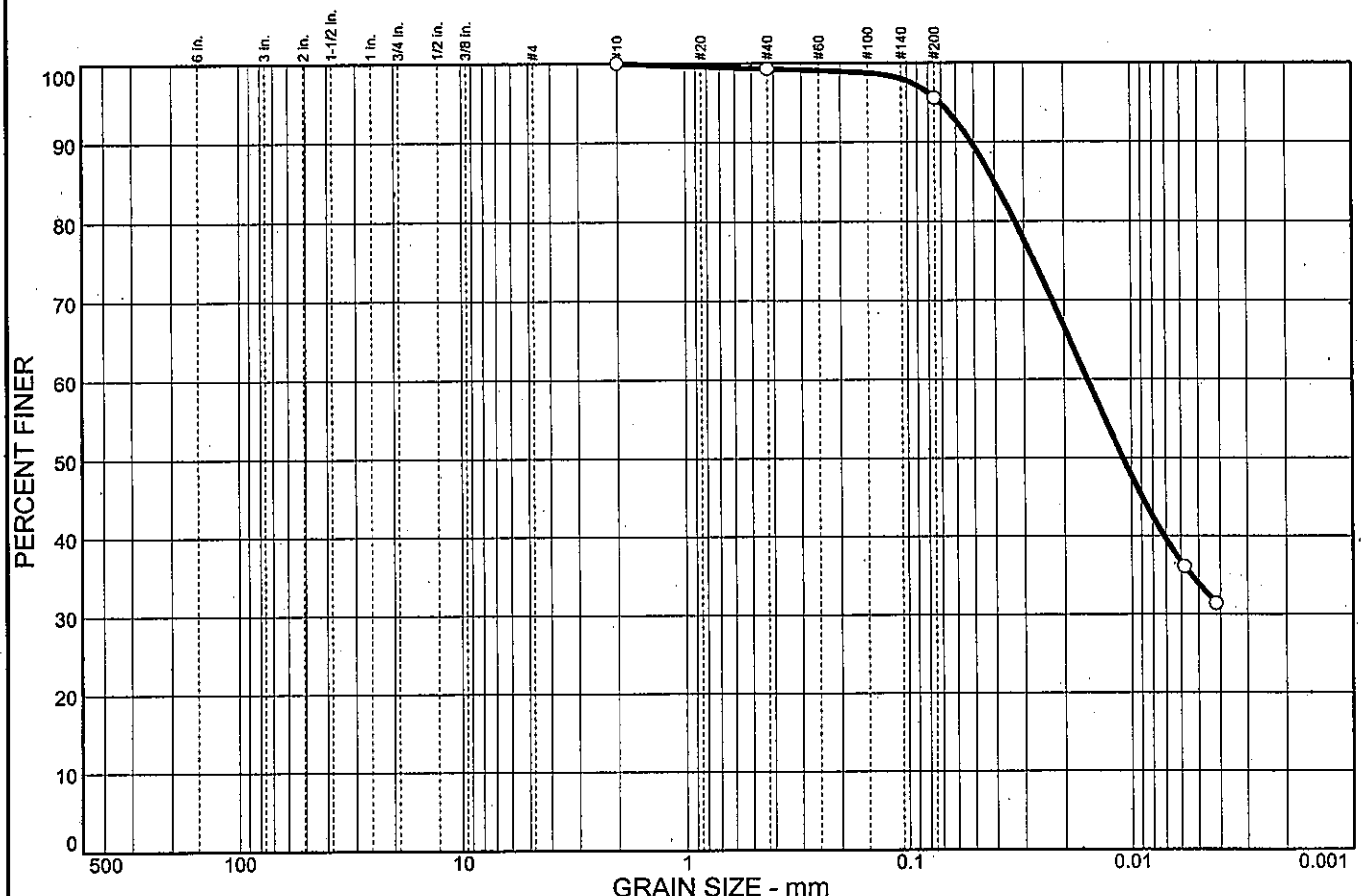
Figure ______



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-51 **Depth:** 6.0 **Sample Number:** ST-1
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

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	3.7	61.7	33.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.3		
#200	95.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 31 PI= 12

Coefficients

D_{85}= 0.0404 D_{60}= 0.0158 D_{50}= 0.0109
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

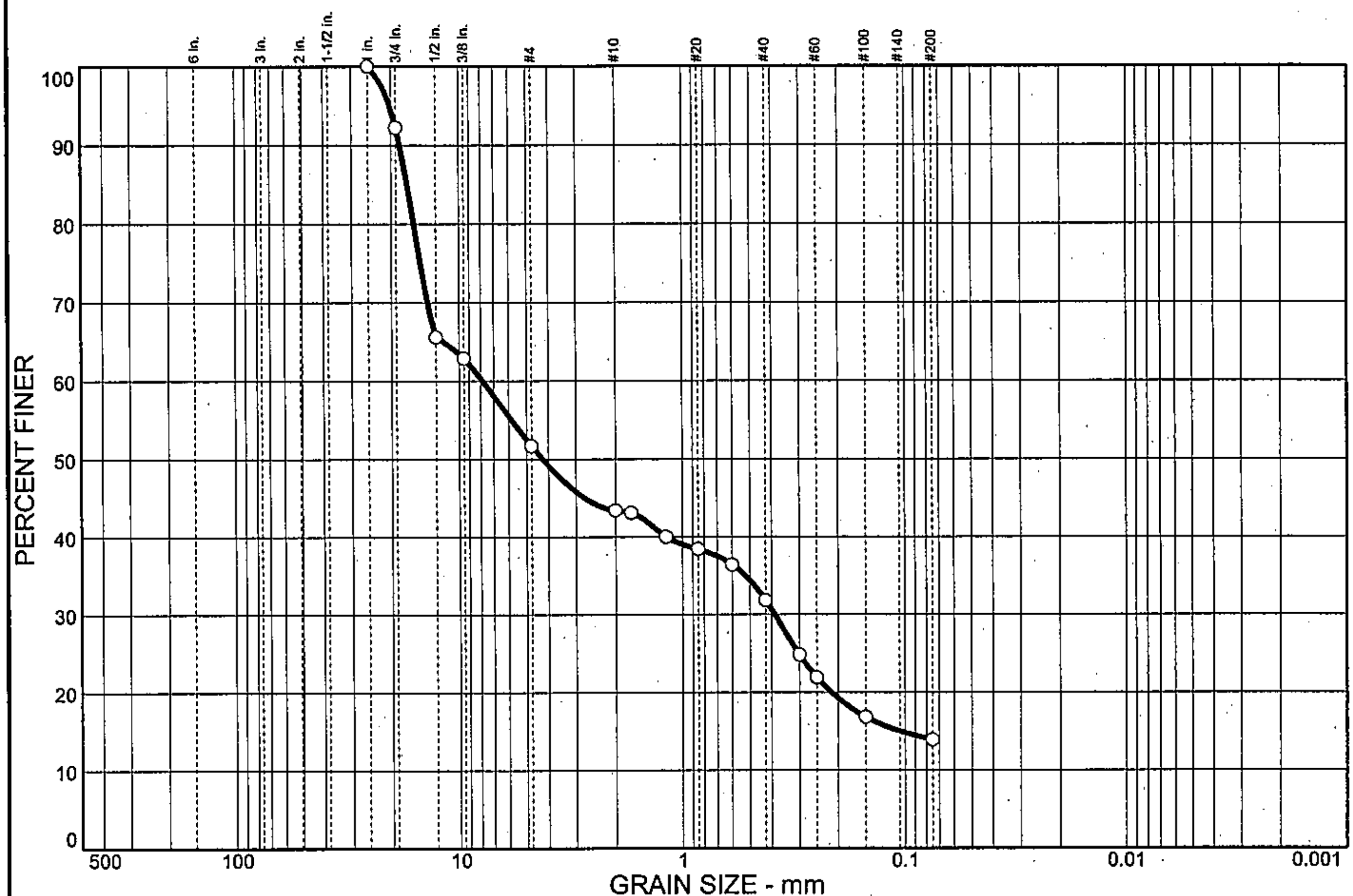
Moisture Content= 21.8%

Sample No.: 2 **Source of Sample:** B-52 **Date:** 6/8/07
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	7.8	40.5	8.3	11.6	17.9	13.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00 in.	100.0		
0.75 in.	92.2		
0.50 in.	65.6		
0.375 in.	62.9		
#4	51.7		
#10	43.4		
#12	43.1		
#16	40.0		
#20	38.4		
#30	36.4		
#40	31.8		
#50	24.8		
#60	21.9		
#100	16.8		
#200	13.9		

* (no specification provided)

Soil Description

Silty gravel with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 17.0 D_{60}= 7.86 D_{50}= 4.24
D_{30}= 0.387 D_{15}= 0.105 D_{10}=
C_u= C_c=

Classification

USCS= GM AASHTO= A-1-b

Remarks

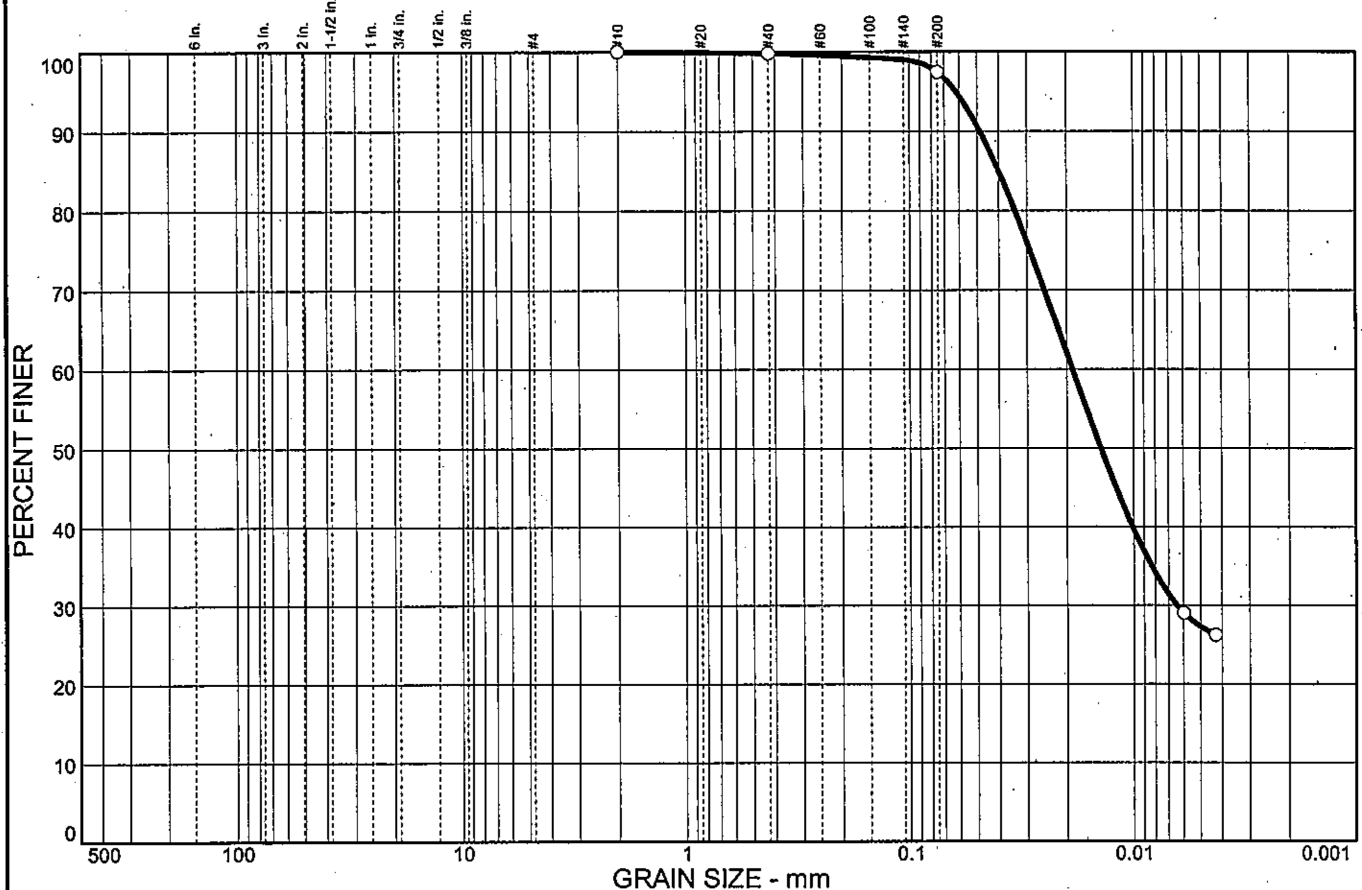
Moisture Content= 10.4%

Sample No.: 4 **Source of Sample:** B-52 **Date:** 6/8/07
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	0.2	2.4	70.1	27.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.8		
#200	97.4		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 32 PI= 13

Coefficients

D_{85}= 0.0402 D_{60}= 0.0188 D_{50}= 0.0140
D_{30}= 0.0064 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(12)

Remarks

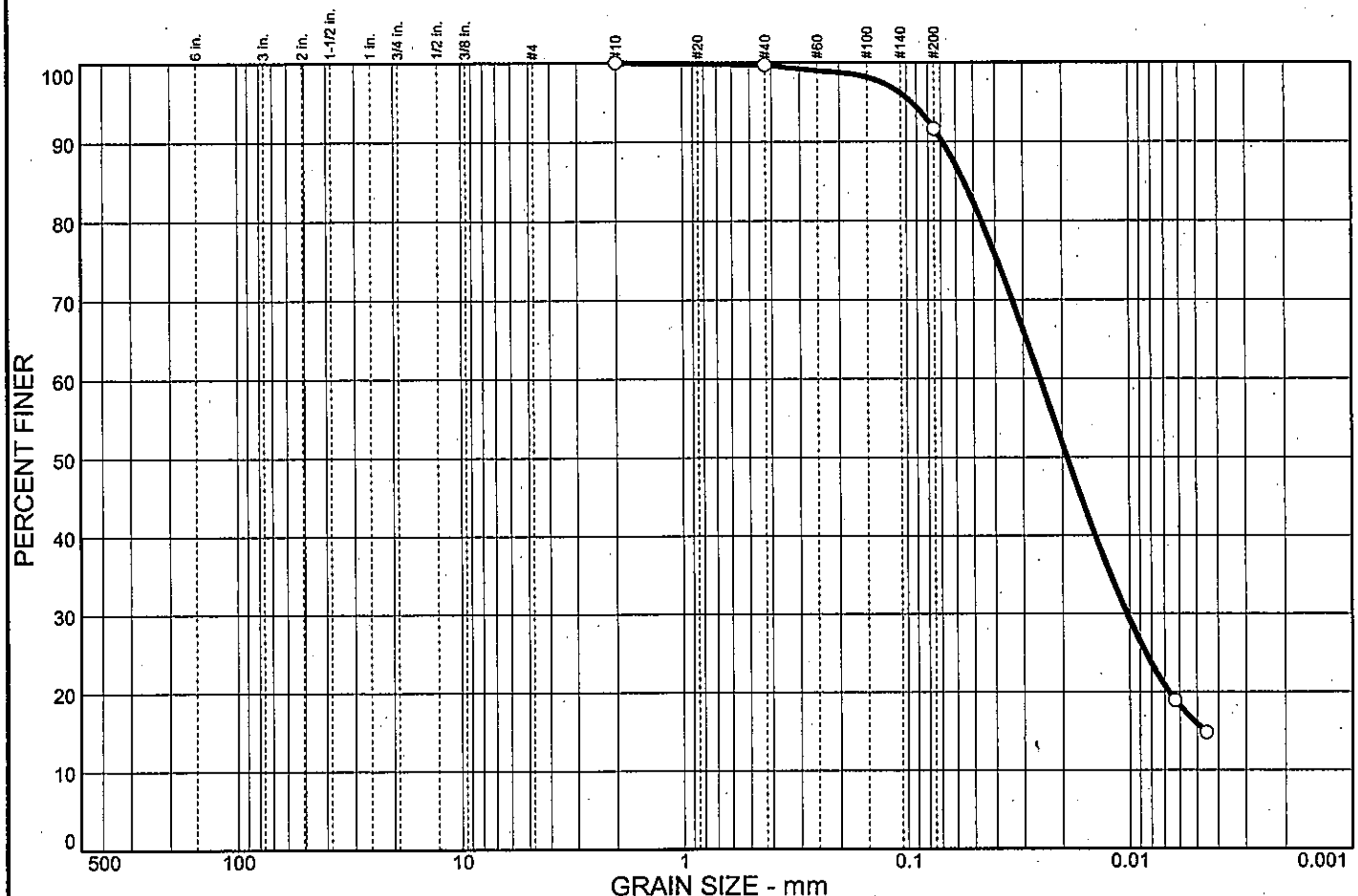
Moisture Content= 24.3%

Sample No.: 2 **Source of Sample:** B-51 **Date:** 6/8/07
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	8.1	75.7	15.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.7		
#200	91.6		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 21 LL= 26 PI= 5

Coefficients

D_{85}= 0.0554 D_{60}= 0.0252 D_{50}= 0.0190
D_{30}= 0.0103 D_{15}= 0.0046 D_{10}=
C_u= C_c=

Classification

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

Remarks

Moisture Content= 25.8%

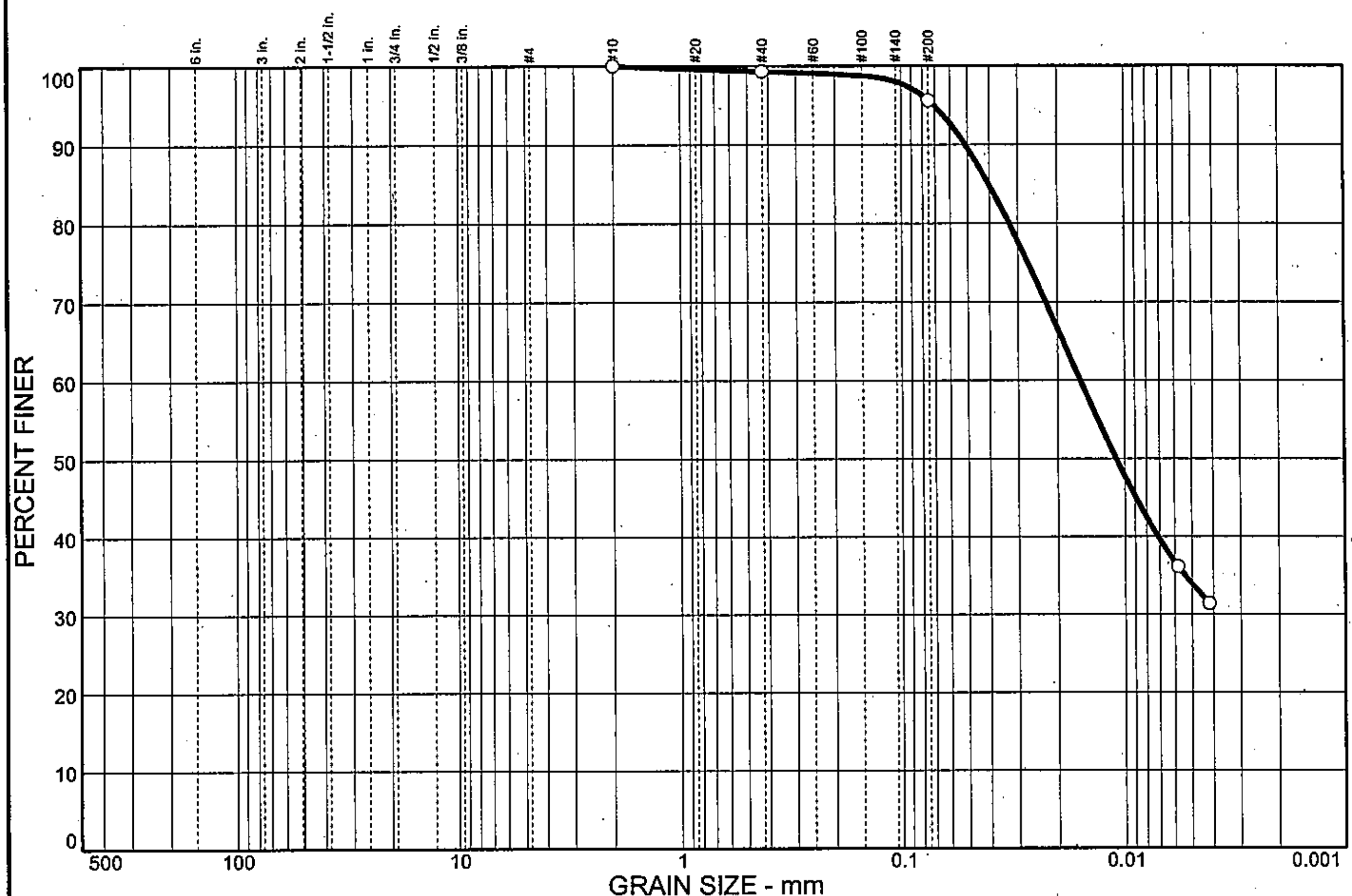
Sample No.: ST-1 **Source of Sample:** B-51 **Date:** 6/12/07
Location: **Elev./Depth:** 6.0

DLZ

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

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.7	3.7	61.7	33.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.3		
#200	95.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 31 PI= 12

Coefficients

D_{85}= 0.0404 D_{60}= 0.0158 D_{50}= 0.0109
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

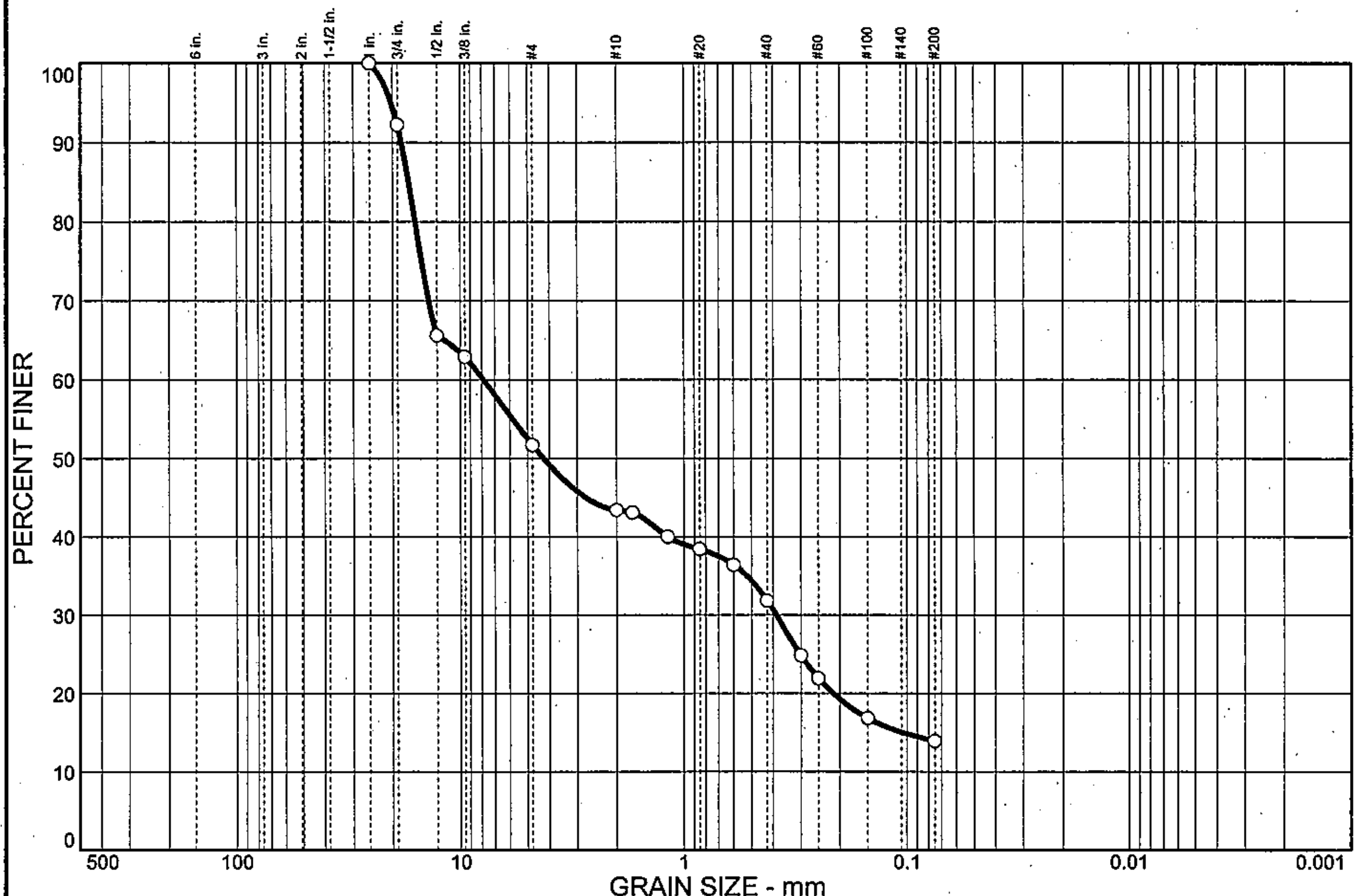
Moisture Content= 21.8%

Sample No.: 2 **Source of Sample:** B-52 **Date:** 6/8/07
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	7.8	40.5	8.3	11.6	17.9	13.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00 in.	100.0		
0.75 in.	92.2		
0.50 in.	65.6		
0.375 in.	62.9		
#4	51.7		
#10	43.4		
#12	43.1		
#16	40.0		
#20	38.4		
#30	36.4		
#40	31.8		
#50	24.8		
#60	21.9		
#100	16.8		
#200	13.9		

* (no specification provided)

Soil Description

Silty gravel with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 17.0 D_{60}= 7.86 D_{50}= 4.24
D_{30}= 0.387 D_{15}= 0.105 D_{10}=
C_u= C_c=

Classification

USCS= GM AASHTO= A-1-b

Remarks

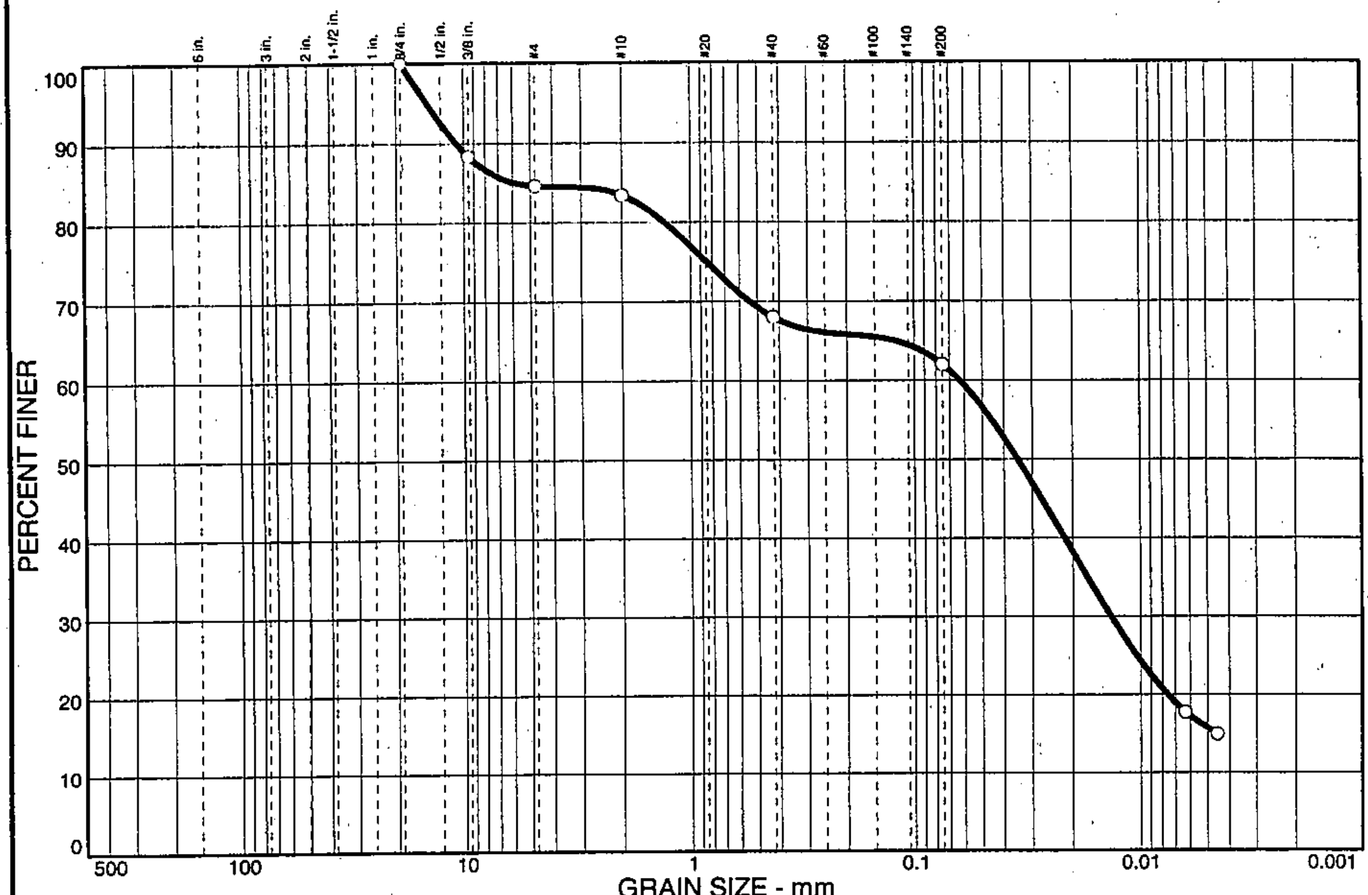
Moisture Content= 10.4%

Sample No.: 4 **Source of Sample:** B-52 **Date:** 6/8/07
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	15.4	1.2	15.4	6.0	46.3	15.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	88.3		
#4	84.6		
#10	83.4		
#40	68.0		
#200	62.0		

* (no specification provided)

Soil Description

Sandy lean clay with gravel

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 5.86 D_{60}= 0.0631 D_{50}= 0.0350
D_{30}= 0.0136 D_{15}= 0.0046 D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(3)

Remarks

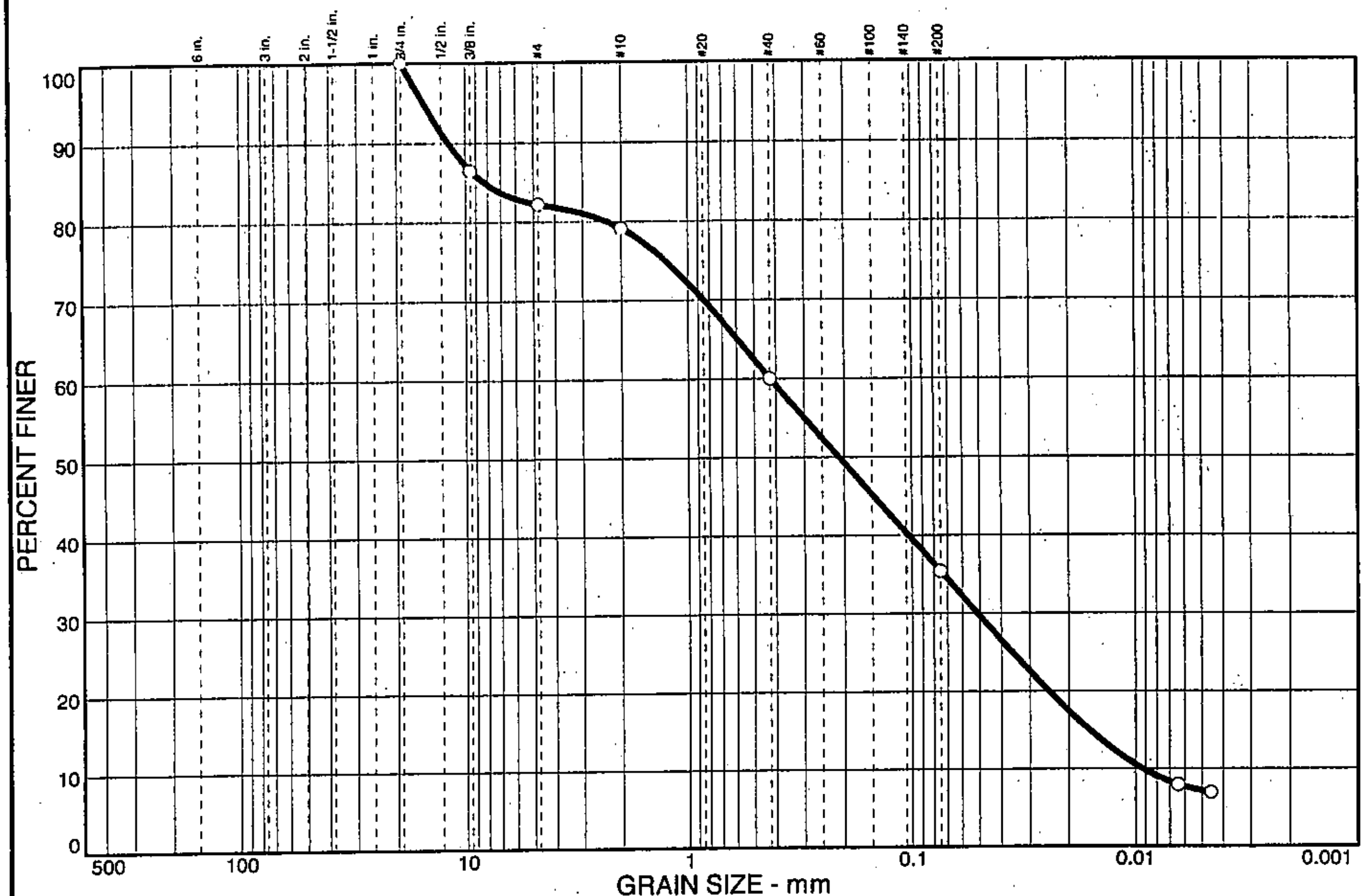
Moisture Content= 10.2%

Sample No.: 1 **Source of Sample:** TR-62 **Date:** 4/12/05
Location: **Elev./Depth:** 1

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	17.8	3.1	19.1	24.6	28.3	7.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	86.5		
#4	82.2		
#10	79.1		
#40	60.0		
#200	35.4		

* (no specification provided)

Soil Description

Silty sand with gravel

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 8.33 D_{60}= 0.425 D_{50}= 0.212
D_{30}= 0.0515 D_{15}= 0.0162 D_{10}= 0.0093
C_u= 45.60 C_c= 0.67

Classification

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

Remarks

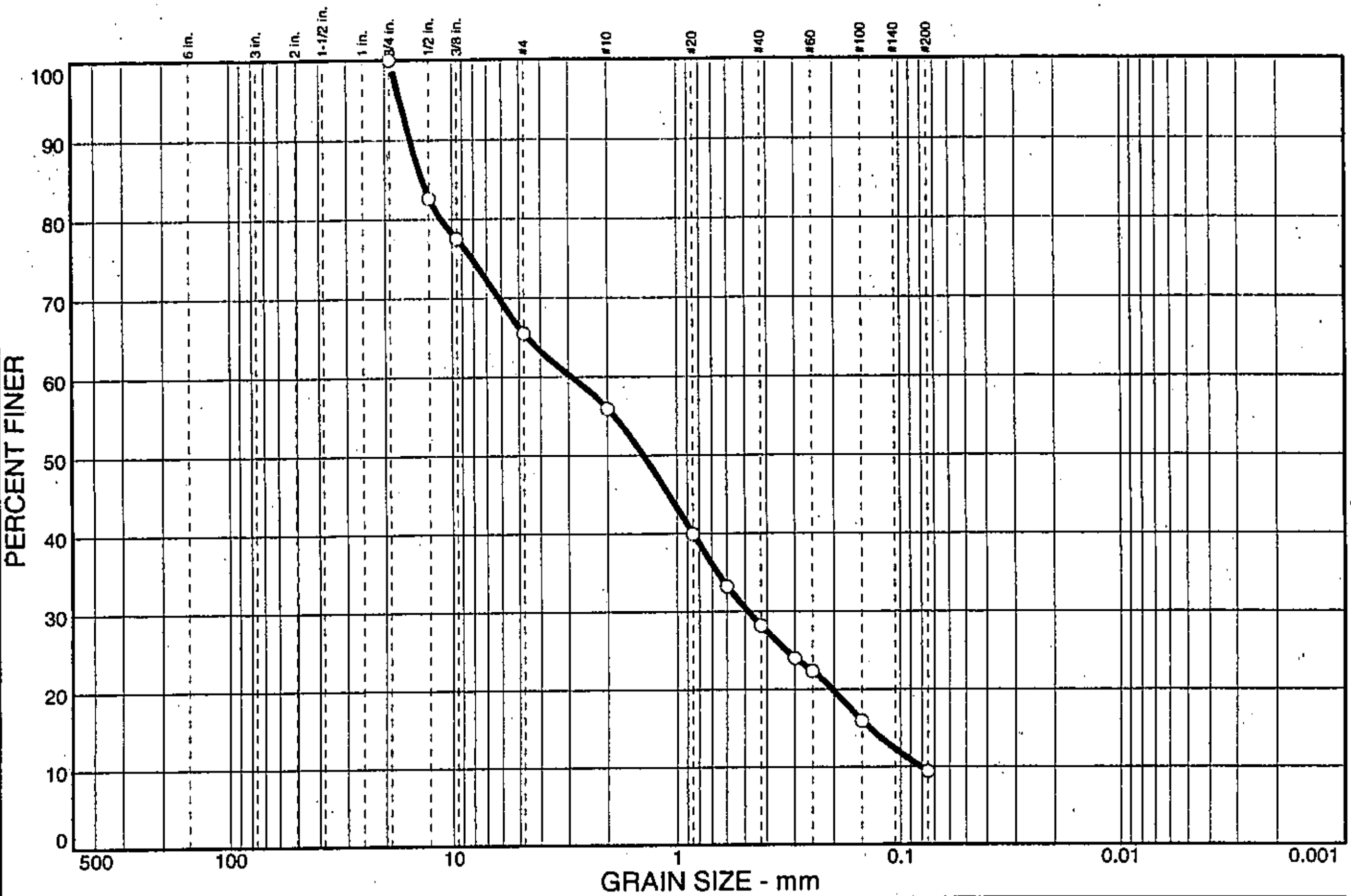
Moisture Content= 11.8%

Sample No.: 3 **Source of Sample:** TR-64 **Date:** 5/28/05
Location: **Elev./Depth:** 6.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	34.5	9.6	27.7	18.8	9.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.50 in.	82.6		
0.375 in.	77.5		
#4	65.5		
#10	55.9		
#20	39.9		
#30	33.2		
#40	28.2		
#50	24.0		
#60	22.3		
#100	15.9		
#200	9.4		

* (no specification provided)

Soil Description

Well-graded sand with silt and gravel

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 13.7 D_{60}= 2.90 D_{50}= 1.40
D_{30}= 0.486 D_{15}= 0.139 D_{10}= 0.0809
C_u= 35.92 C_c= 1.00

Classification

USCS= SW-SM AASHTO= A-1-b

Remarks

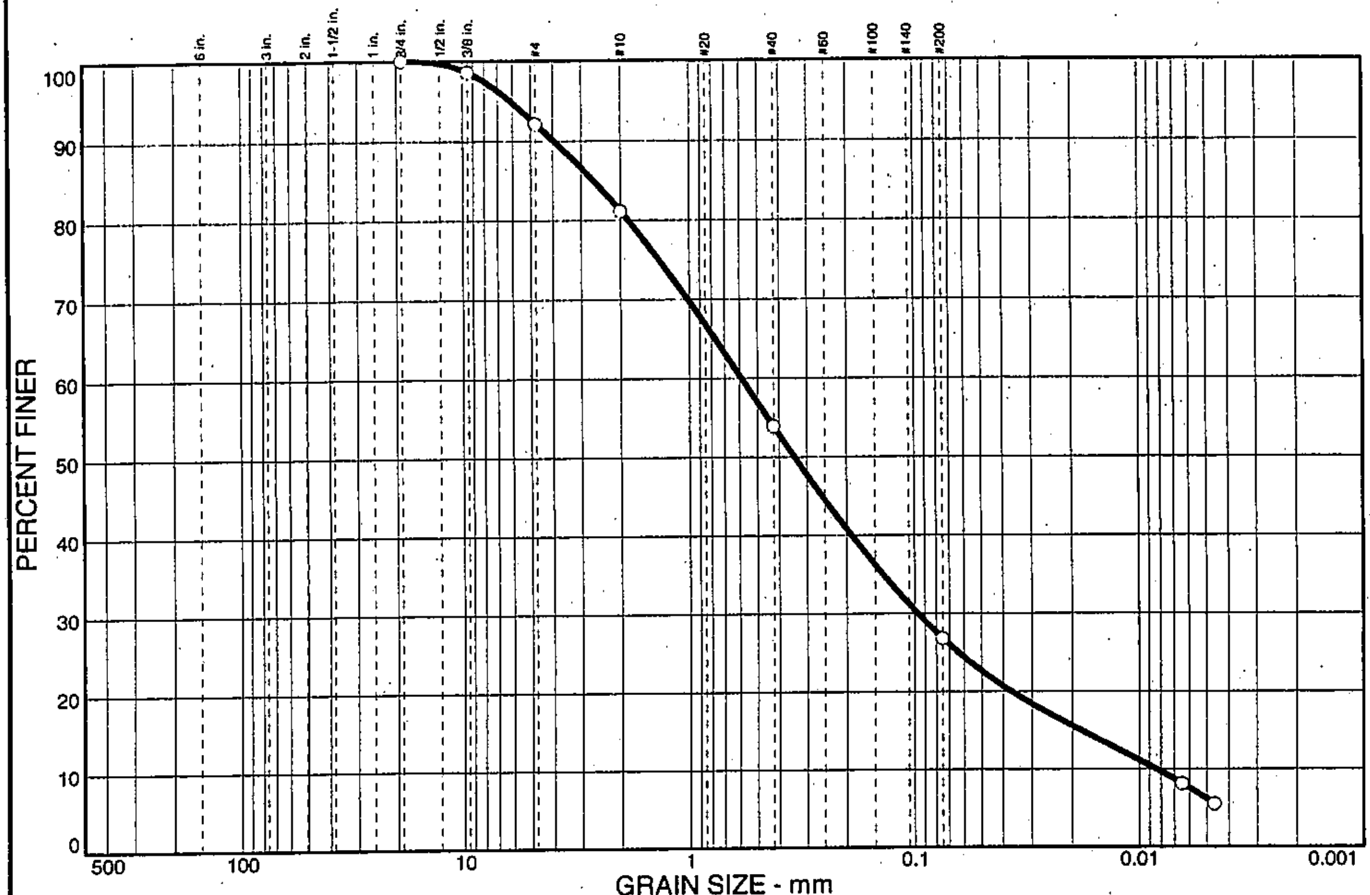
Moisture Content= 9.6%

Sample No.: 1A **Source of Sample:** TR-66 **Date:** 5/28/05
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	7.9	11.0	27.2	27.1	20.7	6.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.6		
#4	92.1		
#10	81.1		
#40	53.9		
#200	26.8		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 2.65 D_{60}= 0.591 D_{50}= 0.343
D_{30}= 0.0972 D_{15}= 0.0184 D_{10}= 0.0085
C_u= 69.78 C_c= 1.89

Classification

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

Remarks

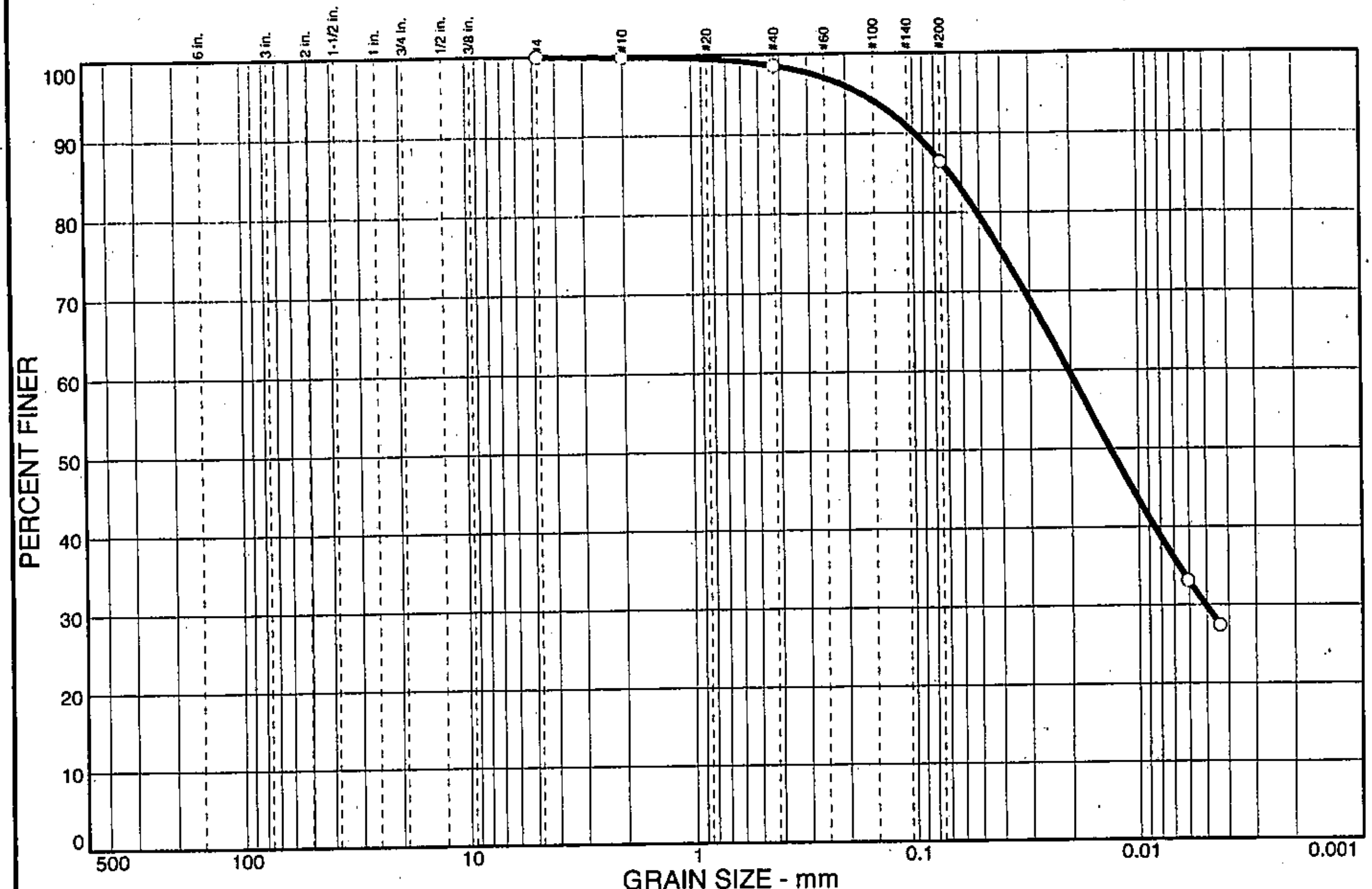
Moisture Content= 9.5%

Sample No.: 1B **Source of Sample:** TR-66 **Date:** 5/28/05
Location: **Elev./Depth:** 2.3

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	1.2	12.2	56.7	29.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#40	98.6		
#200	86.4		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{85}= 0.0685 D_{60}= 0.0205 D_{50}= 0.0134
D_{30}= 0.0051 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

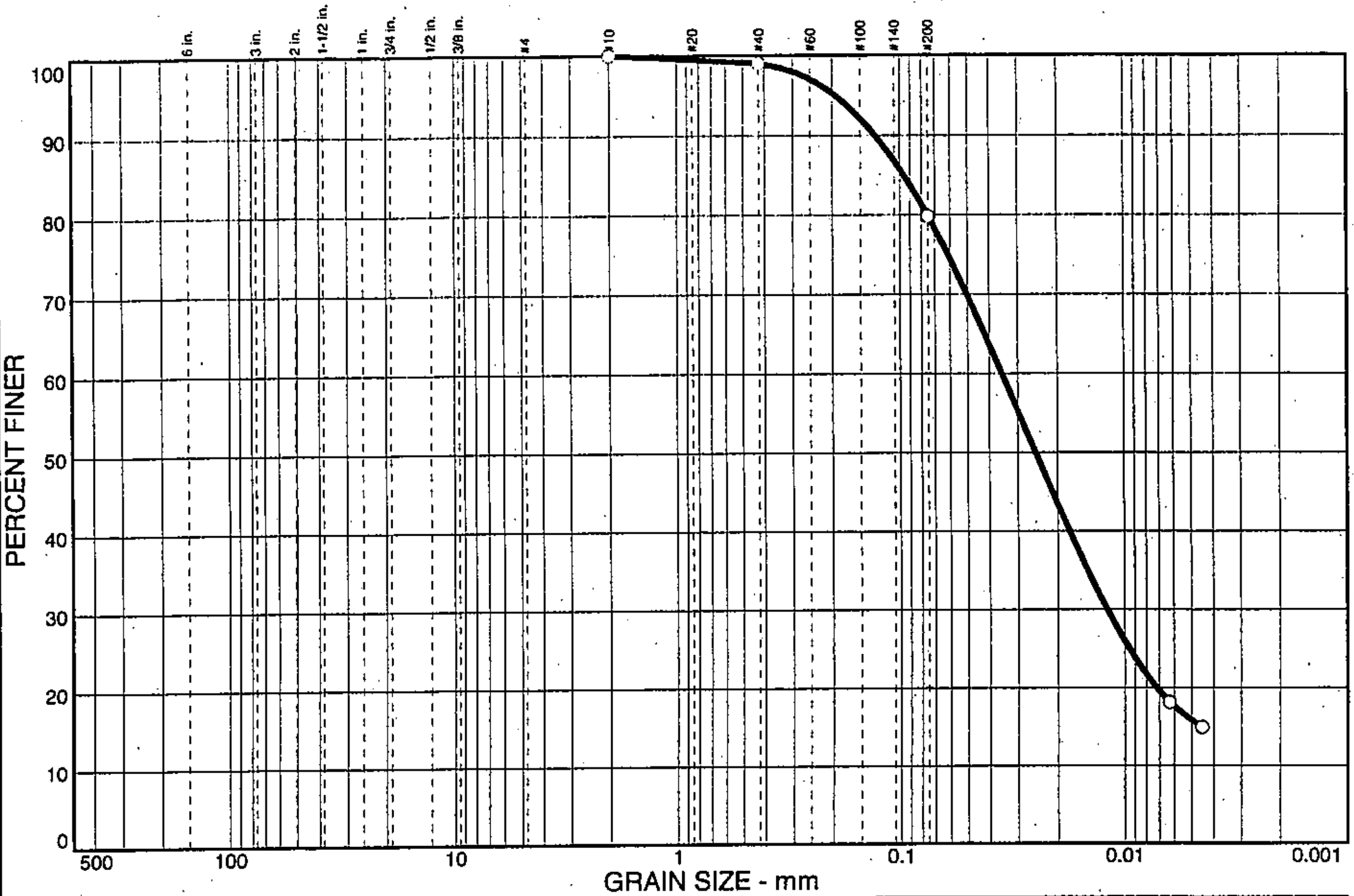
Moisture Content= 20.4%

Sample No.: 2 **Source of Sample:** TR-66 **Date:** 5/28/05
Location: **Elev./Depth:** 3.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	1.0	19.2	64.0	15.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.0		
#200	79.8		

* (no specification provided)

Soil Description

Silty clay with sand

Atterberg Limits

PL= 19 LL= 24 PI= 5

Coefficients

D_{85}= 0.0964 D_{60}= 0.0354 D_{50}= 0.0252
D_{30}= 0.0120 D_{15}= 0.0045 D_{10}=
C_u= C_c=

Classification

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

Remarks

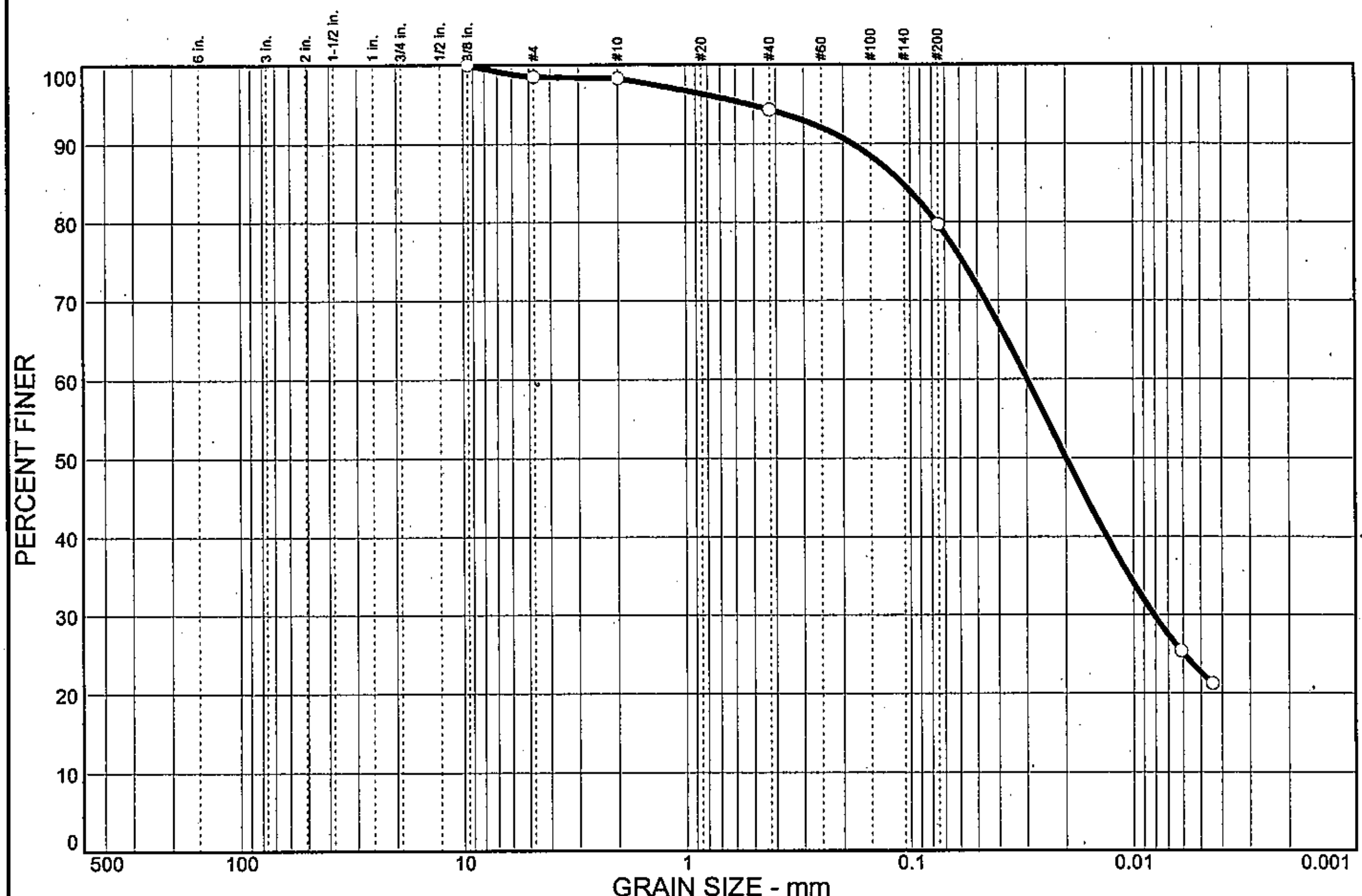
Moisture Content= 23.3%

Sample No.: 5 **Source of Sample:** TR-66 **Date:** 5/28/05
Location: **Elev./Depth:** 11.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.5	0.2	4.0	14.6	57.0	22.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.5		
#10	98.3		
#40	94.3		
#200	79.7		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 18 LL= 29 PI= 11

Coefficients

D_{85}= 0.108 D_{60}= 0.0299 D_{50}= 0.0199
D_{30}= 0.0081 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(7)

Remarks

Moisture Content= 21.8%

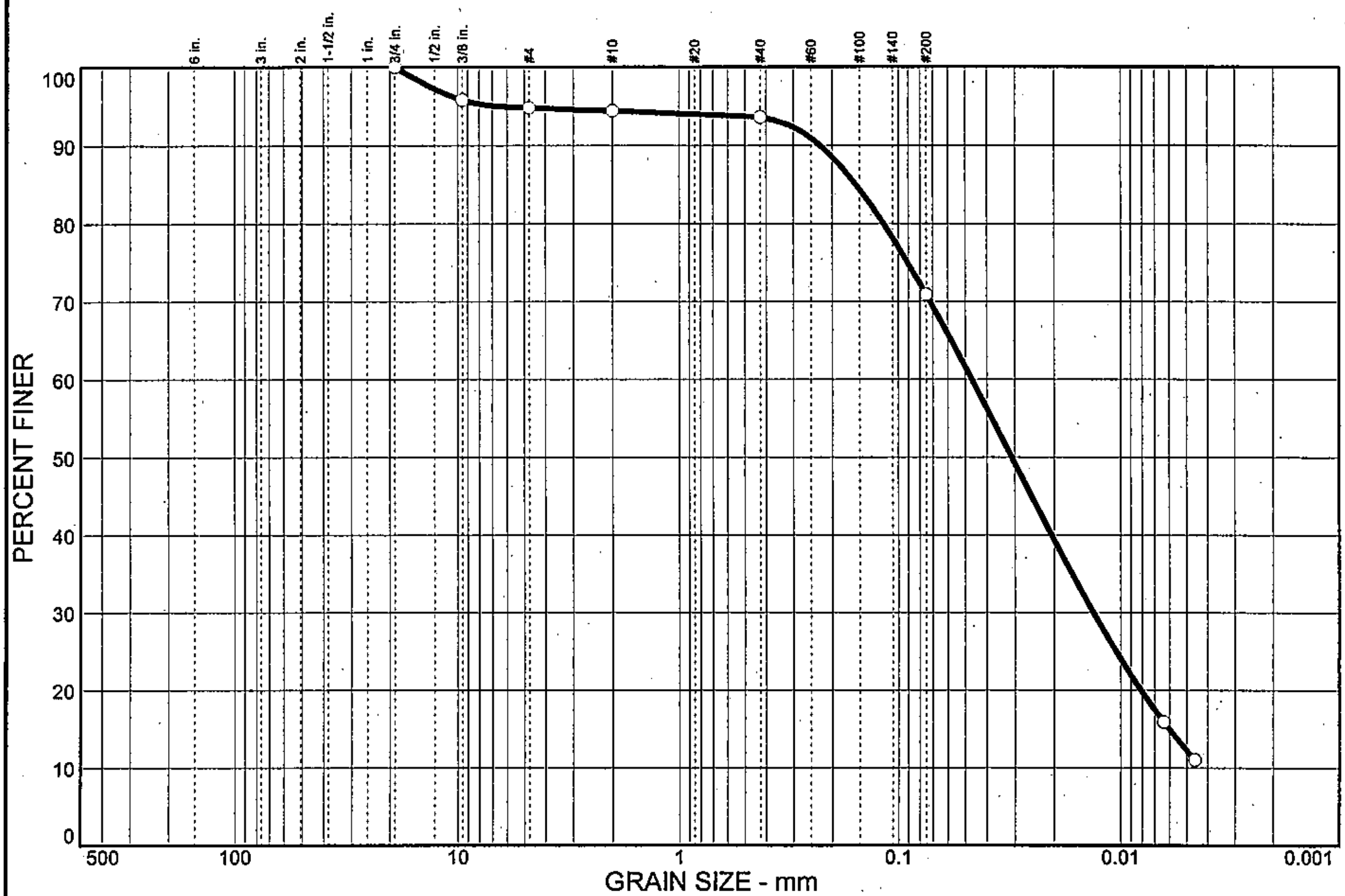
Sample No.: 2 **Source of Sample:** TR-70A **Date:** 10/19/06
Location: **Elev./Depth:** 3.5

DLZ

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

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.2	0.4	0.8	22.7	58.7	12.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	95.8		
#4	94.8		
#10	94.4		
#40	93.6		
#200	70.9		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= 20 LL= 21 PI= 1

Coefficients

D_{85}= 0.156 D_{60}= 0.0468 D_{50}= 0.0310
D_{30}= 0.0133 D_{15}= 0.0060 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

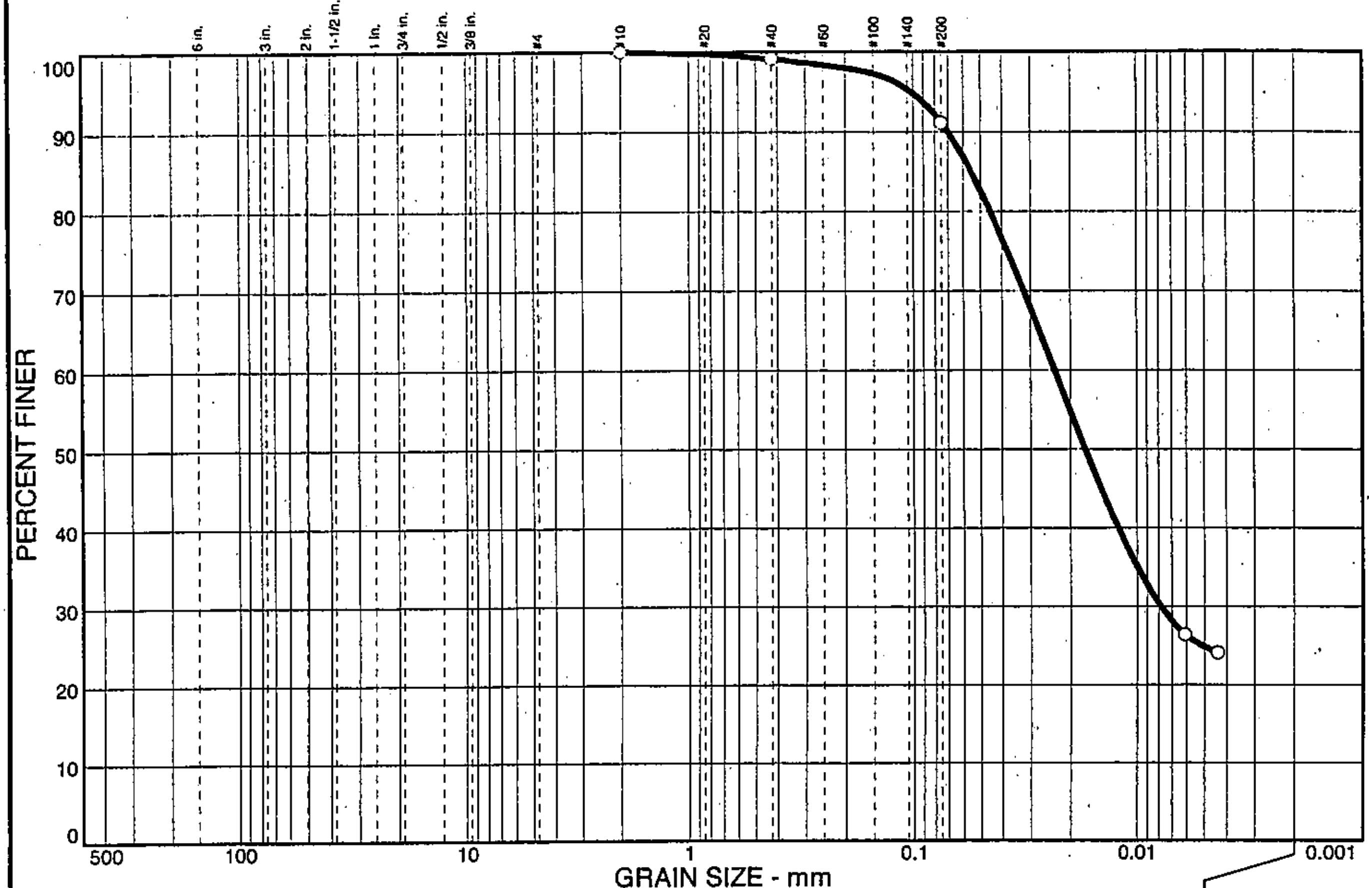
Moisture Content= 23.6%

Sample No.: 4 **Source of Sample:** TR-70A **Date:** 10/19/06
Location: **Elev./Depth:** 8.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.9	8.0	91.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.1		
#200	91.1		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 30 PI= 10

Coefficients

D_{85}= 0.0553 D_{60}= 0.0235 D_{50}= 0.0172
D_{30}= 0.0078 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(8)

Remarks

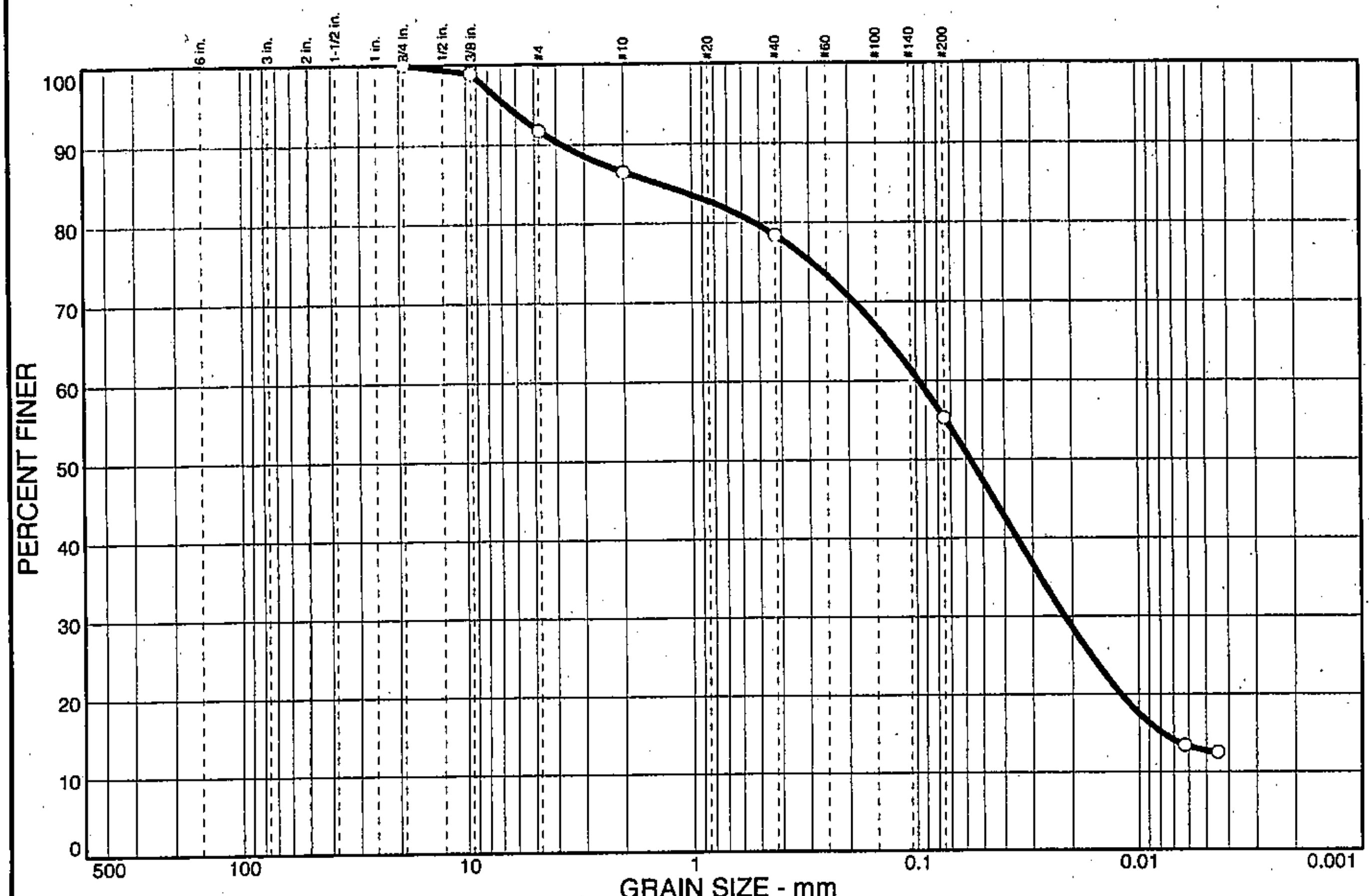
Moisture Content= 18.2%

Sample No.: 2 **Source of Sample:** TR-71A **Date:** 10/23/06
Location: **Elev./Depth:** 3.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	8.3	5.2	8.1	23.1	42.6	12.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.9		
#4	91.7		
#10	86.5		
#40	78.4		
#200	55.3		

* (no specification provided)

Soil Description

Sandy lean clay

Atterberg Limits

PL= 16 LL= 24 PI= 8

Coefficients

D_{85}= 1.41 D_{60}= 0.0974 D_{50}= 0.0572
D_{30}= 0.0217 D_{15}= 0.0079 D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(2)

Remarks

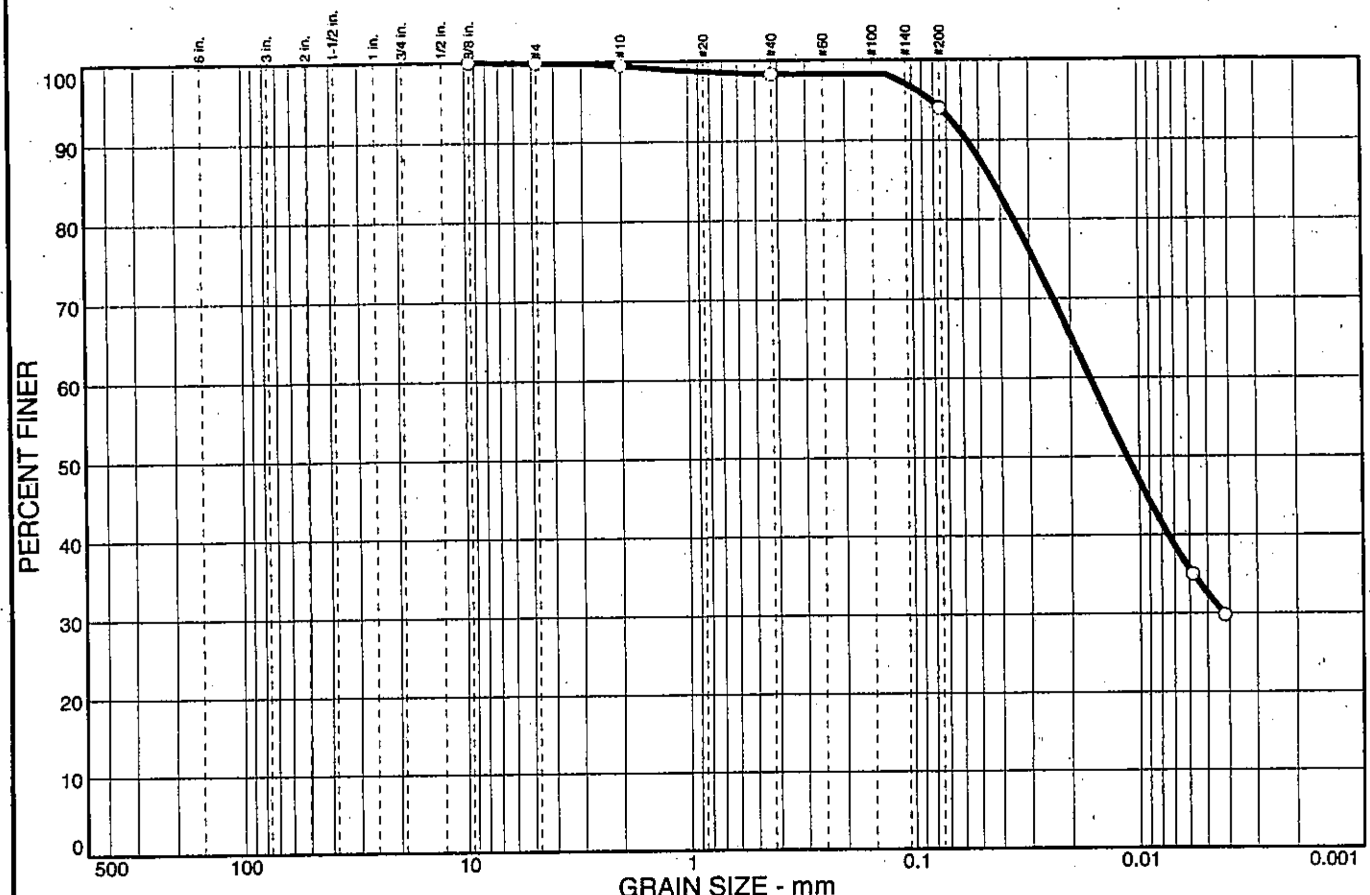
Moisture Content= 17.0%

Sample No.: 4 **Source of Sample:** TR-71A **Date:** 10/23/06
Location: **Elev./Depth:** 8.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.1	0.4	1.2	4.3	61.5	32.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.9		
#10	99.5		
#40	98.3		
#200	94.0		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 36 PI= 15

Coefficients

D_{85}= 0.0440 D_{60}= 0.0165 D_{50}= 0.0114
D_{30}= 0.0042 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(15)

Remarks

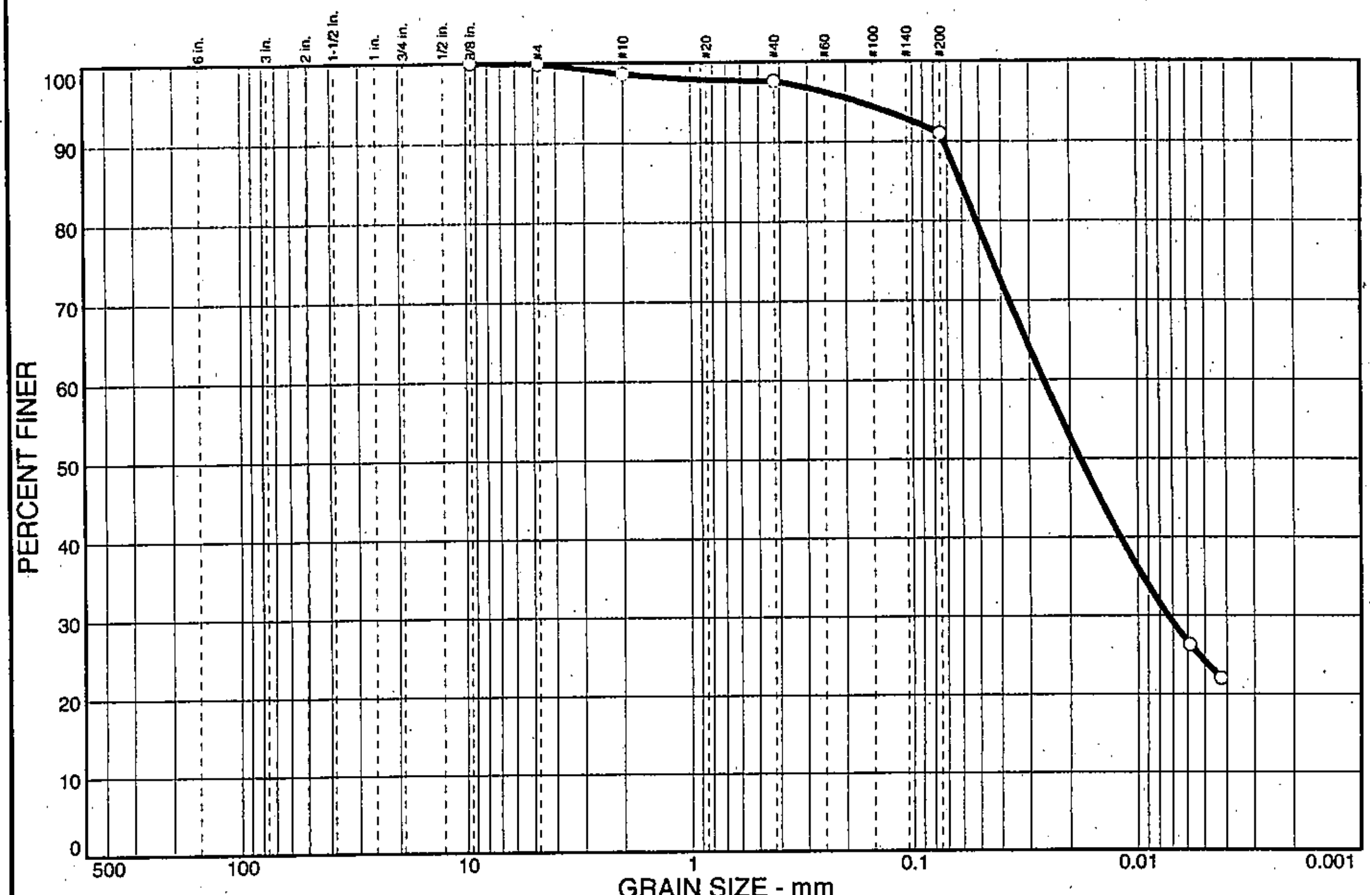
Moisture Content = 21.1%

Sample No.: 1 **Source of Sample:** TR-73A **Date:** 08/16/06
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.1	1.3	1.0	6.6	66.9	24.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.9		
#10	98.6		
#40	97.6		
#200	91.0		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 28 PI= 9

Coefficients

D_{85}= 0.0613 D_{60}= 0.0261 D_{50}= 0.0181
D_{30}= 0.0074 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(7)

Remarks

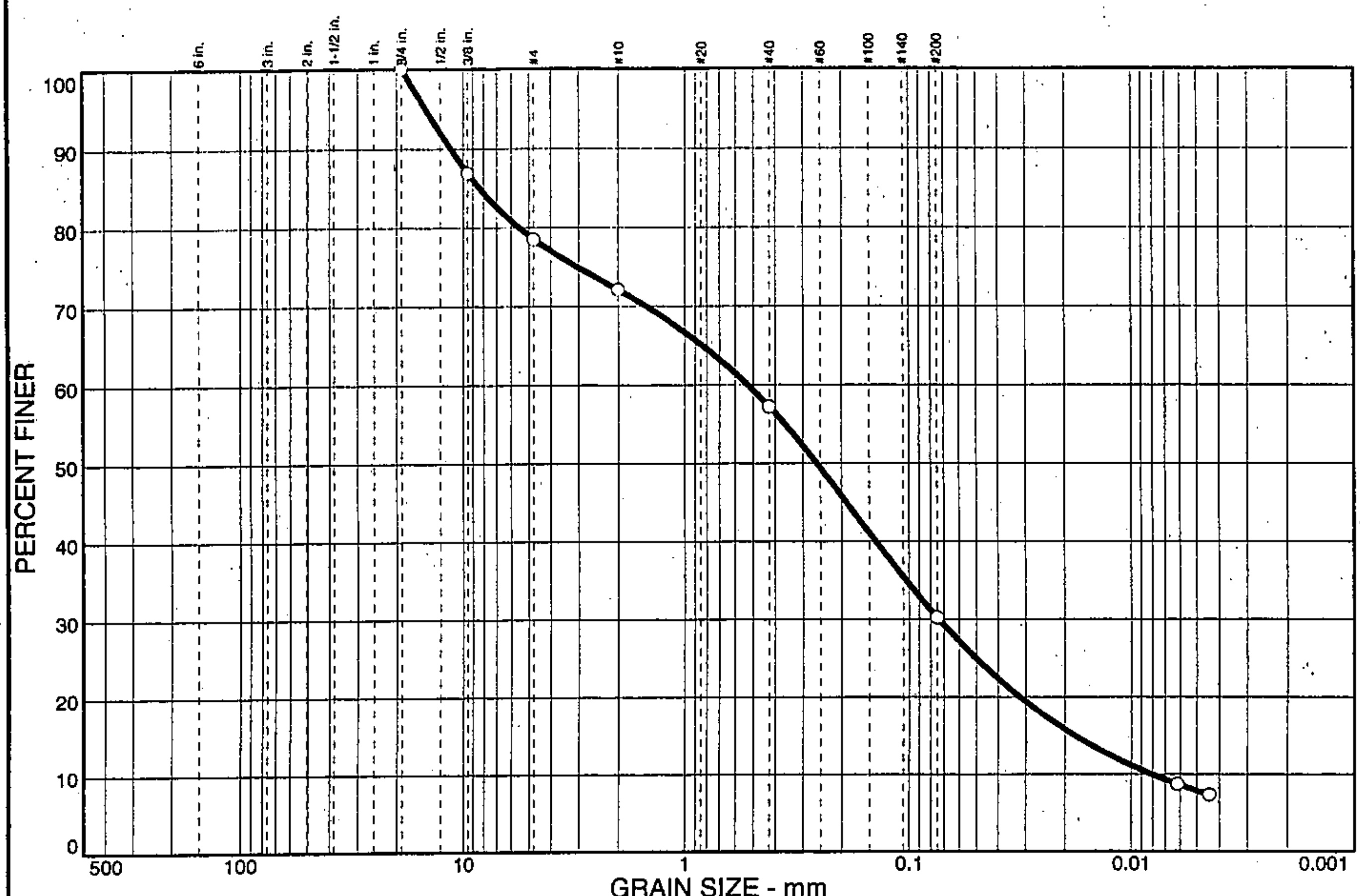
Moisture Content = 23.3%

Sample No.: 2 **Source of Sample:** TR-73A **Date:** 08/16/06
Location: **Elev./Depth:** 3.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	21.5	6.4	14.9	27.0	22.4	7.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	86.9		
#4	78.5		
#10	72.1		
#40	57.2		
#200	30.2		

* (no specification provided)

Soil Description

Silty, clayey sand with gravel

Atterberg Limits

PL= 14 LL= 18 PI= 4

Coefficients

D_{85}= 8.40 D_{60}= 0.529 D_{50}= 0.261
D_{30}= 0.0739 D_{15}= 0.0180 D_{10}= 0.0081
C_u= 65.31 C_c= 1.27

Classification

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

Remarks

Moisture Content = 13.4%

Sample No.: 5A **Source of Sample:** TR-73A **Date:** 08/16/06
Location: **Elev./Depth:** 11.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **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:** 10/3/2006

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
TR-62	1	9.3'-9.7'	1.989	1.987	1.991	1.989	4.691	4.696	4.677	4.688	2.357	0.0084284	590.26	154.40	33,550	**10,794**
			1.989	1.991	1.989											
TR-64	1	19.5'-19.8'	1.980	1.982	1.979	1.982	4.393	4.395	4.392	4.393	2.217	0.0078379	525.02	147.68	39,190	**12,706**
			1.982	1.983	1.984											
TR-66	1	22.8'-23.3'	1.981	1.982	1.980	1.982	4.694	4.678	4.677	4.683	2.362	0.0083603	548.51	144.64	35,380	**11,463**
			1.982	1.982	1.987											
TR-73A	1	19.2'-19.6'	1.969	1.980	1.979	1.977	4.538	4.541	4.539	4.539	2.296	0.0080589	521.31	142.61	34,560	**11,260**
			1.983	1.967	1.983											
TR-71A	1	21.6'-22.0'	1.980	1.990	1.981	1.981	4.781	4.771	4.760	4.771	2.409	0.008501	551.65	143.06	31,450	**10,209**
			1.980	1.976	1.976											

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: 6/12/07**

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-48	2	18.2'-18.6'	1.985	1.984	1.984	1.985	4.570	4.580	4.564	4.571	2.304	0.0081788	558.76	150.6	41,530	**13,427**
			1.984	1.984	1.986											
B-48	3	26.4'-*26.8'	1.967	1.966	1.966	1.969	4.620	4.600	4.603	4.608	2.340	0.0081141	565.70	153.7	32,980	**10,833**
			1.970	1.972	1.972											
B-49	2	21.3'-21.7'	1.977	1.978	1.979	1.977	4.324	4.336	4.320	4.327	2.188	0.0076852	544.57	156.2	36,660	**11,938**
			1.976	1.977	1.977											
B-49	3	27.6'-28.0'	1.964	1.967	1.967	1.967	4.414	4.425	4.418	4.419	2.246	0.00777	525.25	149.0	38,860	**12,784**
			1.968	1.969	1.969											
B-50	3	25.9'-26.3'	1.970	1.969	1.970	1.970	4.362	4.367	4.373	4.367	2.217	0.0077	504.20	144.4	23,090	**7,575**
			1.970	1.971	1.970											
B-51	2	18.1'-18.5'	1.983	1.985	1.985	1.985	4.392	4.386	4.378	4.385	2.209	0.0078486	534.35	150.1	37,330	**12,065**
			1.985	1.986	1.985											
B-51	3	21.3'-21.7'	1.989	1.983	1.984	1.984	4.262	4.263	4.267	4.264	2.149	0.0076251	503.75	145.6	23,510	**7,605**
			1.982	1.983	1.983											

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**

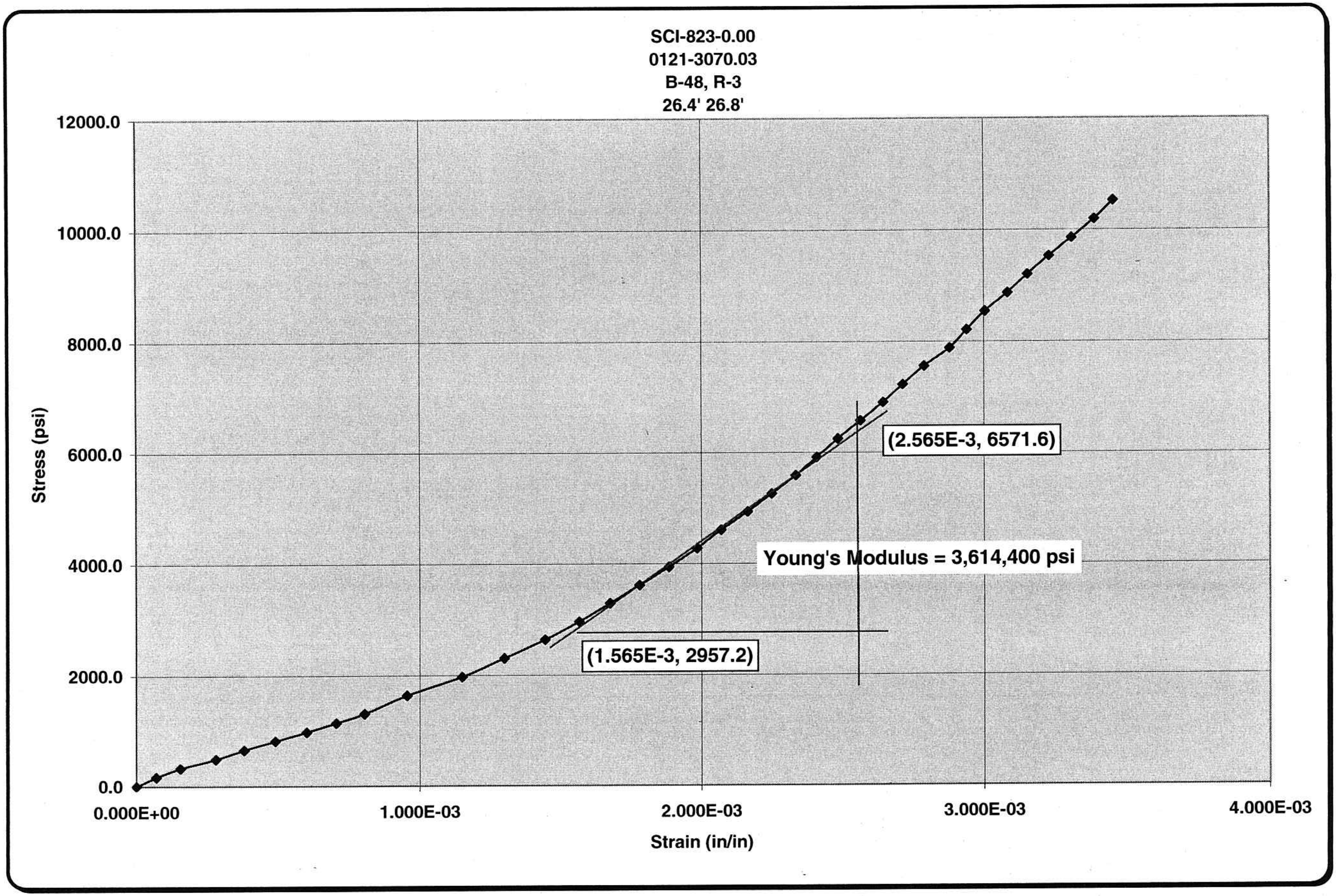
Project Name: SCI-823-0.00 | **Date: 6/12/07**

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-52	2	19.5'-19.9'	1.981	1.982	1.983	1.982	4.374	4.402	4.402	4.393	2.216	0.0078406	504.14	141.8	36,320	**11,770**
			1.982	1.982	1.983											
B-52	3	24.2'-24.6'	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-53	2	13.4'-13.6'	1.980	1.979	1.981	1.981	4.354	4.365	4.366	4.362	2.202	0.0077748	550.09	156.0	31,710	**10,290**
			1.983	1.980	1.982											
B-53	3	17.7'-18.1'	1.986	1.984	1.986	1.985	4.400	4.402	4.387	4.396	2.215	0.0078683	551.27	154.5	44,310	**14,321**
			1.985	1.984	1.984											

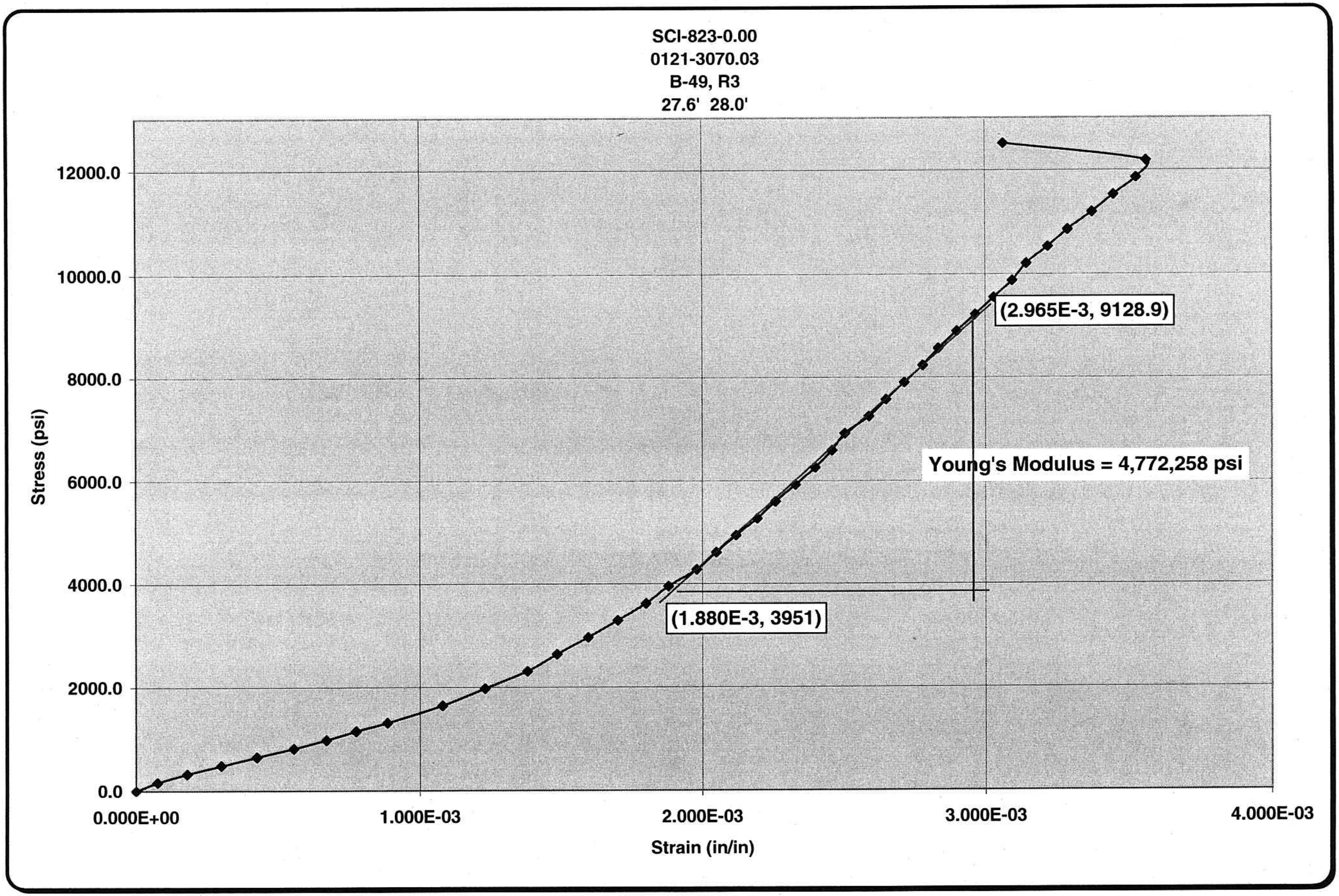
DLZ

Engineers * Architects * Scientists

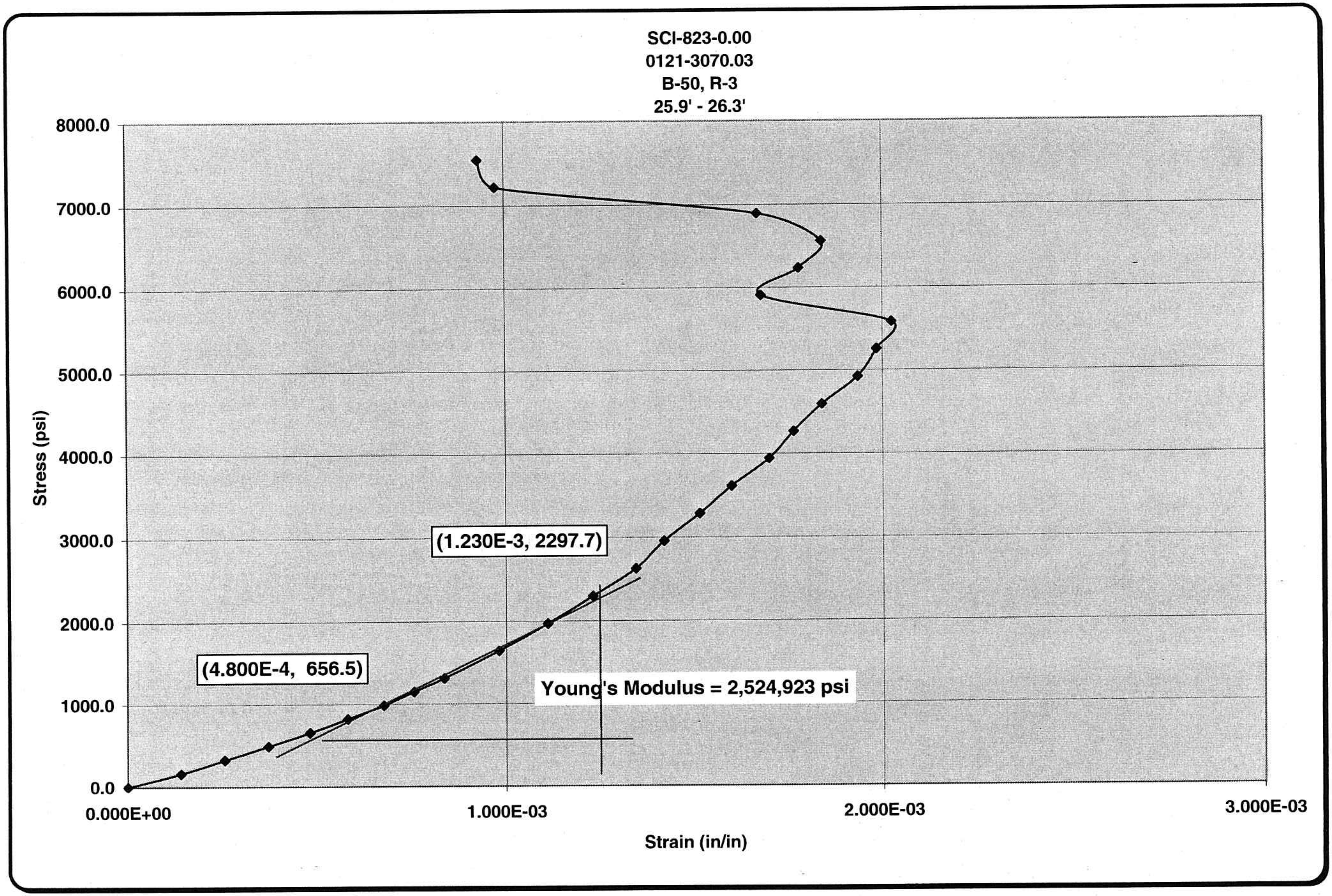
6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415



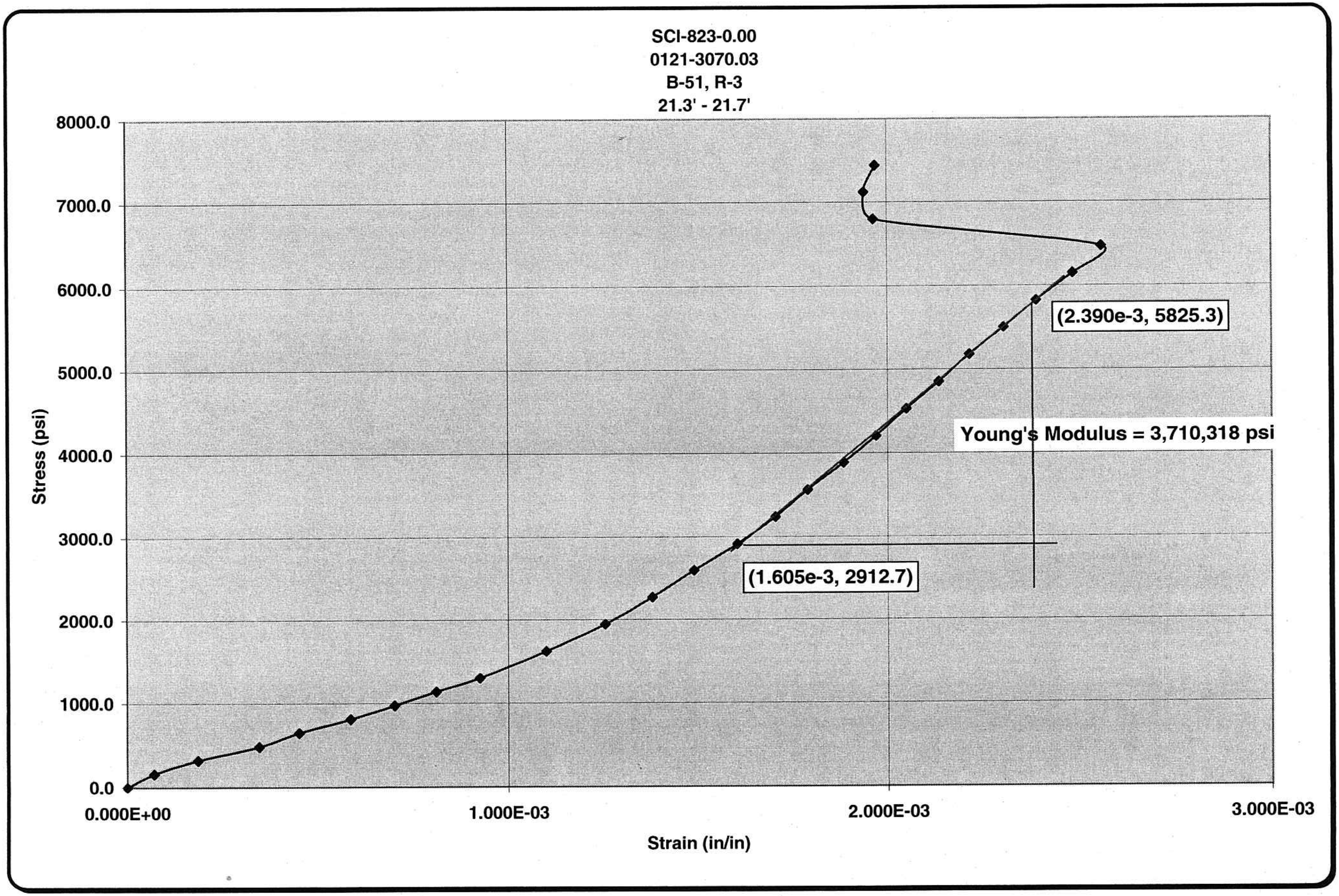
B-48 R-3 Chart 1



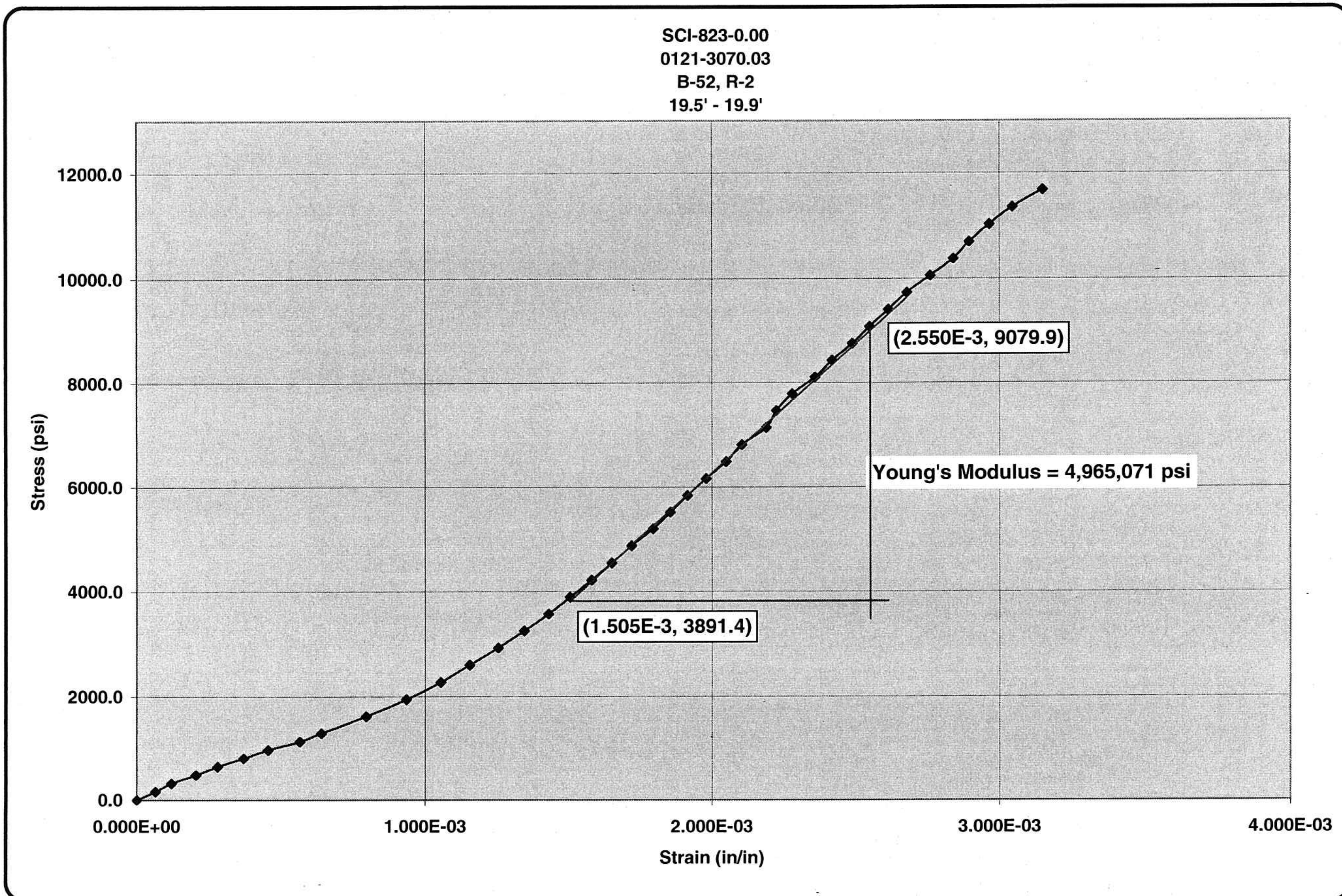
TB49 R3 Chart 1



B-50 R-3 Chart 1



B-51 R3 Chart 1



B52 R2 Chart 1

PROJECT : SCI-823-0.00, Portsmouth
DLZ Job No: 0121-3070.03
Date : 6/12/2007

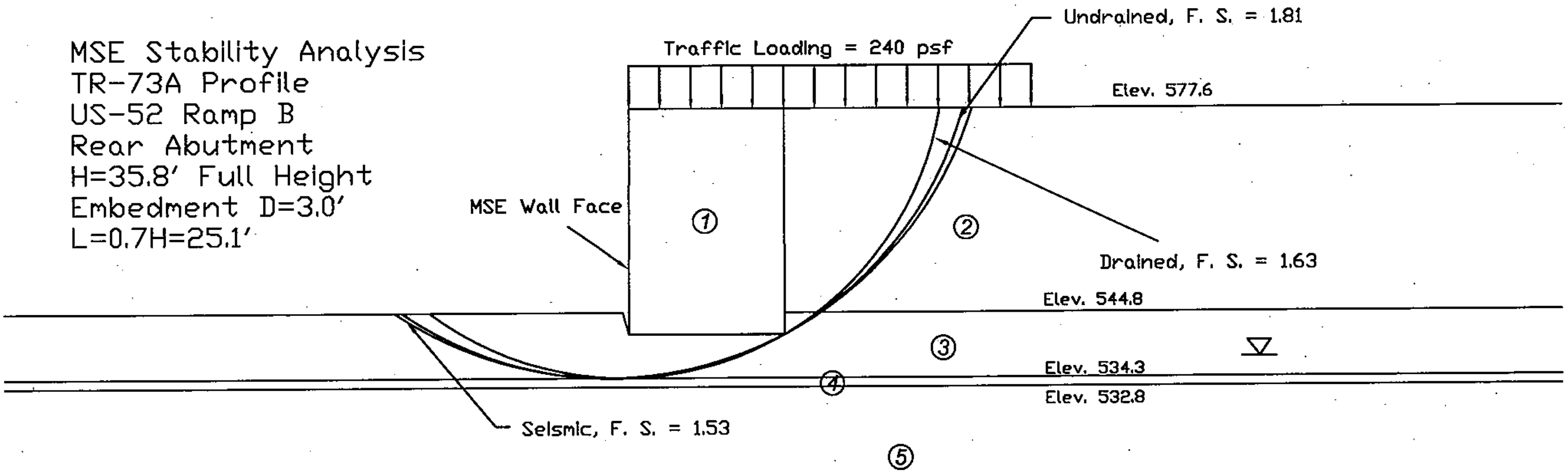
Boring No	Depth (feet)	Diameter (inch)						Diameter	Length (inch)			Length	(L / D)	Weight (gram) (G)	Volume V (ft3) 0.0004543*D*D*L	Unit Wt. (pcf) G/(453.6*V)
		D1	D2	D3	D4	D5	D6	Average (D)	L1	L2	L3	Average (L)				
B-48, R2	18.2-18.6	1.985	1.984	1.984	1.984	1.986	1.984	1.985	4.570	4.580	4.564	4.571	2.3	558.76	0.00818262	150.5
B-48, R-3	26.4-26.8	1.967	1.967	1.966	1.970	1.972	1.972	1.969	4.620	4.600	4.603	4.608	2.3	565.70	0.00811795	153.6
B-49, R-2	21.3-21.7	1.977	1.978	1.979	1.976	1.977	1.977	1.977	4.324	4.336	4.320	4.327	2.2	544.57	0.00768883	156.1
B-49, R-3	27.6-28.0	1.964	1.967	1.967	1.968	1.969	1.969	1.967	4.414	4.425	4.418	4.419	2.2	525.25	0.00777369	149.0
B-50, R-3	25.09-26.3	1.970	1.969	1.970	1.970	1.971	1.970	1.970	4.662	4.367	4.373	4.467	2.3	504.70	0.00788003	141.2
B-51, R-2	18.1-18.5	1.983	1.985	1.985	1.985	1.986	1.985	1.985	4.392	4.386	4.378	4.385	2.2	534.35	0.00785232	150.0
B-51, R-3	21.3-21.7	1.989	1.983	1.984	1.982	1.983	1.983	1.984	4.246	4.263	4.267	4.259	2.1	503.75	0.00761911	145.8
B-52, R-2	19.5-19.9	1.981	1.982	1.983	1.982	1.982	1.983	1.982	4.374	4.402	4.402	4.393	2.2	504.74	0.00784433	141.9
B-52, R-3	24.2-24.6	1.983	1.983	1.984	1.983	1.983	1.985	1.984	4.531	4.522	4.527	4.527	2.3	512.36	0.0080945	139.5
B-53, R-2	13.4-13.7	1.980	1.979	1.981	1.983	1.980	1.982	1.981	4.354	4.365	4.366	4.362	2.2	550.09	0.00777849	155.9
B-53, R-3	17.7-18.1	1.986	1.984	1.986	1.985	1.984	1.984	1.985	4.400	4.402	4.387	4.396	2.2	551.27	0.00787201	154.4

APPENDIX IV

MSE Wall Analysis Results -
Global Stabilty Analyses
Bearing Capacity and Stability Calculations
Settlement Calculations
Drilled Shaft – End Bearing and Side Resistance Calculations
Drilled Shaft – Laterally Loaded LPile Analysis

Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Stiff	Silt	1667	0	0	29	120
Material 4	M. Dense	Gravel	0	32	0	32	125
Material 5		BEDROCK	5000	45	5000	45	150

MSE Stability Analysis
TR-73A Profile
US-52 Ramp B
Rear Abutment
H=35.8' Full Height
Embedment D=3.0'
L=0.7H=25.1'



sheet 1 of 51
EWT 6-18-07

US-52 Ramp B over Ohio River Road
BASED ON BORING TR-73A PROFILE
WITHOUT UNDERCUT

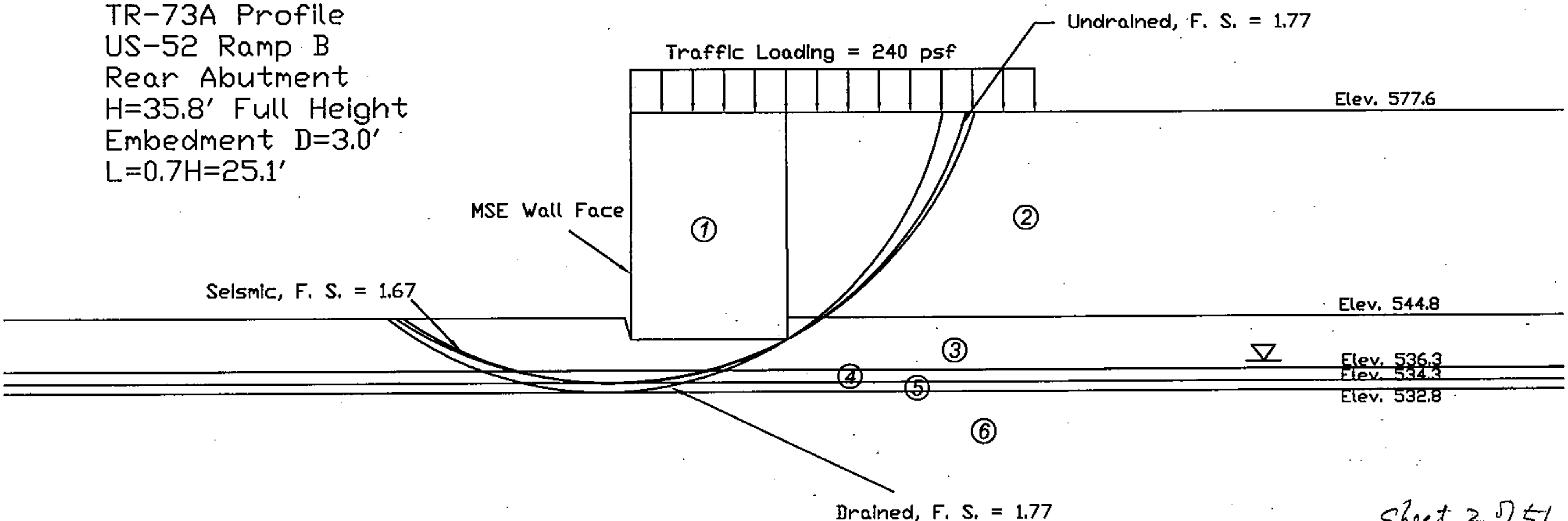
MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: EWT | DATE 6/14/07

Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Granular Fill	0	34	0	34	120
Material 4	Stiff	Silt	1667	0	0	29	120
Material 5	M. Dense	Gravel	0	32	0	32	125
Material 6		Bedrock	5000	45	5000	45	150

MSE Stability Analysis
TR-73A Profile
US-52 Ramp B
Rear Abutment
H=35.8' Full Height
Embedment D=3.0'
L=0.7H=25.1'



Sheet 2 of 51
EWT 6-18-07

US-52 Ramp B over Ohio River Road
BASED ON BORING TR-73A PROFILE
WITH UNDERCUT

MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: EWT	DATE 06/14/07

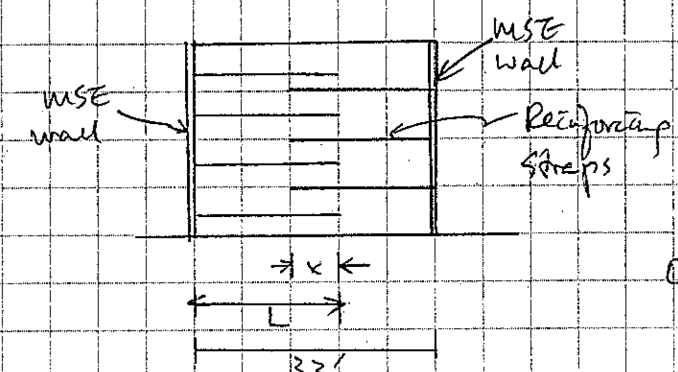
DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth ByPass
SUBJECT MSE wall Bearing Capacity Modification of Ka

PROJECT NO. 0121-3070.03
SHEET NO. 3 OF 57
COMP. BY EWT DATE 6-18-07
CHECKED BY DAA DATE 6-18-07

Based on profile received on 5-17-07

1) Rear Abutment MSE wall as back to back walls
* 90° Turn Back from abutment wall face
* 33' - Ramp B width as per most current cross-sections
* H = 32.8' + 3.0' = 35.8' Including embedment



X = overlap of reinforcing straps
$L = 0.7H = 0.7(35.8) = 25.1'$

$X = 2L - 33 = 2 \times 25.1 - 33$
$= 17.2'$

Overlap as a ratio of height:

$\frac{17.2}{35.8} = 0.48H$

If overlap > 0.3 H → use Ka = 0 for external stability
(see attached, Ref FHWA-NHI-00-043)

2) Analyze forward Abutment MSE wall as back to back walls
* 90° Turn back from abutment wall face
* 33' - Ramp B width
* H = 38.1 + 3.0 = 41.1 Including embedment

X = overlap of reinforcing straps
$L = 0.7H = 0.7(41.1) = 28.8'$

$X = (2L) - 33 = 24.6'$ ⇒ $\frac{24.6}{41.1} = 0.6H$ overlap

If overlap > 0.3 H use Ka = 0 for external stability
Ref FHWA-NHI-00-043 (attached)

Ref: FHWA-NHI-00-043

5.4 BACK-TO-BACK WALLS

For walls which are built back-to-back as shown in figure 50, a modified value of backfill thrust influences the external stability calculations. As indicated in figure 50, two cases can be considered.

- For Case I, the overall base width is large enough so that each wall behaves and can be designed independently. In particular, there is no overlapping of the reinforcements. Theoretically, if the distance, D, between the two walls is shorter than:

  $$D = H_1 \tan(45° - \varphi/2) \tag{55}$$

  then the active wedges at the back of each wall cannot fully spread out and the active thrust is reduced. However, it is assumed that for values of:

  $$D > H_1 \tan(45° - \varphi/2) \approx 0.5\, H_1 \tag{56}$$

  full active thrust is mobilized.

- For Case II, there is an overlapping of the reinforcements such that the two walls interact. When the overlap, L_R, is greater than 0.3 H_2, where H_2 is the shorter of the parallel walls, no active earth thrust from the backfill needs to be considered for external stability calculations. For intermediate geometries between Case I and Case II, the active earth thrust may be linearly interpolated from the full active case to zero. For Case II geometries with overlaps greater than 0.3 H_2, L/H ratios for each wall as low as 0.6 may be considered.

  Considering this case, designers might be tempted to use single reinforcements connected to both wall facings. This alternative completely changes the strain patterns in the structure and results in higher reinforcement tensions such that the design method in this manual is no longer applicable. In addition, difficulties in maintaining wall alignment could be encountered during construction, especially when the walls are not in a tangent section.

  Based on a performance review, back-to-back walls with overlapping reinforcements may be designed for static load conditions with a distance between parallel facing as low as L/H = 0.6, where H is the height of each wall, and for conditions where the seismic horizontal accelerations at the foundation level is less than 0.05g. For walls in more seismically active areas (up to 0.19g) a distance of 1.1H_1 is presently recommended. For walls subjected to significant seismic loading (up to 0.40g) successful performance has been observed when the distance between parallel facings was at least 1.2H_1.

  Justification of narrower back-to-back distances (< 1.1H_1) between faces in seismically active areas require a more detailed analysis be performed to include effects of potential non-uniform distribution of seismic and inertial forces within the wall, as suggested by numerical studies and not provided for in the present design methodology.

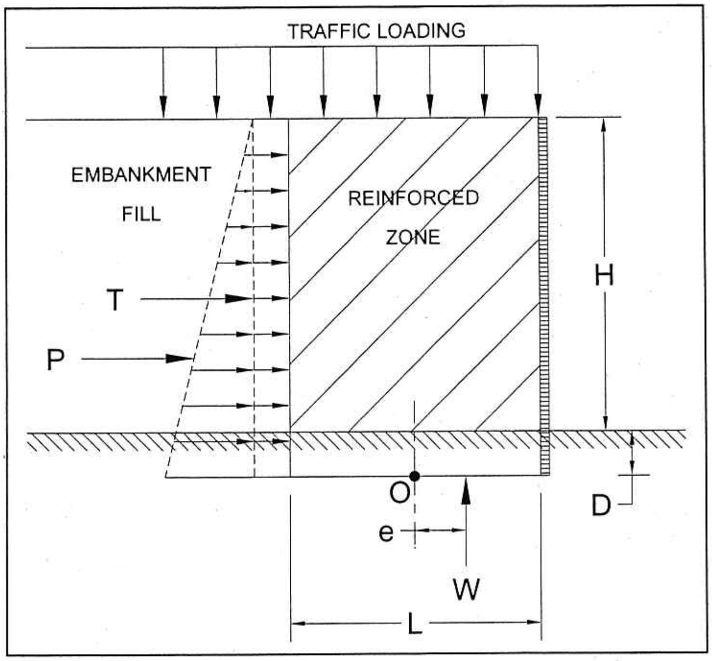
DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	5 OF 51
Item	MSE Wall Bearing Capacity (US 52 Ramp B)	COMP. BY	[illegible] DATE 6-18-07
Rear Abutment, Based upon boring TR-73A		CHECKED BY	DAA DATE 6-18-07

Analysis assumes overlapping soil reinforcement — Based upon existing foundation soils

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1667	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	25.06	ft	Length of MSE reinforcement
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	35.8	ft	
H	=	32.8	ft	Height of wall
Ka	=	**0.00**		**Ka = 0.0*K, Due to overlap**
Γ Pa	=	11.933	ft	Moment arm
Γ Wt	=	17.9	ft	Moment arm
B'	=	25.06	ft	
γ'	=	57.6	pcf	
W_t		6,014	lb/ft of wall	Weight from traffic
W_{mse}	=	107,658	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,536 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 8,741 psf

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

q_{ALL} = 3,496 psf

Factor of Safety = **1.93** **No Good**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 16,799 psf

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

q_{ALL} = 6,720 psf

Factor of Safety = **3.70** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force — Kern

e = 0.00 ft — e < L/6 = 4.18 ft

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	6 OF 51
Item	MSE Wall Stability (US 52 Ramp B)	COMP. BY	GWT DATE 6-18-07
Rear Abutment, Based upon Boring TR-73A		CHECKED BY	DAA DATE 6-18-07

Analysis assumes overlapping soil reinforcement

Based upon existing foundation soils

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=32.8'
2 Sssume bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 For External Stability, Ka=0.0*K from reinforcing strap overlap

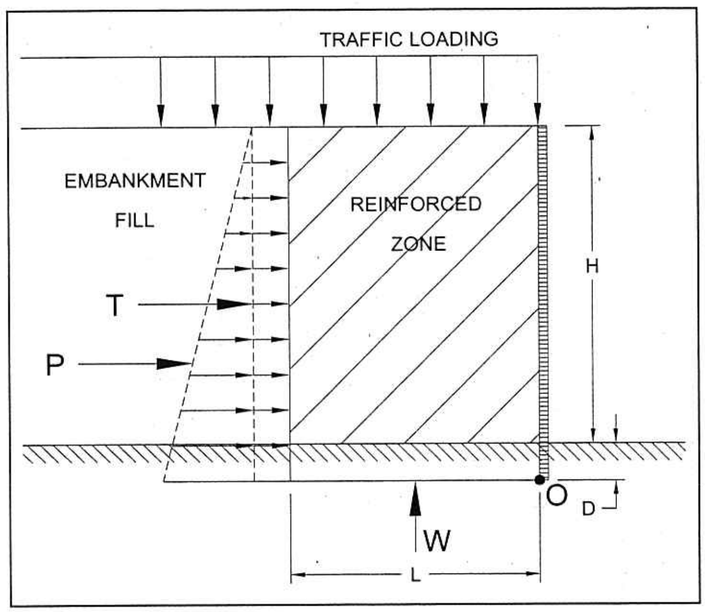
Wall Properties			Foundational Soil Properties			
H+D =	35.8	feet	c =	1667	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	29	deg	Friction angle
L =	25.06	feet	ω_T =	240	psf	Traffic loading
L factor =	0.70		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE

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

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ **K_a = 0.00**

P_a = 0 lbs per foot of wall



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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.37

P_r = 39,833 lbs per foot of wall

USE THIS VALUE

$$P_r = L(c)$$ (Undrained)

P_r = 41,775 lbs per foot of wall

Use Drained Value

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

Calculated FS = ###### Required FS = 1.50

Resistance Against Sliding is **OK**

FS = ∞, FS > 1.5 due to Ka = 0.0

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 1,348,952 lb-ft

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

$\Sigma M_{overturning}$ = 0 lb-ft

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

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

Calculated FS = ###### Required FS = 2.00

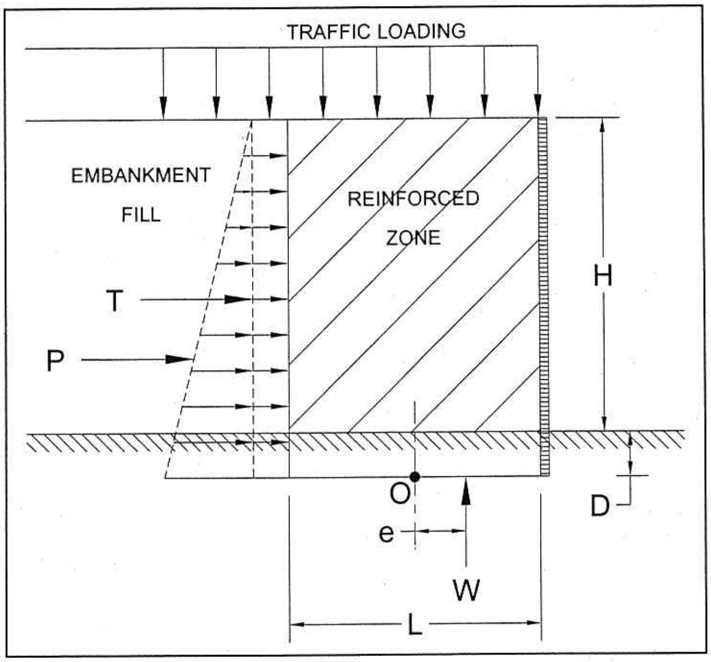
Resistance Against Overturning is **OK**

FS = ∞, FS > 2.0 due to Ka = 0.0

Analysis assumes overlapping soil reinforcement　　Based upon undercut of existing foundation soils

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

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

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	25.06	ft	Length of MSE reinforcement
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	35.8	ft	
H	=	32.8	ft	Height of wall
Ka	=	**0.00**		**Ka = 0.0*K, Due to overlap**
Γ Pa	=	11.933	ft	Moment arm
Γ Wt	=	17.9	ft	Moment arm
B'	=	25.06	ft	
γ'	=	57.6	pcf	
W_t		6,014	lb/ft of wall	Weight from traffic
W_{mse}	=	107,658	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,536 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 19,346 psf

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

q_{ALL} = 7,738 psf

Factor of Safety = **4.26** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 19,346 psf

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

q_{ALL} = 7,738 psf

Factor of Safety = **4.26** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	30.14	N_c	30.14
N_q	18.40	N_q	18.40
N_γ	22.40	N_γ	22.40

Eccentricity of Resultant Force

e = 0.00 ft

Kern

e < L/6 = 4.18 ft

DLZ

SUBJECT				
Client	TranSystems Corp	JOB NUMBER	0121-3070.03	
Project	SCI-823 Portsmouth Bypass	SHEET NO.	8	OF 51
Item	MSE Wall Stability(US 52 Ramp B)	COMP. BY	EWT	DATE 6-8-07
Rear Abutment, Based upon Boring TR-73A		CHECKED BY	DAA	DATE 6-18-07

Analysis assumes overlapping soil reinforcement   Based upon undercut of existing foundation soils

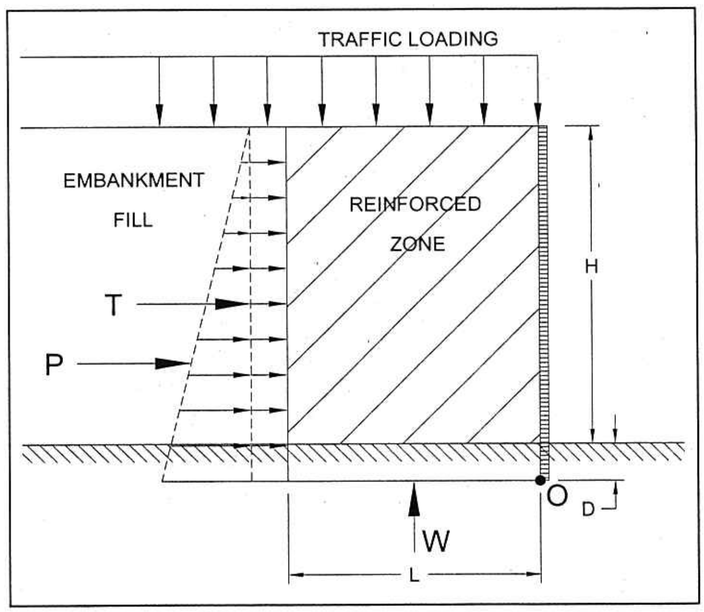
STABILITY OF MSE WALL

Assumptions:

1. Estimated height of embankment; H=32.8'
2. Sssume bridge is supported on deep foundations
3. Ground water; Dw=0.0'
4. Traffic loading is neglected in resisting forces
5. For External Stability, Ka=0.0*K from reinforcing strap overlap

Wall Properties			Foundational Soil Properties			
H+D =	35.8	feet	c =	0	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	30	deg	Friction angle
L =	25.06	feet	ω_T =	240	psf	Traffic loading
L factor =	0.70		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE



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

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = **0.00**

P_a = 0 lbs per foot of wall

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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.39

P_r = 41,987 lbs per foot of wall

USE THIS VALUE

$$P_r = L(c)$$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

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

Calculated	Required
FS = ######	FS = 1.50

Resistance Against Sliding is **OK**

FS = ∞, FS > 1.5 due to ka = 0.0

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 1,348,952 lb-ft $$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 0 lb-ft $$\Sigma M_{overturnig} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

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

Calculated	Required
FS = ######	FS = 2.00

Resistance Against Overturning is **OK**

FS = ∞, FS > 2.0 due to ka = 0.0

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	9 OF 51
Item	MSE Wall Stability(US 52 Ramp B)	COMP. BY	EWT DATE 6-18-07
Forward Abutment Wall, Based on TR-62		CHECKED BY	DAA DATE 6-18-07

Analysis assumes overlapping soil reinforcement

Based upon MSE wall leveling pad founded on bedrock

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=38.1'
2 Sssume bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 For External Stability, Ka=0.0*K from reinforcing strap overlap

Wall Properties

H+D	=	41.1	feet
γ_{mse}	=	120	pcf
L	=	28.852	feet
L factor	=	0.70	
ϕ	=	30	deg

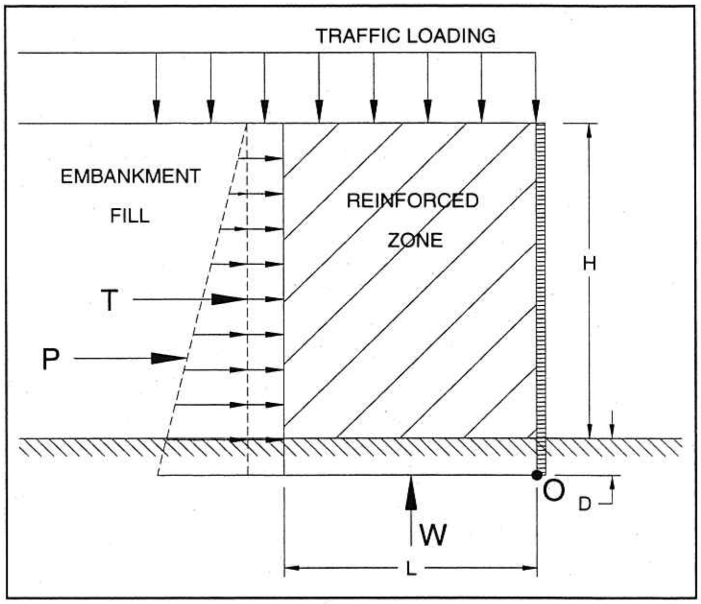
Foundational Soil Properties

c	=	0	psf	Cohesion
ϕ'	=	34	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



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

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ $\quad K_a = 0.00$

P_a = 0 lbs per foot of wall

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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ $\quad \mu = 0.45$

P_r = 64,035 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

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

Calculated FS = ###### Required FS = 1.50

Resistance Against Sliding is **OK**

FS = ∞ , FS > 1.5 due to Ka = 0.0

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 2,052,820 lb-ft

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

$\Sigma M_{overturning}$ = 0 lb-ft

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

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

Calculated FS = ###### Required FS = 2.00

Resistance Against Overturning is **OK**

FS = ∞

FS > 2.0 due to Ka = 0.0

DLZ

SUBJECT	Client TranSystems Corp	JOB NUMBER	0121-3070.03
	Project SCI-823 Portsmouth Bypass	SHEET NO.	10 OF 51
	Item MSE Wall Stability(US 52 Ramp B)	COMP. BY	GWT DATE 6-18-07
	Forward Abutment Wall, Based on TR-62	CHECKED BY	DAA DATE 6-18-07

Analysis assumes **no** overlapping soil reinforcement

Based upon MSE wall leveling pad founded on bedrock

STABILITY OF MSE WALL

Assumptions:

1. Estimated height of embankment; H=38.1'
2. Sssume bridge is supported on deep foundations
3. Ground water; Dw=0.0'
4. Traffic loading is neglected in resisting forces
5. No overlapping of soil reinforcement

Wall Properties

H+D	=	41.1	feet
γ_{mse}	=	120	pcf
L	=	28.852	feet
L factor	=	0.70	Length factor-range (0.7 - 1.0)
ϕ	=	30	deg — Friction Angle of Embankment Fill

Foundational Soil Properties

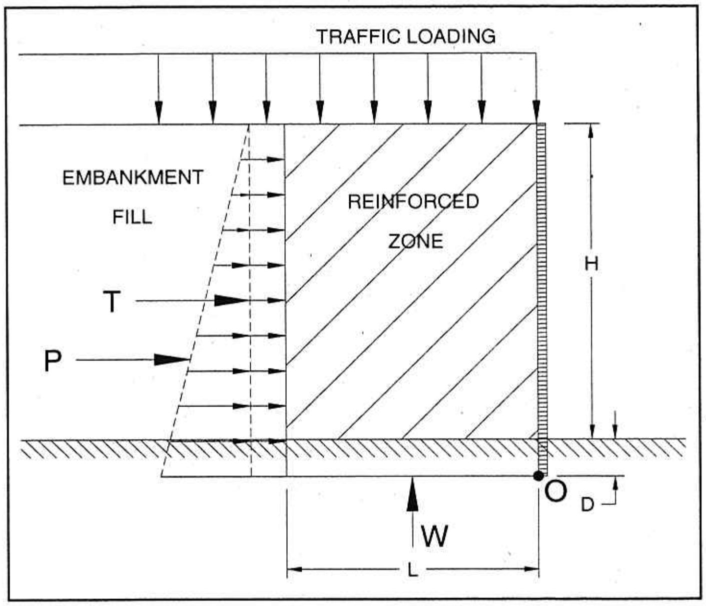
c	=	0	psf	Cohesion
ϕ'	=	34	deg	Friction angle
ω_T	=	240	psf	Traffic loading

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 36,701 lbs per foot of wall



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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.45

P_r = 64,035 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

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

Calculated FS = **1.74** Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 2,052,820 lb-ft $$\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 525,108 lb-ft $$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

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

Calculated FS = **3.91** Required FS = 2.00

Resistance Against Overturning is **OK**

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth Bypass SHEET NO. 11 OF 51
SUBJECT Consolidation Parameters COMP. BY GWT DATE 6-12-07
US 52 Ramp B Bridge over Ohio River Road CHECKED BY DAA DATE 6-18-07

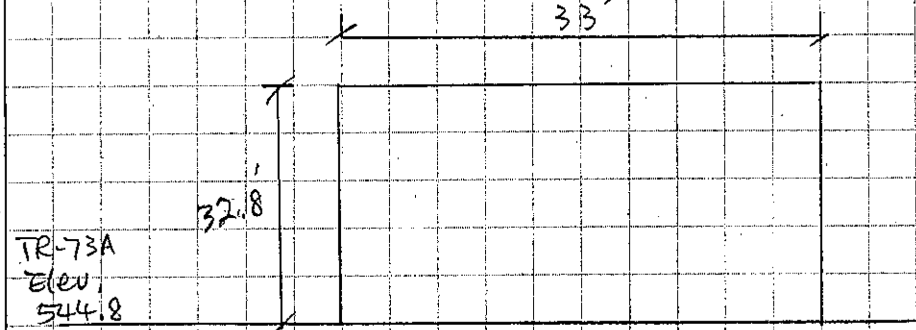
MSE Walls at Rear Abutment of Ramp B Bridge
@ station 35 + 42.74

Comparing subsurface conditions at Borings TR71A and TR73A.
Boring TR-73A appears to be more critical.

Boring TR-73A : Ground surface elevation @ TR-73A = 544.8

Wall height including embankment fill = 578.13 - 542.32
= 35.8'

Where Elev 578.13 is proposed finished grade



- Assume 90° Turnback
- Assume soils are normally consolidated
- Assume $G_s = 2.65$

① compacted Granular fill $\gamma = 120$ pcf, Incompressible (MSE Fill and 12" Recompacted m'tl below bottom of leveling pad)

540.3

6' ② A-4b (cohesive) ▽ Elev 537.5 $\gamma = 120$ pcf, $\bar{w} = 23\%$ $C_c = 0.23$ (FHWA-NHI-00-045)
Assume saturated $e_0 = wG_s = 0.61$

534.3

1.5' ③ A-2-4 $\gamma = 125$ pcf $N = 27$ $N' \approx 1.25 \times 27 = 33$ ⇒ $C' = 110$ (FHWA)
⇒ $C_c = 0.0182$, $e_0 = 1.0$ (assumed) (see calculations below)

532.8

11.9' ④ Bedrock $\gamma = 145$ pcf

520.9

Consolidation parameters are estimated from FHWA-NHI-00-045 for cohesive soils based on moisture, and cohesionless soils based on average SPT N-values

$$\frac{1}{C'} = \frac{C_c}{1+e_0}$$ Assumed $e_0 = 1.0$

$$\frac{1}{C'} = \frac{C_c}{1+1.0} \Rightarrow C_c = \frac{2}{C'}$$

When $C' = 110$, $$C_c = \frac{2}{110} = 0.0182$$

$$\text{Differential Settlement} = \frac{(5.16 - 3.33)/12}{33/2} \times 100\% = 0.92\%$$

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth ByPass SHEET NO. 12 OF 51
SUBJECT Consolidation Parameters COMP. BY EWT DATE 6-12-07
US52 Ramp B over Ohio River Road CHECKED BY DAA DATE 6-18-07

Replace 5' of existing soils below the bottom of leveling pad

$$\text{Differential Settlement} = \frac{(1.62 - 1.05)/12}{33/2} \times 100\%$$

$$= 0.29\%$$

Sheet 13 of 51
EWT 6-18-07

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name	: US 52 Ramp B	Client	: ODOT9
File Name	: B-RA	Project Manager	: PN
Date	: 6/12/10	Computed by	: EWT

Settlement for X-Direction

Embank. slope, x direc.	= 0.10 (ft)	Height of fill H	= 32.80 (ft)
y direc.	= 0.10 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 33.00 (ft)	p load/unit area	= 3936.00 (psf)
Embankment bottom width	= 33.20 (ft)	Foundation Elev.	= 544.80 (ft)
Ground Surface Elev.	= 544.80 (ft)		
Water table Elev.	= 537.50 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	4.5	-----	-----	-----	120.00	----	----
2	COMP.	6.0	0.230	0.000	0.000	120.00	2.65	0.61
3	COMP.	1.5	0.018	0.000	0.000	125.00	2.65	1.00
4	INCOMP.	11.9	-----	-----	-----	145.00	----	----

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	6.00	537.30	887.52	887.52
3	1.50	533.55	1107.27	1107.27
4	INCOMP.			

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 3.30 Stress (psf)	Sett. (in.)	X = 6.60 Stress (psf)	Sett. (in.)	X = 9.90 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	962.86	3.28	1449.49	4.32	1720.10	4.81	1833.63	5.00
3	958.62	0.04	1300.36	0.05	1551.27	0.06	1698.07	0.07
4	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
		3.33		4.38		4.88		5.07

Layer	X = 13.20 Stress (psf)	Sett. (in.)	X = 16.50 Stress (psf)	Sett. (in.)	X = 19.80 Stress (psf)	Sett. (in.)	X = 23.10 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	1877.52	5.08	1889.43	5.10	1878.96	5.08	1837.69	5.01
3	1769.94	0.07	1791.83	0.07	1772.52	0.07	1704.24	0.07
4	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
		5.14		5.16		5.15		5.08

Sheet 14 of 51
GWT 6-18-07

B~RA

Layer	X = Stress (psf)	26.40 Sett. (in.)	X = Stress (psf)	29.70 Sett. (in.)	X = Stress (psf)	33.00 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.			
2	1730.31	4.83	1472.38	4.37	995.85	3.36
3	1562.91	0.06	1318.69	0.06	980.50	0.04
4	INCOMP.	INCOMP.	INCOMP.			
		------		------		------
		4.89		4.42		3.41

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

sheet 15 of 51
EWT 6-18-07

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name	: US 52 Ramp B	Client	: ODOT9
File Name	: B-RAUndercut	Project Manager	: PN
Date	: 6/12/10	Computed by	: EWT

Settlement for X-Direction

Embank. slope, x direc.	= 0.10 (ft)	Height of fill H	= 32.80 (ft)
y direc.	= 0.10 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 33.00 (ft)	p load/unit area	= 3936.00 (psf)
Embankment bottom width	= 33.20 (ft)	Foundation Elev.	= 544.80 (ft)
Ground Surface Elev.	= 544.80 (ft)		
Water table Elev.	= 537.50 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	4.5	-----	-----	-----	120.00	----	----
2	INCOMP.	4.0	-----	-----	-----	120.00	----	----
3	COMP.	2.0	0.230	0.000	0.000	120.00	2.65	0.61
4	COMP.	1.5	0.018	0.000	0.000	125.00	2.65	1.00
5	INCOMP.	11.9	-----	-----	-----	145.00	----	----

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	INCOMP.			
3	2.00	535.30	1002.72	1002.72
4	1.50	533.55	1107.27	1107.27
5	INCOMP.			

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	3.30 Sett. (in.)	X = Stress (psf)	6.60 Sett. (in.)	X = Stress (psf)	9.90 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
3	961.96	1.00	1360.70	1.28	1626.76	1.44	1764.26	1.51
4	958.62	0.04	1300.36	0.05	1551.27	0.06	1698.07	0.07
5	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
		------		------		------		------
		1.05		1.33		1.50		1.58

Layer	X = Stress (psf)	13.20 Sett. (in.)	X = Stress (psf)	16.50 Sett. (in.)	X = Stress (psf)	19.80 Sett. (in.)	X = Stress (psf)	23.10 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
3	1825.45	1.54	1843.20	1.55	1827.56	1.55	1769.68	1.51
4	1769.94	0.07	1791.83	0.07	1772.52	0.07	1704.24	0.07

B-RAUNDE

5	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
		------		------		------		------
		1.61		1.62		1.61		1.58

	X =	26.40	X =	29.70	X =	33.00
Layer	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.			
2	INCOMP.	INCOMP.	INCOMP.			
3	1638.22	1.44	1381.18	1.29	987.97	1.02
4	1562.91	0.06	1318.69	0.06	980.50	0.04
5	INCOMP.	INCOMP.	INCOMP.			
		------		------		------
		1.50		1.34		1.07

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT TranSystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth ByPass
SUBJECT Drilled shaft End Bearing + Skin Friction US 52 Ramp B

PROJECT NO. 0121-3070-03
SHEET NO. 17 OF 51
COMP. BY GWT DATE 6-15-07
CHECKED BY ________ DATE ________

From Rock core testing, $q_u = 7575$ psi (minimum value)
$= 52.27$ MPa

1) End Bearing Capacity : For RQD = 70 - 100% ; $q_u > 5.2$ tsf (0.5 MPa)

FHWA - IF - 99-025 Eq. 11.6

$$q_{max} = 4.83 [q_u (MPa)]^{0.51}$$
$$= 4.83 (52.27)^{0.51}$$
$$= 36.3 \text{ MPa} = 757 \text{ ksf}$$

$$q_a = \frac{q_{max}}{3} = \frac{757}{3} = 252.5 \text{ ksf}$$

Say $q_a = 80$ ksf. For Design, use $q_a = 40$ ksf*
(Reduction for Argillaceous rock)

2) Side friction (smooth rock socket), Eq. 11.24

$$f_{max} = 0.65 P_a \left[\frac{q_u}{P_a}\right]^{0.5} \leq 0.65 P_a \left[\frac{f'_c}{P_a}\right]^{0.5}$$
$$= 0.65 (14.7) \left(\frac{7575}{14.7}\right)^{0.5} \leq 0.65 (14.7) \left(\frac{4500}{14.7}\right)^{0.5}$$
$$\Rightarrow 216.9 \text{ psi} \leq 167 \text{ psi}$$

$\Rightarrow$ Use $f_{max} = 167$ psi

$$f_{all} = \frac{167}{3} = 55.7 \text{ psi} = 8021 \text{ psf}$$

Say $f_{all} = 7,500$ psf

For Design, use $f_{all} = 3,750$ psf (Reduction for Argillaceous rock)

Factored Lateral Loads for drilled shaft analysis

US 52 Ramp B Pier 2 (most critical)
Right column

Assumes loading from Pier 1 Left (most critical for pier structure)
Horizontal forces from 6 span alternative

AASHTO Load Case	γ	W	WL	LF	T	Loads (kips) W	WL	LF	T	Pier 1 Left Column Factored Load (kips)
I	1.3	0.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	0.00
IB	1.3	0.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	0.00
II	1.3	1.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	41.76
III	1.3	0.3	1.0	1.0	0.0	32.12	9.38	19.43	19.63	49.98
IV	1.3	0.0	0.0	0.0	1.0	32.12	9.38	19.43	19.63	25.52
V	1.25	1.0	0.0	0.0	1.0	32.12	9.38	19.43	19.63	67.28
VI	1.25	0.3	1.0	1.0	1.0	32.12	9.38	19.43	19.63	75.50
VII	1.3	0.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	0.00
VIII	1.3	0.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	0.00
IX	1.2	1.0	0.0	0.0	0.0	32.12	9.38	19.43	19.63	41.76

Vertical Reactions from 5 span alternative (most critical)

Service	(kips)
DL	920.8
LL+I	261.52

Factored Axial Loading
P=1.3[1.67(261.52)+1.0(920.8)] = 1765 kips

Use P = 1765 kips

Sheet 18 of 51
GWT 6-18-07

Sheet 19 of 51
GWT 6-18-07

Vertical rxns (kips)

	Pier 2 Left column	Pier 2 RIght column	Pier 3 Left column	Pier 3 Right column
DL	494.68	726.08	706.69	664.15
LL+I				

Horizontal (in longitudinal direction of bridge)) rxns (kips)

	Pier 2 Left column	Pier 2 RIght column	Pier 3 Left column	Pier 3 Right column
Long. Force	13.235	19.43	16.84	15.82
WL (wind on LL)	6.39	9.38	8.13	7.64
Wind (W)	21.88	32.12	27.84	26.16
Thermal (T)	13.37	19.63	15.82	14.87
Service Level summation =	54.875	80.56	68.63	64.49

Vertical rxns (kips)

	Pier 1 Left column	Pier 1 Right column	Pier 2 Left column	Pier 2 Right column
DL	677.92	920.8	665.12	624.52
LL+I	160.48	261.52		
Service Level summation =	838.4	**1182.32**		

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

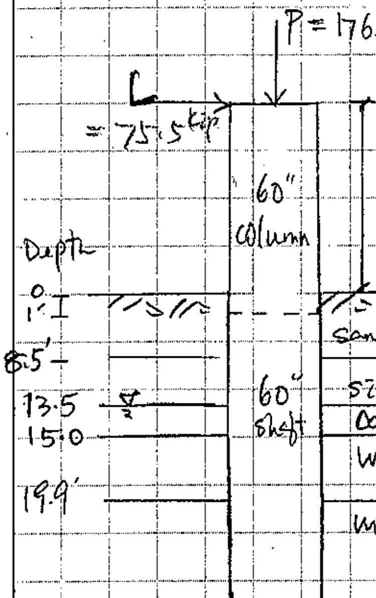
CLIENT Transystems Corp/ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth ByPass SHEET NO. 21 OF 51
SUBJECT US 52 Ramp B – Pier 1 COMP. BY EWT DATE 6-18-07
Laterally Loaded Drilled Shaft CHECKED BY SJR/DAA DATE 6-18-07

* Loads were provided by Transystems based on preliminary designs → Service Loads only
Factored lateral load = 75.5 kips. AASHTO Case VI loading
Assumed that lateral loading applied at elevation 571 ft, 22' above ground surface @ B-49 (Pier 1)
Profile encountered in Boring B-49 deemed to be more critical when comparing with B-48. Consequently, B-49 was used for lateral load analyses.

Factored axial load from Case I

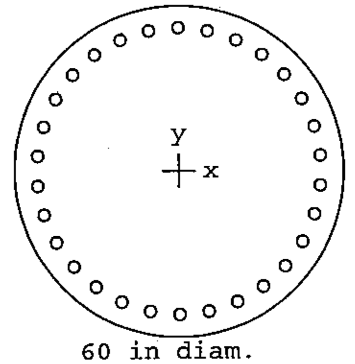
$$P = 1.3\,[1.67 \times 261.52^{k} + (1.0)(920.8)] = 1765 \text{ kips}$$

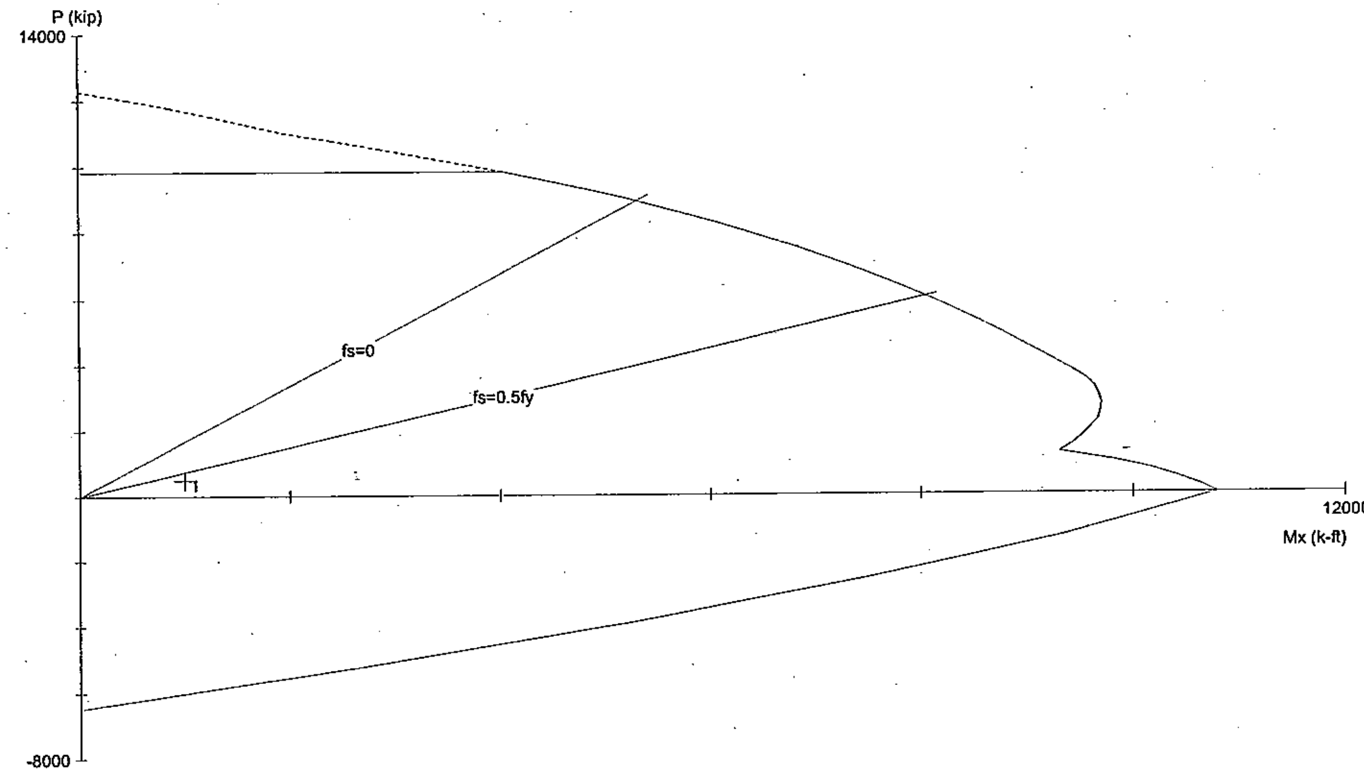
For analyses: (1) Use 60" column and 60" drilled shaft
(2) Assume column extends 1' below ground surface at Boring B-49
(3) Neglect upper 5' of soil for lateral capacity
(4) Column (60" diam), 30-#18, $\rho = 4.24\%$
$f_y = 60$ ksi, $f'_c = 4.5$ ksi, $E_s = 29 \times 10^6$ psi
$E_c = 3.824 \times 10^6$ psi
(5) Drilled Shaft (60" diam), 30-#18 $\rho = 4.24\%$
$f_y = 60$ ksi
$f'_c = 4.5$ ksi
$E_s = 29 \times 10^6$ psi
$E_c = 3.824 \times 10^6$ psi



Sandy silt/silt → Assume c = 2500 psf $\gamma = 0.072$ pci
Silt → Assume c = 250 psf $\gamma = 0.0666$ pci
Coarse + fine sand → $\phi = 32°$, $\gamma = 0.036$ pci
Weathered SS. → $q_u = 750$ psi, $\gamma = 0.081$ pci
Med hard SS → $q_u = 5969$ psi $= \frac{11938}{2}$ (lower q_u in the boring)

Results: (1) 20 ft rock socket should be provided to resist lateral loading, equivalent to Elevation 514.6
(2) Pile head deflection (at pier cap) = 0.95 inches
(3) From Lpile analysis (using factored loads)
$V_{max} = 348$ kips
$M_{max} = 2547$ k-ft





Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/15/07

PCACOL V3.00 (PCA 1999) - Licensed to: Licensee name not yet specified.

File: C:\DOCUME~1\DLZ\DESKTOP\PORTSM~1\US52RA~1\PIER1.COL

Project: US 52 Ramp B

Column: Pier 1		Engineer: EWT	
f'c = 4.5 ksi	fy = 60 ksi	Ag = 2827.43 in^2	30 #18 bars
Ec = 3824 ksi	Es = 29000 ksi	As = 120.00 in^2	Rho = 4.24%
fc = 3.825 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 636173 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 636173 in^4
Beta1 = 0.825		Clear spacing = 3.15 in	Clear cover = 3.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7		

Sheet 22 of 51
EWT 6-18-07

Sheet 25 of 51
GWT 6-18-07

===

LPILE Plus for Windows, Version 5.0 (5.0.7)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This program is licensed to:

E. Tse
DLZ

```
Path to file locations:       C:\Documents and
Settings\DLZ\Desktop\Portsmouth-US52\US 52 Ramp B Bridge\
Name of input data file:      Ramp B Lat Analysis - Pier 1.lpd
Name of output file:          Ramp B Lat Analysis - Pier 1.lpo
Name of plot output file:     Ramp B Lat Analysis - Pier 1.lpp
Name of runtime file:         Ramp B Lat Analysis - Pier 1.lpr
```

Time and Date of Analysis

Date: June 18, 2007   Time: 17:44:26

Problem Title

SCI-823 US 52 Ramp B - Pier 1

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment
  Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:

Ramp B Lat Analysis - Pier 1.lpo

- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 684.00 in
Depth of ground surface below top of pile = 324.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	60.00000000	636173.0000	2827.0000	3823700.
2	276.0000	60.00000000	636173.0000	2827.0000	3823700.
3	276.0000	60.00000000	636173.0000	2827.0000	3823700.
4	660.0000	60.00000000	636173.0000	2827.0000	3823700.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 324.000 in
Distance from top of pile to bottom of layer = 366.000 in

Layer 2 is soft clay, p-y criteria by Matlock, 1970
Distance from top of pile to top of layer = 366.000 in
Distance from top of pile to bottom of layer = 426.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 426.000 in
Distance from top of pile to bottom of layer = 444.000 in
p-y subgrade modulus k for top of soil layer = .000 lbs/in**3
p-y subgrade modulus k for bottom of layer = .000 lbs/in**3

NOTE: Internal default values for p-y subgrade modulus will be computed for the above soil layer.

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 444.000 in
Distance from top of pile to bottom of layer = 502.800 in

Ramp B Lat Analysis - Pier 1.lpo

Layer 5 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 502.800 in
Distance from top of pile to bottom of layer = 1000.000 in

(Depth of lowest layer extends 316.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	324.00	.07200
2	366.00	.07200
3	366.00	.06660
4	426.00	.06660
5	426.00	.03600
6	444.00	.03600
7	444.00	.04500
8	502.80	.04500
9	502.80	.04500
10	1000.00	.04500

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	324.000	17.36000	.00	------	------
2	366.000	17.36000	.00	------	------
3	366.000	1.73600	.00	------	------
4	426.000	1.73600	.00	------	------
5	426.000	.00000	32.00	------	------
6	444.000	.00000	32.00	------	------
7	444.000	750.00000	.00	------	------
8	502.800	750.00000	.00	------	------
9	502.800	5969.00000	.00	------	------
10	1000.000	5969.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Sheet 20 of 71
GWT 6-18-07

Loading Type

Static loading criteria was used for computation of p-y curves

---

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 75500.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 1765000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

---

Number of pile sections = 2

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 60.0000 In

Material Properties:

Compressive Strength of Concrete = 4.500 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2
Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 30
Area of Single Bar = 4.00000 In**2
Number of Rows of Reinforcing Bars = 15
Cover Thickness (edge to bar center) = 3.000 In

Unfactored Axial Squash Load Capacity = 17555.93 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	8.000000	26.8521
2	8.000000	25.6785
3	8.000000	23.3827
4	8.000000	20.0649
5	8.000000	15.8702
6	8.000000	10.9819

Sheet 27 of 51
RWT 6-18-07

7	8.000000	5.6136
8	8.000000	.0000
9	8.000000	-5.6136
10	8.000000	-10.9819
11	8.000000	-15.8702
12	8.000000	-20.0649
13	8.000000	-23.3827
14	8.000000	-25.6785
15	8.000000	-26.8521

Axial Thrust Force = 1765000.00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
3572244.	3.572244E+12	.00000100	.00015615	156.14880
17852002.	3.570400E+12	.00000500	.00027756	55.51242828
28558936.	3.173215E+12	.00000900	.00038873	43.19171906
37222649.	2.863281E+12	.00001300	.00049108	37.77523041
45148204.	2.655777E+12	.00001700	.00058920	34.65900421
52738792.	2.511371E+12	.00002100	.00068528	32.63248444
60125152.	2.405006E+12	.00002500	.00078019	31.20769501
67382071.	2.323520E+12	.00002900	.00087471	30.16239166
74534149.	2.258611E+12	.00003300	.00096913	29.36771393
81599955.	2.205404E+12	.00003700	.00106383	28.75225067
88561809.	2.160044E+12	.00004100	.00115822	28.24916840
95465821.	2.121463E+12	.00004500	.00125371	27.86029816
1.022693E+08	2.087129E+12	.00004900	.00134883	27.52704620
1.089987E+08	2.056579E+12	.00005300	.00144458	27.25627899
1.156514E+08	2.028972E+12	.00005700	.00154101	27.03517914
1.222359E+08	2.003866E+12	.00006100	.00163888	26.86695099
1.287234E+08	1.980359E+12	.00006500	.00173643	26.71428680
1.349790E+08	1.956218E+12	.00006900	.00183398	26.57947540
1.397836E+08	1.914844E+12	.00007300	.00192523	26.37302399
1.438543E+08	1.868238E+12	.00007700	.00201343	26.14849091
1.471772E+08	1.817002E+12	.00008100	.00209949	25.91960907
1.503835E+08	1.769218E+12	.00008500	.00218385	25.69232941
1.527595E+08	1.716399E+12	.00008900	.00226331	25.43048859
1.551070E+08	1.667817E+12	.00009300	.00234328	25.19657135
1.572240E+08	1.620866E+12	.00009700	.00242631	25.01346588
1.589314E+08	1.573578E+12	.00010100	.00251006	24.85210419
1.655556E+08	1.263783E+12	.00013100	.00314400	23.99997711
1.684576E+08	1.046321E+12	.00016100	.00379144	23.54930878
1.684576E+08	8.819771E+11	.00019100	.00457499	23.95282745

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 164050.92276 In-Kip

Pile Section No. 2

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 60.0000 In

Material Properties:

Compressive Strength of Concrete = 4.500 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2

sheet 28 of 51
SWT 6-18-07

Ramp B Lat Analysis - Pier 1.lpo

Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 30
Area of Single Bar = 4.00000 In**2
Number of Rows of Reinforcing Bars = 15
Cover Thickness (edge to bar center) = 3.000 In

Unfactored Axial Squash Load Capacity = 17555.93 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	8.000000	26.8521
2	8.000000	25.6785
3	8.000000	23.3827
4	8.000000	20.0649
5	8.000000	15.8702
6	8.000000	10.9819
7	8.000000	5.6136
8	8.000000	.0000
9	8.000000	-5.6136
10	8.000000	-10.9819
11	8.000000	-15.8702
12	8.000000	-20.0649
13	8.000000	-23.3827
14	8.000000	-25.6785
15	8.000000	-26.8521

Axial Thrust Force = 1765000.00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
3572244.	3.572244E+12	.00000100	.00015615	156.14880
17852002.	3.570400E+12	.00000500	.00027756	55.51242828
28558936.	3.173215E+12	.00000900	.00038873	43.19171906
37222649.	2.863281E+12	.00001300	.00049108	37.77523041
45148204.	2.655777E+12	.00001700	.00058920	34.65900421
52738792.	2.511371E+12	.00002100	.00068528	32.63248444
60125152.	2.405006E+12	.00002500	.00078019	31.20769501
67382071.	2.323520E+12	.00002900	.00087471	30.16239166
74534149.	2.258611E+12	.00003300	.00096913	29.36771393
81599955.	2.205404E+12	.00003700	.00106383	28.75225067
88561809.	2.160044E+12	.00004100	.00115822	28.24916840
95465821.	2.121463E+12	.00004500	.00125371	27.86029816
1.022693E+08	2.087129E+12	.00004900	.00134883	27.52704620
1.089987E+08	2.056579E+12	.00005300	.00144458	27.25627899
1.156514E+08	2.028972E+12	.00005700	.00154101	27.03517914
1.222359E+08	2.003866E+12	.00006100	.00163888	26.86695099
1.287234E+08	1.980359E+12	.00006500	.00173643	26.71428680
1.349790E+08	1.956218E+12	.00006900	.00183398	26.57947540
1.397836E+08	1.914844E+12	.00007300	.00192523	26.37302399
1.438543E+08	1.868238E+12	.00007700	.00201343	26.14849091
1.471772E+08	1.817002E+12	.00008100	.00209949	25.91960907
1.503835E+08	1.769218E+12	.00008500	.00218385	25.69232941
1.527595E+08	1.716399E+12	.00008900	.00226331	25.43048859
1.551070E+08	1.667817E+12	.00009300	.00234328	25.19657135
1.572240E+08	1.620866E+12	.00009700	.00242631	25.01346588
1.589314E+08	1.573578E+12	.00010100	.00251006	24.85210419

Sheet 29 of 51
EWT 6-18-07

1.655556E+08	1.263783E+12	.00013100	.00314400	23.99997711
1.684576E+08	1.046321E+12	.00016100	.00379144	23.54930878
1.684576E+08	8.819771E+11	.00019100	.00457499	23.95282745

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 164050.92276 In-Kip

---

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 75500.000 lbs
Specified moment at pile head = .000 in-lbs
Specified axial load at pile head = 1765000.000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	.953181	-.000203	75500.	-.002867	624.337	3.57E+12	0.000
6.840	.933571	5.51E+05	75500.	-.002866	650.322	3.57E+12	0.000
13.680	.913969	1.10E+06	75500.	-.002865	676.306	3.57E+12	0.000
20.520	.894382	1.65E+06	75500.	-.002862	702.289	3.57E+12	0.000
27.360	.874815	2.20E+06	75500.	-.002858	728.271	3.57E+12	0.000
34.200	.855278	2.75E+06	75500.	-.002854	754.249	3.57E+12	0.000
41.040	.835777	3.31E+06	75500.	-.002848	780.225	3.57E+12	0.000
47.880	.816319	3.86E+06	75500.	-.002841	806.198	3.57E+12	0.000
54.720	.796912	4.41E+06	75500.	-.002833	832.166	3.57E+12	0.000
61.560	.777562	4.96E+06	75500.	-.002824	858.129	3.57E+12	0.000
68.400	.758278	5.51E+06	75500.	-.002814	884.087	3.57E+12	0.000
75.240	.739065	6.06E+06	75500.	-.002803	910.039	3.57E+12	0.000
82.080	.719932	6.61E+06	75500.	-.002791	935.984	3.57E+12	0.000
88.920	.700885	7.16E+06	75500.	-.002778	961.922	3.57E+12	0.000
95.760	.681933	7.71E+06	75500.	-.002763	987.853	3.57E+12	0.000
102.600	.663081	8.26E+06	75500.	-.002748	1013.775	3.57E+12	0.000
109.440	.644337	8.81E+06	75500.	-.002732	1039.687	3.57E+12	0.000
116.280	.625709	9.36E+06	75500.	-.002714	1065.591	3.57E+12	0.000
123.120	.607204	9.91E+06	75500.	-.002696	1091.484	3.57E+12	0.000
129.960	.588828	1.05E+07	75500.	-.002677	1117.366	3.57E+12	0.000
136.800	.570589	1.10E+07	75500.	-.002656	1143.237	3.57E+12	0.000
143.640	.552494	1.16E+07	75500.	-.002634	1169.096	3.57E+12	0.000
150.480	.534551	1.21E+07	75500.	-.002612	1194.942	3.57E+12	0.000
157.320	.516766	1.26E+07	75500.	-.002588	1220.775	3.57E+12	0.000
164.160	.499147	1.32E+07	75500.	-.002563	1246.594	3.57E+12	0.000
171.000	.481701	1.37E+07	75500.	-.002537	1272.399	3.57E+12	0.000
177.840	.464435	1.43E+07	75500.	-.002511	1298.189	3.57E+12	0.000
184.680	.447357	1.48E+07	75500.	-.002483	1323.963	3.57E+12	0.000
191.520	.430472	1.54E+07	75500.	-.002454	1349.722	3.57E+12	0.000
198.360	.413789	1.59E+07	75500.	-.002424	1375.463	3.57E+12	0.000
205.200	.397315	1.65E+07	75500.	-.002393	1401.187	3.57E+12	0.000
212.040	.381057	1.70E+07	75500.	-.002361	1426.893	3.57E+12	0.000
218.880	.365022	1.76E+07	75500.	-.002328	1452.580	3.57E+12	0.000
225.720	.349217	1.81E+07	75500.	-.002293	1478.249	3.55E+12	0.000
232.560	.333650	1.87E+07	75500.	-.002258	1503.897	3.52E+12	0.000
239.400	.318331	1.92E+07	75500.	-.002221	1529.525	3.49E+12	0.000
246.240	.303270	1.97E+07	75500.	-.002182	1555.131	3.46E+12	0.000

sheet 30 of 51
QWT 6-18-07

253.080	.288476	2.03E+07	75500.	-.002143	1580.716	3.43E+12	0.000
259.920	.273958	2.08E+07	75500.	-.002102	1606.277	3.41E+12	0.000
266.760	.259725	2.14E+07	75500.	-.002059	1631.814	3.38E+12	0.000
273.600	.245788	2.19E+07	75500.	-.002015	1657.327	3.36E+12	0.000
280.440	.232156	2.24E+07	75500.	-.001970	1682.814	3.34E+12	0.000
287.280	.218838	2.30E+07	75500.	-.001923	1708.276	3.32E+12	0.000
294.120	.205844	2.35E+07	75500.	-.001875	1733.710	3.30E+12	0.000
300.960	.193183	2.41E+07	75500.	-.001826	1759.117	3.29E+12	0.000
307.800	.180864	2.46E+07	75500.	-.001775	1784.495	3.27E+12	0.000
314.640	.168898	2.51E+07	75500.	-.001723	1809.844	3.26E+12	0.000
321.480	.157292	2.57E+07	75500.	-.001670	1835.162	3.24E+12	0.000
328.320	.146057	2.62E+07	71887.	-.001615	1860.450	3.23E+12	-1056.565
335.160	.135203	2.67E+07	64630.	-.001559	1883.375	3.21E+12	-1065.333
342.000	.124737	2.71E+07	57318.	-.001501	1903.918	3.20E+12	-1072.481
348.840	.114667	2.75E+07	49964.	-.001443	1922.061	3.20E+12	-1077.949
355.680	.105000	2.79E+07	42578.	-.001383	1937.793	3.19E+12	-1081.670
362.520	.095741	2.81E+07	35173.	-.001323	1951.104	3.18E+12	-1083.570
369.360	.086897	2.84E+07	31180.	-.001263	1961.990	3.18E+12	-83.952
376.200	.078470	2.86E+07	30598.	-.001201	1972.656	3.17E+12	-86.083
383.040	.070464	2.88E+07	30004.	-.001139	1983.097	3.16E+12	-87.814
389.880	.062886	2.90E+07	29399.	-.001076	1993.308	3.15E+12	-89.133
396.720	.055738	2.92E+07	28786.	-.001013	2003.287	3.14E+12	-90.025
403.560	.049027	2.94E+07	28168.	-.000949	2013.032	3.13E+12	-90.477
410.400	.042756	2.96E+07	27550.	-.000884	2022.540	3.12E+12	-90.476
417.240	.036929	2.98E+07	26932.	-.000819	2031.811	3.11E+12	-90.004
424.080	.031551	3.00E+07	26320.	-.000753	2040.847	3.10E+12	-89.049
430.920	.026626	3.02E+07	25250.	-.000687	2049.648	3.09E+12	-223.741
437.760	.022159	3.04E+07	23802.	-.000619	2057.918	3.09E+12	-199.840
444.600	.018152	3.06E+07	-23442.	-.000552	2065.708	3.08E+12	-13614.
451.440	.014610	3.01E+07	-1.07E+05	-.000485	2043.423	3.10E+12	-10957.
458.280	.011521	2.91E+07	-1.75E+05	-.000420	1996.927	3.15E+12	-8640.972
465.120	.008866	2.77E+07	-2.27E+05	-.000359	1931.330	3.19E+12	-6649.474
471.960	.006617	2.60E+07	-2.67E+05	-.000301	1851.029	3.23E+12	-4962.726
478.800	.004745	2.41E+07	-2.96E+05	-.000249	1759.747	3.29E+12	-3558.406
485.640	.003215	2.20E+07	-3.16E+05	-.000201	1660.586	3.36E+12	-2411.138
492.480	.001991	1.98E+07	-3.29E+05	-.000159	1556.080	3.46E+12	-1493.356
499.320	.001035	1.75E+07	-3.37E+05	-.000123	1448.257	3.57E+12	-776.011
506.160	.000307	1.51E+07	-3.46E+05	-9.19E-05	1338.703	3.57E+12	-1833.477
513.000	-.000222	1.27E+07	-3.48E+05	-6.51E-05	1225.087	3.57E+12	1324.213
519.840	-.000584	1.04E+07	-3.31E+05	-4.30E-05	1114.379	3.57E+12	3485.529
526.680	-.000810	8.21E+06	-3.03E+05	-2.52E-05	1011.349	3.57E+12	4834.120
533.520	-.000928	6.25E+06	-2.67E+05	-1.13E-05	918.976	3.57E+12	5540.896
540.360	-.000965	4.55E+06	-2.29E+05	-9.89E-07	838.821	3.57E+12	5759.090
547.200	-.000942	3.12E+06	-1.90E+05	6.35E-06	771.367	3.57E+12	5621.668
554.040	-.000878	1.95E+06	-1.53E+05	1.12E-05	716.312	3.57E+12	5240.503
560.880	-.000789	1.03E+06	-1.19E+05	1.41E-05	672.817	3.57E+12	4706.863
567.720	-.000686	3.26E+05	-88629.	1.54E-05	639.706	3.57E+12	4092.853
574.560	-.000579	-1.85E+05	-62821.	1.55E-05	633.049	3.57E+12	3453.365
581.400	-.000474	-5.34E+05	-41338.	1.48E-05	649.512	3.57E+12	2828.319
588.240	-.000376	-7.51E+05	-23987.	1.36E-05	659.733	3.57E+12	2245.008
595.080	-.000288	-8.62E+05	-10426.	1.20E-05	665.001	3.57E+12	1720.377
601.920	-.000212	-8.94E+05	-221.897	1.03E-05	666.473	3.57E+12	1263.159
608.760	-.000147	-8.66E+05	7093.319	8.66E-06	665.156	3.57E+12	875.793
615.600	-9.32E-05	-7.97E+05	11990.	7.07E-06	661.907	3.57E+12	556.097
622.440	-5.00E-05	-7.02E+05	14914.	5.63E-06	657.429	3.57E+12	298.684
629.280	-1.61E-05	-5.93E+05	16264.	4.39E-06	652.292	3.57E+12	96.131
636.120	1.01E-05	-4.79E+05	16387.	3.37E-06	646.942	3.57E+12	-60.078
642.960	3.00E-05	-3.69E+05	15570.	2.56E-06	641.724	3.57E+12	-178.813
649.800	4.50E-05	-2.66E+05	14040.	1.95E-06	636.901	3.57E+12	-268.722
656.640	5.66E-05	-1.77E+05	11965.	1.52E-06	632.670	3.57E+12	-337.803
663.480	6.59E-05	-1.03E+05	9465.817	1.26E-06	629.183	3.57E+12	-393.071
670.320	7.38E-05	-47240.	6615.679	1.11E-06	626.564	3.57E+12	-440.303
677.160	8.11E-05	-12302.	3455.100	1.05E-06	624.917	3.57E+12	-483.843

Sheet 31 of 51
GWT 6-18-07

684.000	8.82E-05	0.000	0.000	1.04E-06	624.337	3.57E+12	-526.421

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =      .95318080 in
Computed slope at pile head       =     -.00286687
Maximum bending moment            =      30565379. lbs-in
Maximum shear force               =  -347846.28364 lbs
Depth of maximum bending moment   =      444.60000 in
Depth of maximum shear force      =      513.00000 in
Number of iterations              =             11
Number of zero deflection points  =              2
```

Summary of Pile-Head Response(s)

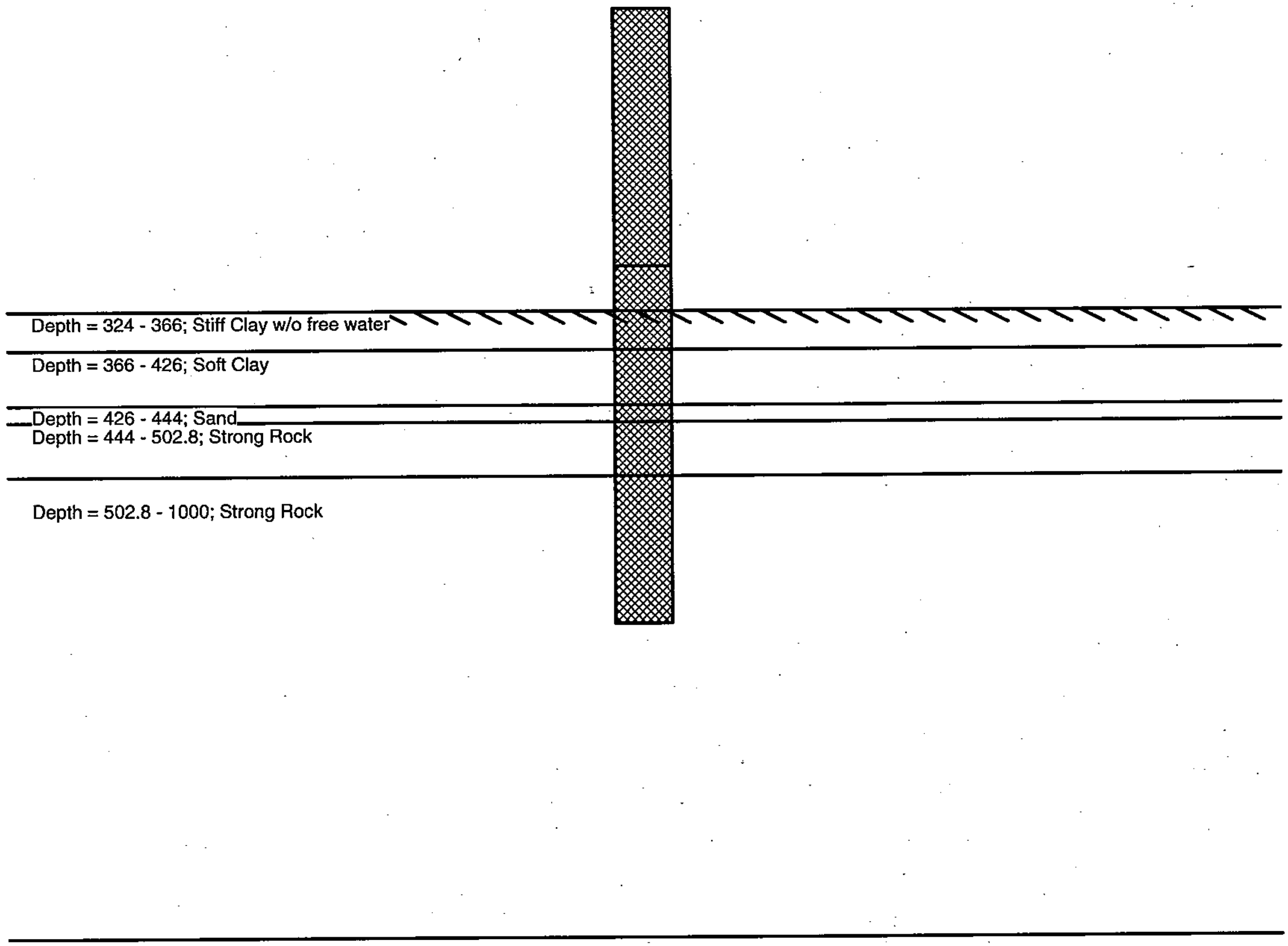
Definition of Symbols for Pile-Head Loading Conditions:

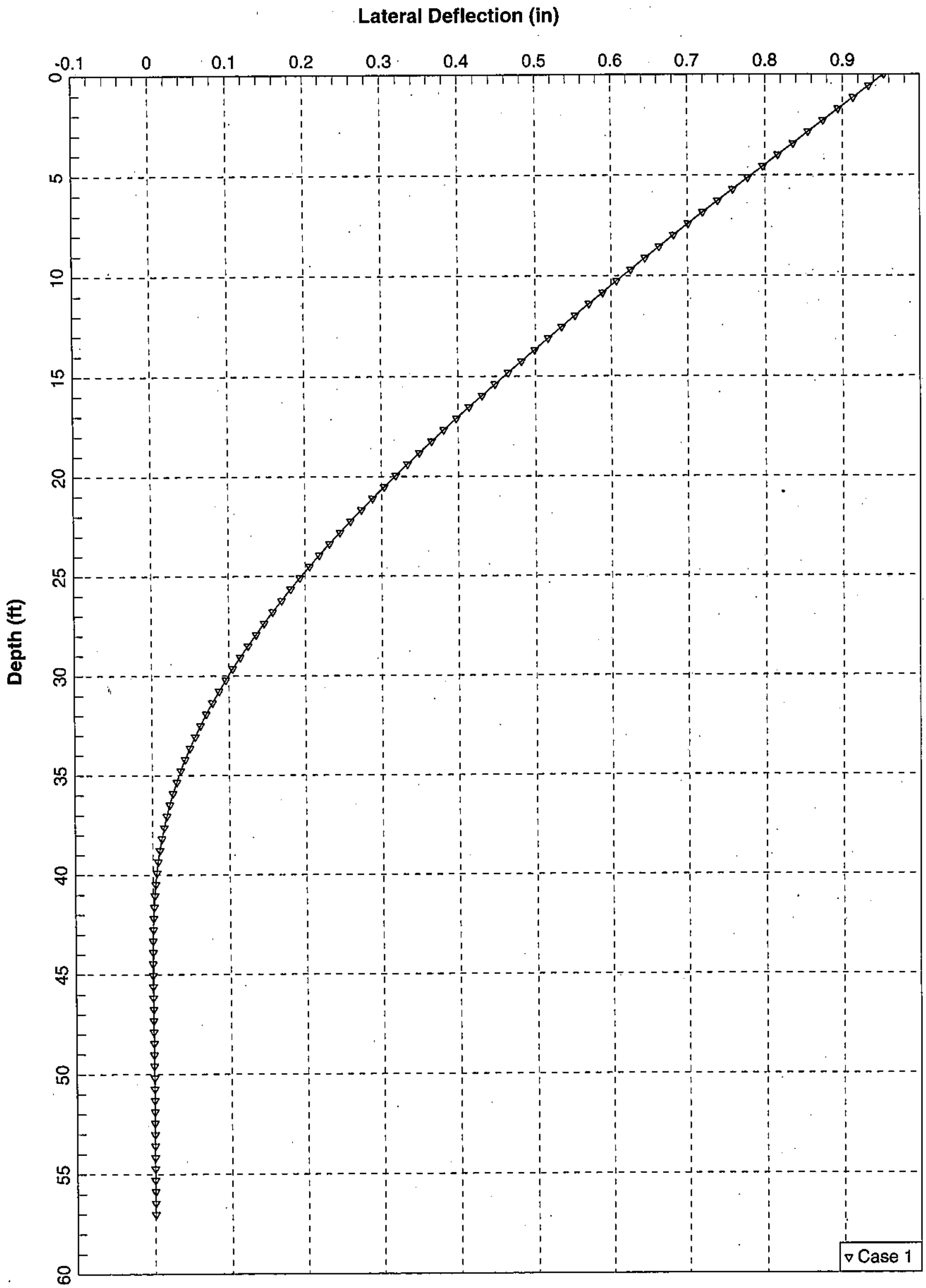
```
Type 1 = Shear and Moment,          y = pile-head displacment in
Type 2 = Shear and Slope,           M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness,  V = pile-head shear force lbs
Type 4 = Deflection and Moment,     S = pile-head slope, radians
Type 5 = Deflection and Slope,      R = rotational stiffness of pile-headin-lbs/rad
```

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 75500.	M= 0.000	1765000.	.9531808	3.0565E+07	-347846.

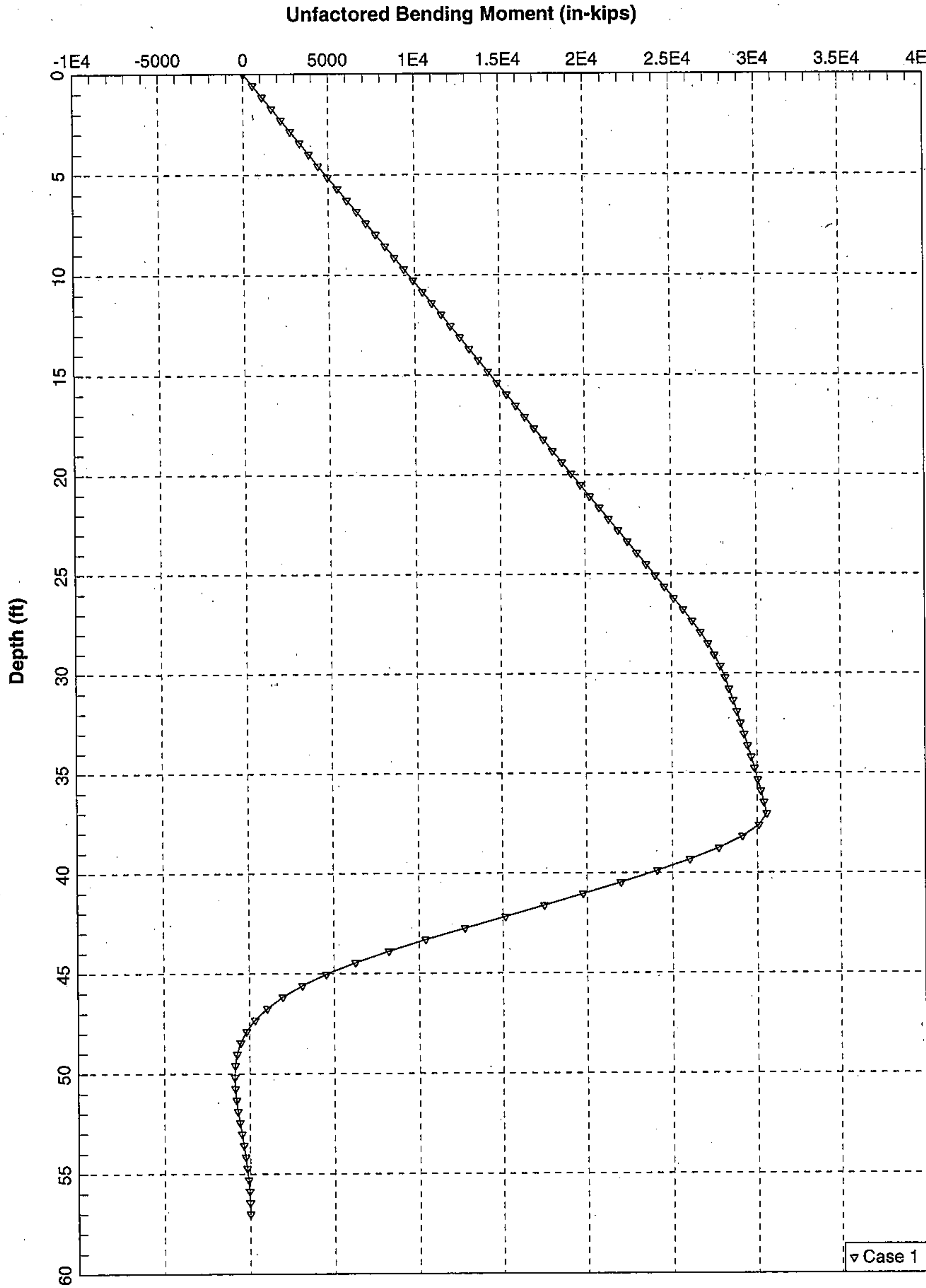
The analysis ended normally.

sheet 32 of 51
GWT 6-18-07

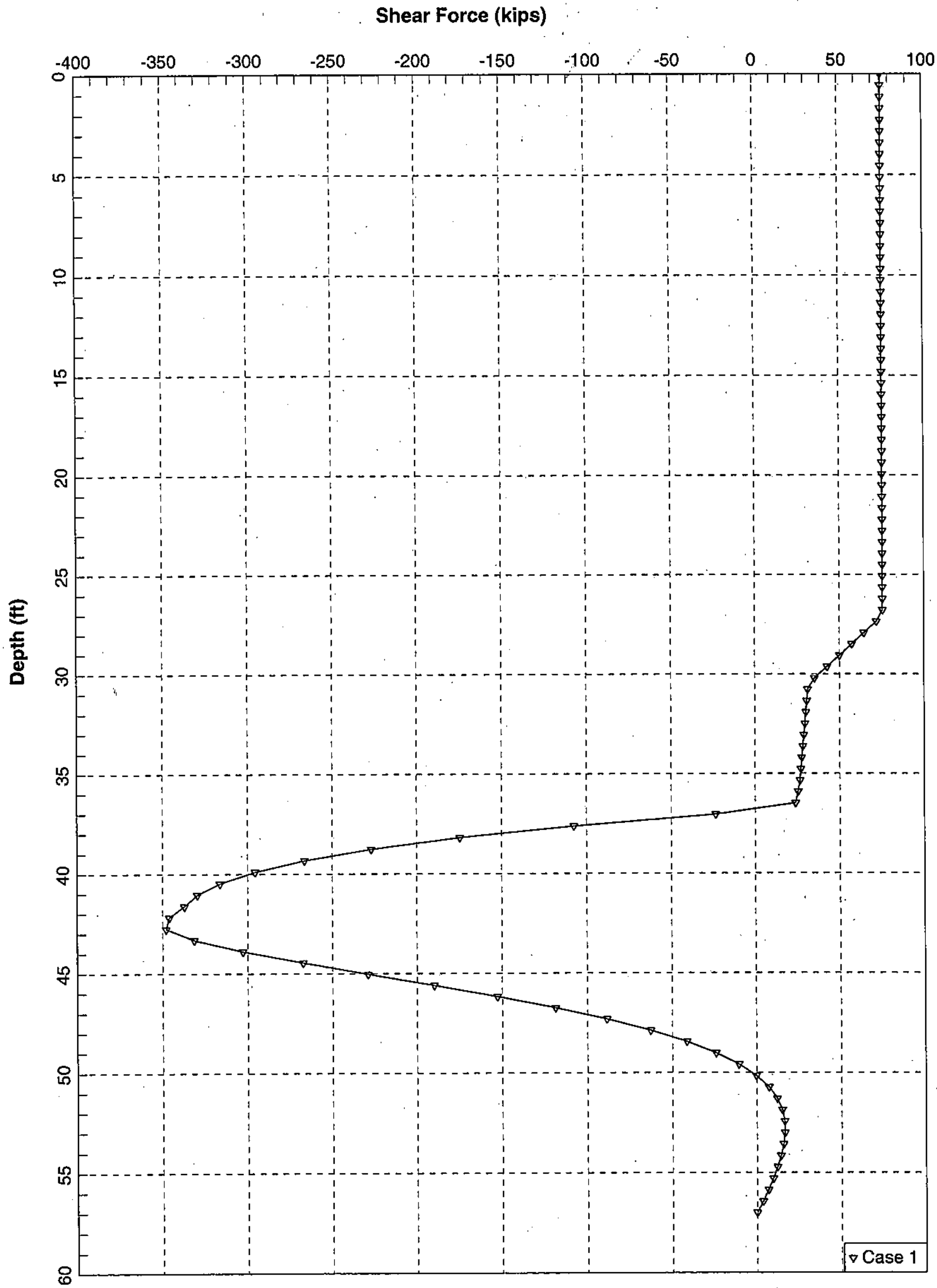



Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Case 1

sheet 34 of 51
EWT 6-18-07



sheet 35 of 51
EWZ 6-18-07



DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

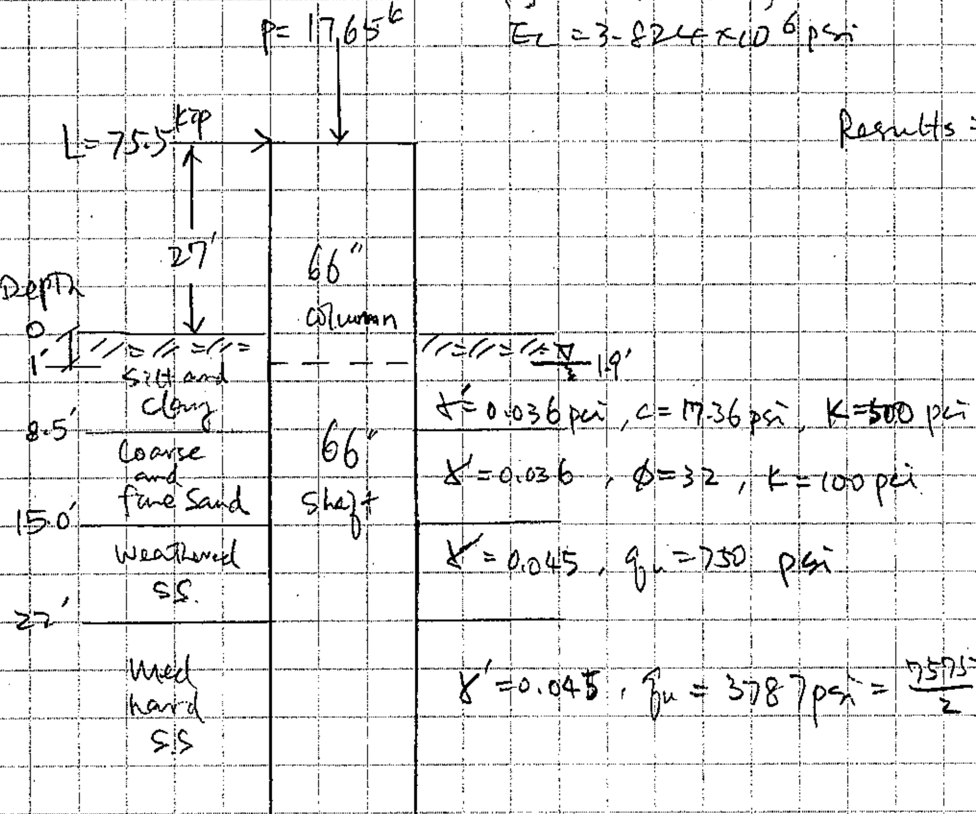
CLIENT Transystem Corp / ODOT D-9.
PROJECT SCI-823 Portsmouth ByPass
SUBJECT US 52 Ramp B - Pier 2
Laterally Loaded Drilled Shaft

PROJECT NO. 0121-3070.03
SHEET NO. 36 OF 51
COMP. BY SWT DATE 6-18-07
CHECKED BY SJR/DAA DATE 6-18-07

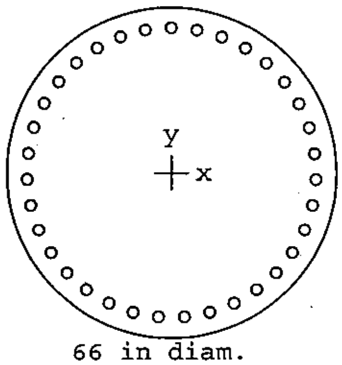
Using the same loading for Pier 1
Profile encountered in Boring B-50 deemed to be more critical
Comparing with B-51, As a result, use B-50 for analyses.

Apply Loads P = 1765 kips and lat load = 75.5 kips
at elevation 577, 27' above ground surface (Pier 2) of B-50

For analyses : (1) Use 66" column and 66" drilled shaft
(2) Assume column extends 1' below ground surface at B-50.
(3) Neglect upper 5' for capacity
(4) Column (66" diam), 35 - #18, $\rho = 4.09\%$
$f_y = 60$ ksi $f'_c = 4.5$ ksi, $E_s = 29 \times 10^6$ psi
$E_c = 3.824 \times 10^6$ psi
(5) Drilled shaft (66" diam), 35 - #18, $\rho = 4.09\%$
$f_y = 60$ ksi, $f'_c = 4.5$ ksi, $E_s = 29 \times 10^6$ psi
$E_c = 3.824 \times 10^6$ psi



Results : (1) 20 ft rock socket to resist lateral loads, equivalent to Elevation 515.2
(2) Pile Load deflection (at pier cap) = 0.96 inches
(3) From LPile analysis (using factored loads)
$V_{max} = 348$ kips
$M_{max} = 2959$ ft-kip



Code: ACI 318-95

Units: English

Run axis: About X-axis

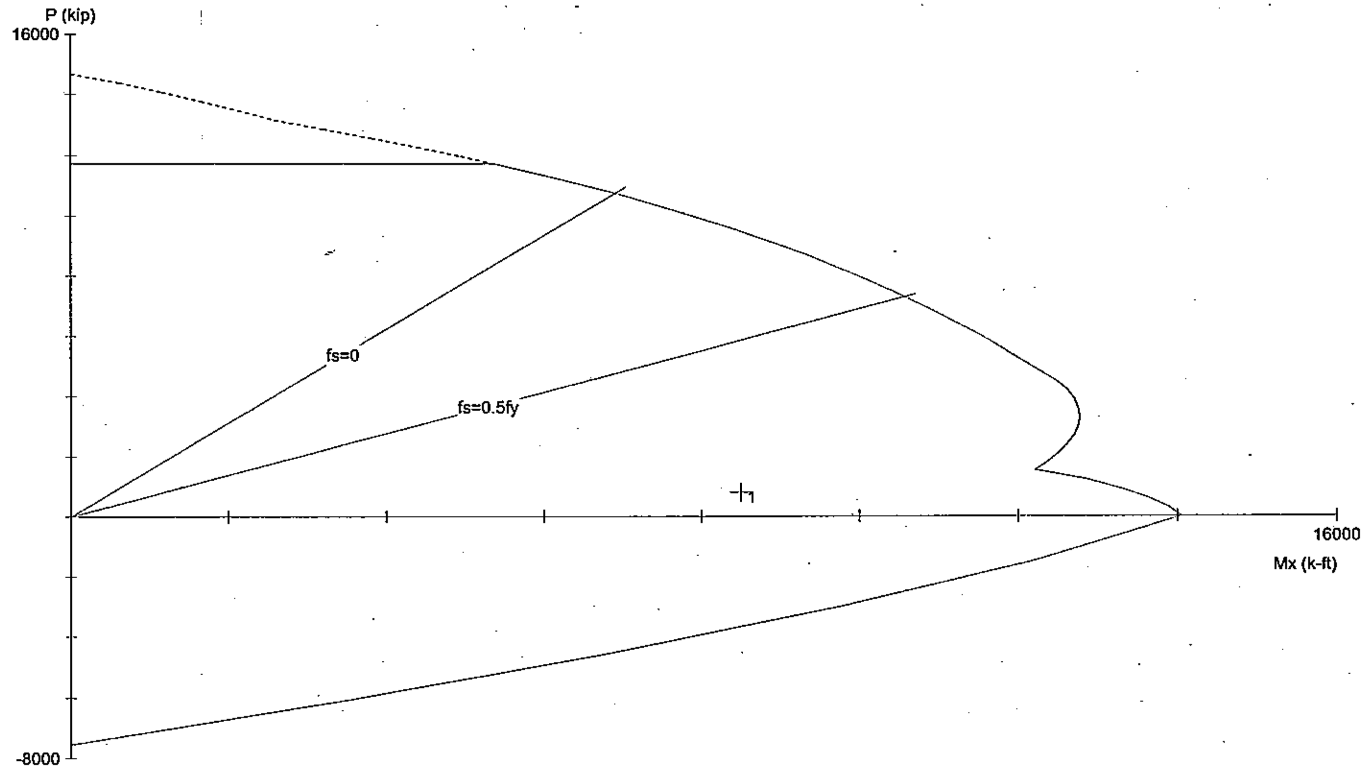
Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 06/15/07



PCACOL V3.00 (PCA 1999) - Licensed to: Licensee name not yet specified.

File: C:\DOCUME~1\DLZ\DESKTOP\PORTSM~1\US52RA~1\PIER2.COL

Project: US 52 Ramp B

Column: Pier 2		Engineer: EWT	
f'c = 4.5 ksi	fy = 60 ksi	Ag = 3421.19 in^2	35 #18 bars
Ec = 3824 ksi	Es = 29000 ksi	As = 140.00 in^2	Rho = 4.09%
fc = 3.825 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 931420 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 931420 in^4
Beta1 = 0.825		Clear spacing = 2.92 in	Clear cover = 3.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7		

sheet 37 of 51
EWT 6-18-07
SJR/DAA 6-18-07

Sheet 38 of 51
EWT 6-18-07

===

LPILE Plus for Windows, Version 5.0 (5.0.7)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This program is licensed to:

E. Tse
DLZ

Path to file locations: C:\Documents and Settings\DLZ\Desktop\Portsmouth-US52\US 52 Ramp B Bridge\
Name of input data file: Ramp B Lat Analysis - Pier 2.lpd
Name of output file: Ramp B Lat Analysis - Pier 2.lpo
Name of plot output file: Ramp B Lat Analysis - Pier 2.lpp
Name of runtime file: Ramp B Lat Analysis - Pier 2.lpr

Time and Date of Analysis

Date: June 18, 2007 Time: 17:59:10

Problem Title

SCI-823 US 52 Ramp B - Pier 2

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment
  Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:

Ramp B Lat Analysis - Pier 2.lpo

- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 744.00 in
Depth of ground surface below top of pile = 384.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 4 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	66.00000000	931420.0000	3421.0000	3823700.
2	336.0000	66.00000000	931420.0000	3421.0000	3823700.
3	336.0000	66.00000000	931420.0000	3421.0000	3823700.
4	1000.0000	66.00000000	931420.0000	3421.0000	3823700.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 384.000 in
Distance from top of pile to bottom of layer = 426.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 426.000 in
Distance from top of pile to bottom of layer = 504.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 504.000 in
Distance from top of pile to bottom of layer = 588.000 in

Layer 4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 588.000 in
Distance from top of pile to bottom of layer = 1000.000 in

Sheet 40 of 51
SWT 6-8-07

Ramp B Lat Analysis - Pier 2.lpo

(Depth of lowest layer extends 256.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	384.00	.03600
2	426.00	.03600
3	426.00	.03600
4	504.00	.03600
5	504.00	.04500
6	588.00	.04500
7	588.00	.04500
8	1000.00	.04500

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	384.000	17.36000	.00	------	------
2	426.000	17.36000	.00	------	------
3	426.000	.00000	32.00	------	------
4	504.000	.00000	32.00	------	------
5	504.000	750.00000	.00	------	------
6	588.000	750.00000	.00	------	------
7	588.000	3787.00000	.00	------	------
8	1000.000	3787.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

Sheet 41 of 51
GWT 6-18-07

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 75500.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 1765000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

---

Number of pile sections = 2

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 66.0000 In

Material Properties:

Compressive Strength of Concrete = 4.500 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2
Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 35
Area of Single Bar = 4.00000 In**2
Number of Rows of Reinforcing Bars = 35
Cover Thickness (edge to bar center) = 3.000 In

Unfactored Axial Squash Load Capacity = 20950.57 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	4.000000	29.9698
2	4.000000	29.7285
3	4.000000	29.2478
4	4.000000	28.5317
5	4.000000	27.5858
6	4.000000	26.4179
7	4.000000	25.0372
8	4.000000	23.4549
9	4.000000	21.6838
10	4.000000	19.7382
11	4.000000	17.6336
12	4.000000	15.3870
13	4.000000	13.0165
14	4.000000	10.5412
15	4.000000	7.9811

sheet 42 of 51
SWT 6-18-07

16	4.000000	5.3567
17	4.000000	2.6892
18	4.000000	.0000
19	4.000000	-2.6892
20	4.000000	-5.3567
21	4.000000	-7.9811
22	4.000000	-10.5412
23	4.000000	-13.0165
24	4.000000	-15.3870
25	4.000000	-17.6336
26	4.000000	-19.7382
27	4.000000	-21.6838
28	4.000000	-23.4549
29	4.000000	-25.0372
30	4.000000	-26.4179
31	4.000000	-27.5858
32	4.000000	-28.5317
33	4.000000	-29.2478
34	4.000000	-29.7285
35	4.000000	-29.9698

Axial Thrust Force = 1765000.00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
5017520.	5.017520E+12	.00000100	.00013849	138.48950
26156261.	5.231252E+12	.00000500	.00027142	54.28375626
38683684.	4.298187E+12	.00000900	.00038431	42.70107651
50342017.	3.872463E+12	.00001300	.00049155	37.81119919
61381006.	3.610647E+12	.00001700	.00059575	35.04424667
72077154.	3.432245E+12	.00002100	.00069819	33.24711227
82589542.	3.303582E+12	.00002500	.00080021	32.00840378
92952519.	3.205259E+12	.00002900	.00090216	31.10882950
1.031916E+08	3.127018E+12	.00003300	.00100445	30.43786240
1.132909E+08	3.061916E+12	.00003700	.00110662	29.90864182
1.232964E+08	3.007228E+12	.00004100	.00120999	29.51185226
1.331634E+08	2.959186E+12	.00004500	.00131313	29.18077469
1.429178E+08	2.916689E+12	.00004900	.00141705	28.91943741
1.525560E+08	2.878415E+12	.00005300	.00152177	28.71273422
1.620865E+08	2.843623E+12	.00005700	.00162839	28.56821823
1.712121E+08	2.806755E+12	.00006100	.00173360	28.41967392
1.780414E+08	2.739098E+12	.00006500	.00183116	28.17168045
1.836107E+08	2.661025E+12	.00006900	.00192451	27.89146042
1.882816E+08	2.579201E+12	.00007300	.00201478	27.59965897
1.923924E+08	2.498602E+12	.00007700	.00210441	27.33001328
1.959753E+08	2.419448E+12	.00008100	.00219030	27.04072952
1.990993E+08	2.342344E+12	.00008500	.00227443	26.75799179
2.018713E+08	2.268216E+12	.00008900	.00235778	26.49187088
2.042128E+08	2.195836E+12	.00009300	.00244651	26.30656815
2.060078E+08	2.123791E+12	.00009700	.00253218	26.10490036
2.076442E+08	2.055883E+12	.00010100	.00261905	25.93117905
2.146011E+08	1.638177E+12	.00013100	.00328910	25.10763931
2.157880E+08	1.340298E+12	.00016100	.00396661	24.63733292

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 211599.50445 In-Kip

sheet 43 of 51
GWT 6-18-07

Pile Section No. 2

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 66.0000 In

Material Properties:

Compressive Strength of Concrete	=	4.500 Kip/In**2
Yield Stress of Reinforcement	=	60. Kip/In**2
Modulus of Elasticity of Reinforcement	=	29000. Kip/In**2
Number of Reinforcing Bars	=	35
Area of Single Bar	=	4.00000 In**2
Number of Rows of Reinforcing Bars	=	35
Cover Thickness (edge to bar center)	=	3.000 In

Unfactored Axial Squash Load Capacity = 20950.57 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	4.000000	29.9698
2	4.000000	29.7285
3	4.000000	29.2478
4	4.000000	28.5317
5	4.000000	27.5858
6	4.000000	26.4179
7	4.000000	25.0372
8	4.000000	23.4549
9	4.000000	21.6838
10	4.000000	19.7382
11	4.000000	17.6336
12	4.000000	15.3870
13	4.000000	13.0165
14	4.000000	10.5412
15	4.000000	7.9811
16	4.000000	5.3567
17	4.000000	2.6892
18	4.000000	.0000
19	4.000000	-2.6892
20	4.000000	-5.3567
21	4.000000	-7.9811
22	4.000000	-10.5412
23	4.000000	-13.0165
24	4.000000	-15.3870
25	4.000000	-17.6336
26	4.000000	-19.7382
27	4.000000	-21.6838
28	4.000000	-23.4549
29	4.000000	-25.0372
30	4.000000	-26.4179
31	4.000000	-27.5858
32	4.000000	-28.5317
33	4.000000	-29.2478
34	4.000000	-29.7285
35	4.000000	-29.9698

Axial Thrust Force = 1765000.00 lbs

Sheet 44 of 51
EWT 6-18-07

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
5017520.	5.017520E+12	.00000100	.00013849	138.48950
26156261.	5.231252E+12	.00000500	.00027142	54.28375626
38683684.	4.298187E+12	.00000900	.00038431	42.70107651
50342017.	3.872463E+12	.00001300	.00049155	37.81119919
61381006.	3.610647E+12	.00001700	.00059575	35.04424667
72077154.	3.432245E+12	.00002100	.00069819	33.24711227
82589542.	3.303582E+12	.00002500	.00080021	32.00840378
92952519.	3.205259E+12	.00002900	.00090216	31.10882950
1.031916E+08	3.127018E+12	.00003300	.00100445	30.43786240
1.132909E+08	3.061916E+12	.00003700	.00110662	29.90864182
1.232964E+08	3.007228E+12	.00004100	.00120999	29.51185226
1.331634E+08	2.959186E+12	.00004500	.00131313	29.18077469
1.429178E+08	2.916689E+12	.00004900	.00141705	28.91943741
1.525560E+08	2.878415E+12	.00005300	.00152177	28.71273422
1.620865E+08	2.843623E+12	.00005700	.00162839	28.56821823
1.712121E+08	2.806755E+12	.00006100	.00173360	28.41967392
1.780414E+08	2.739098E+12	.00006500	.00183116	28.17168045
1.836107E+08	2.661025E+12	.00006900	.00192451	27.89146042
1.882816E+08	2.579201E+12	.00007300	.00201478	27.59965897
1.923924E+08	2.498602E+12	.00007700	.00210441	27.33001328
1.959753E+08	2.419448E+12	.00008100	.00219030	27.04072952
1.990993E+08	2.342344E+12	.00008500	.00227443	26.75799179
2.018713E+08	2.268216E+12	.00008900	.00235778	26.49187088
2.042128E+08	2.195836E+12	.00009300	.00244651	26.30656815
2.060078E+08	2.123791E+12	.00009700	.00253218	26.10490036
2.076442E+08	2.055883E+12	.00010100	.00261905	25.93117905
2.146011E+08	1.638177E+12	.00013100	.00328910	25.10763931
2.157880E+08	1.340298E+12	.00016100	.00396661	24.63733292

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 211599.50445 In-Kip

---

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 75500.000 lbs
Specified moment at pile head = .000 in-lbs
Specified axial load at pile head = 1765000.000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	.956557	.000463	75500.	-.002525	515.931	5.02E+12	0.000
7.440	.937768	5.95E+05	75500.	-.002525	537.008	5.02E+12	0.000
14.880	.918986	1.19E+06	75500.	-.002524	558.084	5.02E+12	0.000
22.320	.900216	1.78E+06	75500.	-.002521	579.159	5.02E+12	0.000
29.760	.881467	2.38E+06	75500.	-.002518	600.233	5.02E+12	0.000
37.200	.862744	2.97E+06	75500.	-.002514	621.306	5.02E+12	0.000
44.640	.844054	3.57E+06	75500.	-.002509	642.376	5.02E+12	0.000
52.080	.825403	4.16E+06	75500.	-.002504	663.444	5.02E+12	0.000

Sheet 45 of 51
EWT 6-18-07

59.520	.806798	4.76E+06	75500.	-.002497	684.509	5.02E+12	0.000
66.960	.788245	5.35E+06	75500.	-.002490	705.571	5.03E+12	0.000
74.400	.769751	5.95E+06	75500.	-.002481	726.629	5.06E+12	0.000
81.840	.751323	6.54E+06	75500.	-.002472	747.683	5.08E+12	0.000
89.280	.732965	7.14E+06	75500.	-.002462	768.732	5.09E+12	0.000
96.720	.714686	7.73E+06	75500.	-.002451	789.777	5.11E+12	0.000
104.160	.696490	8.32E+06	75500.	-.002440	810.816	5.12E+12	0.000
111.600	.678384	8.92E+06	75500.	-.002427	831.850	5.13E+12	0.000
119.040	.660374	9.51E+06	75500.	-.002414	852.878	5.14E+12	0.000
126.480	.642466	1.01E+07	75500.	-.002400	873.900	5.15E+12	0.000
133.920	.624668	1.07E+07	75500.	-.002385	894.914	5.16E+12	0.000
141.360	.606984	1.13E+07	75500.	-.002369	915.922	5.16E+12	0.000
148.800	.589421	1.19E+07	75500.	-.002352	936.922	5.17E+12	0.000
156.240	.571985	1.25E+07	75500.	-.002335	957.913	5.17E+12	0.000
163.680	.554683	1.31E+07	75500.	-.002316	978.897	5.18E+12	0.000
171.120	.537521	1.37E+07	75500.	-.002297	999.872	5.18E+12	0.000
178.560	.520504	1.43E+07	75500.	-.002277	1020.838	5.19E+12	0.000
186.000	.503639	1.48E+07	75500.	-.002256	1041.794	5.19E+12	0.000
193.440	.486933	1.54E+07	75500.	-.002234	1062.740	5.19E+12	0.000
200.880	.470391	1.60E+07	75500.	-.002212	1083.676	5.20E+12	0.000
208.320	.454020	1.66E+07	75500.	-.002189	1104.602	5.20E+12	0.000
215.760	.437826	1.72E+07	75500.	-.002164	1125.516	5.20E+12	0.000
223.200	.421814	1.78E+07	75500.	-.002139	1146.419	5.21E+12	0.000
230.640	.405992	1.84E+07	75500.	-.002114	1167.310	5.21E+12	0.000
238.080	.390365	1.90E+07	75500.	-.002087	1188.189	5.21E+12	0.000
245.520	.374940	1.96E+07	75500.	-.002059	1209.055	5.21E+12	0.000
252.960	.359723	2.02E+07	75500.	-.002031	1229.908	5.22E+12	0.000
260.400	.344719	2.07E+07	75500.	-.002002	1250.748	5.22E+12	0.000
267.840	.329935	2.13E+07	75500.	-.001972	1271.574	5.22E+12	0.000
275.280	.315378	2.19E+07	75500.	-.001941	1292.386	5.22E+12	0.000
282.720	.301053	2.25E+07	75500.	-.001909	1313.183	5.22E+12	0.000
290.160	.286966	2.31E+07	75500.	-.001877	1333.966	5.22E+12	0.000
297.600	.273124	2.37E+07	75500.	-.001844	1354.733	5.23E+12	0.000
305.040	.259533	2.43E+07	75500.	-.001810	1375.484	5.23E+12	0.000
312.480	.246199	2.48E+07	75500.	-.001775	1396.220	5.23E+12	0.000
319.920	.233128	2.54E+07	75500.	-.001739	1416.939	5.23E+12	0.000
327.360	.220326	2.60E+07	75500.	-.001702	1437.641	5.23E+12	0.000
334.800	.207799	2.66E+07	75500.	-.001665	1458.326	5.17E+12	0.000
342.240	.195557	2.72E+07	75500.	-.001626	1478.993	5.10E+12	0.000
349.680	.183609	2.78E+07	75500.	-.001585	1499.642	5.04E+12	0.000
357.120	.171967	2.83E+07	75500.	-.001544	1520.272	4.97E+12	0.000
364.560	.160641	2.89E+07	75500.	-.001500	1540.881	4.92E+12	0.000
372.000	.149640	2.95E+07	75500.	-.001456	1561.471	4.86E+12	0.000
379.440	.138976	3.01E+07	75500.	-.001410	1582.039	4.81E+12	0.000
386.880	.128657	3.07E+07	74811.	-.001363	1602.586	4.76E+12	-185.267
394.320	.118696	3.12E+07	71843.	-.001314	1622.748	4.72E+12	-612.469
401.760	.109100	3.18E+07	66199.	-.001264	1641.685	4.68E+12	-904.924
409.200	.099881	3.23E+07	59159.	-.001213	1658.824	4.64E+12	-987.380
416.640	.091047	3.27E+07	51605.	-.001161	1674.002	4.61E+12	-1043.342
424.080	.082605	3.31E+07	43670.	-.001108	1687.110	4.59E+12	-1089.654
431.520	.074561	3.34E+07	38559.	-.001054	1698.056	4.57E+12	-284.259
438.960	.066922	3.37E+07	36367.	-.000999	1708.419	4.55E+12	-304.924
446.400	.059692	3.39E+07	34056.	-.000944	1718.158	4.53E+12	-316.391
453.840	.052876	3.42E+07	31690.	-.000888	1727.251	4.52E+12	-319.604
461.280	.046479	3.44E+07	29328.	-.000831	1735.691	4.51E+12	-315.518
468.720	.040505	3.46E+07	27019.	-.000774	1743.486	4.49E+12	-305.098
476.160	.034957	3.48E+07	24808.	-.000717	1750.656	4.48E+12	-289.322
483.600	.029841	3.50E+07	22730.	-.000659	1757.232	4.47E+12	-269.175
491.040	.025157	3.52E+07	20815.	-.000600	1763.252	4.46E+12	-245.648
498.480	.020911	3.54E+07	19084.	-.000541	1768.763	4.45E+12	-219.742
505.920	.017104	3.55E+07	-29454.	-.000482	1773.816	4.45E+12	-12828.
513.360	.013739	3.49E+07	-1.16E+05	-.000423	1753.684	4.48E+12	-10304.
520.800	.010806	3.38E+07	-1.84E+05	-.000367	1713.316	4.54E+12	-8104.429

Sheet 40 of 51
GWT 6-18-07

528.240	.008285	3.22E+07	-2.37E+05	-.000313	1657.029	4.65E+12	-6213.465
535.680	.006147	3.03E+07	-2.78E+05	-.000264	1588.531	4.79E+12	-4610.284
543.120	.004359	2.81E+07	-3.07E+05	-.000219	1510.971	5.00E+12	-3269.259
550.560	.002882	2.57E+07	-3.27E+05	-.000180	1426.979	5.23E+12	-2161.370
558.000	.001677	2.32E+07	-3.40E+05	-.000145	1338.731	5.22E+12	-1257.585
565.440	.000718	2.07E+07	-3.46E+05	-.000114	1248.002	5.22E+12	-538.336
572.880	-2.20E-05	1.81E+07	-3.48E+05	-8.65E-05	1156.204	5.21E+12	16.493
580.320	-.000570	1.55E+07	-3.47E+05	-6.25E-05	1064.425	5.20E+12	427.258
587.760	-.000952	1.29E+07	-3.42E+05	-4.22E-05	973.474	5.18E+12	714.308
595.200	-.001197	1.04E+07	-3.23E+05	-2.54E-05	883.916	5.15E+12	4533.339
602.640	-.001330	8.11E+06	-2.87E+05	-1.20E-05	803.241	5.12E+12	5037.312
610.080	-.001376	6.11E+06	-2.49E+05	-1.60E-06	732.440	5.06E+12	5209.023
617.520	-.001354	4.40E+06	-2.11E+05	6.15E-06	671.850	5.02E+12	5127.739
624.960	-.001284	2.97E+06	-1.74E+05	1.16E-05	621.314	5.02E+12	4862.596
632.400	-.001181	1.82E+06	-1.39E+05	1.52E-05	580.311	5.02E+12	4473.188
639.840	-.001058	9.07E+05	-1.07E+05	1.72E-05	548.081	5.02E+12	4007.863
647.280	-.000925	2.20E+05	-79388.	1.80E-05	523.709	5.02E+12	3504.627
654.720	-.000790	-2.74E+05	-55219.	1.80E-05	525.651	5.02E+12	2992.220
662.160	-.000658	-6.03E+05	-34821.	1.73E-05	537.281	5.02E+12	2491.274
669.600	-.000532	-7.93E+05	-18056.	1.63E-05	544.024	5.02E+12	2015.503
677.040	-.000415	-8.72E+05	-4706.885	1.51E-05	546.815	5.02E+12	1572.860
684.480	-.000308	-8.63E+05	5484.032	1.38E-05	546.520	5.02E+12	1166.634
691.920	-.000210	-7.90E+05	12787.	1.26E-05	543.936	5.02E+12	796.479
699.360	-.000121	-6.73E+05	17458.	1.15E-05	539.790	5.02E+12	459.347
706.800	-3.97E-05	-5.31E+05	19727.	1.06E-05	534.743	5.02E+12	150.351
714.240	3.60E-05	-3.80E+05	19778.	9.90E-06	529.401	5.02E+12	-136.463
721.680	.000108	-2.37E+05	17755.	9.44E-06	524.325	5.02E+12	-407.394
729.120	.000177	-1.16E+05	13753.	9.18E-06	520.049	5.02E+12	-668.426
736.560	.000244	-32524.	7826.964	9.07E-06	517.083	5.02E+12	-924.602
744.000	.000311	0.000	0.000	9.04E-06	515.931	5.02E+12	-1179.420

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =       .95655657 in
Computed slope at pile head       =      -.00252537
Maximum bending moment            =       35503626. lbs-in
Maximum shear force               =   -348367.38408 lbs
Depth of maximum bending moment   =       505.92000 in
Depth of maximum shear force      =       572.88000 in
Number of iterations              =              10
Number of zero deflection points  =               2
```

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Sheet 4 / 7
GWT 6-18-07

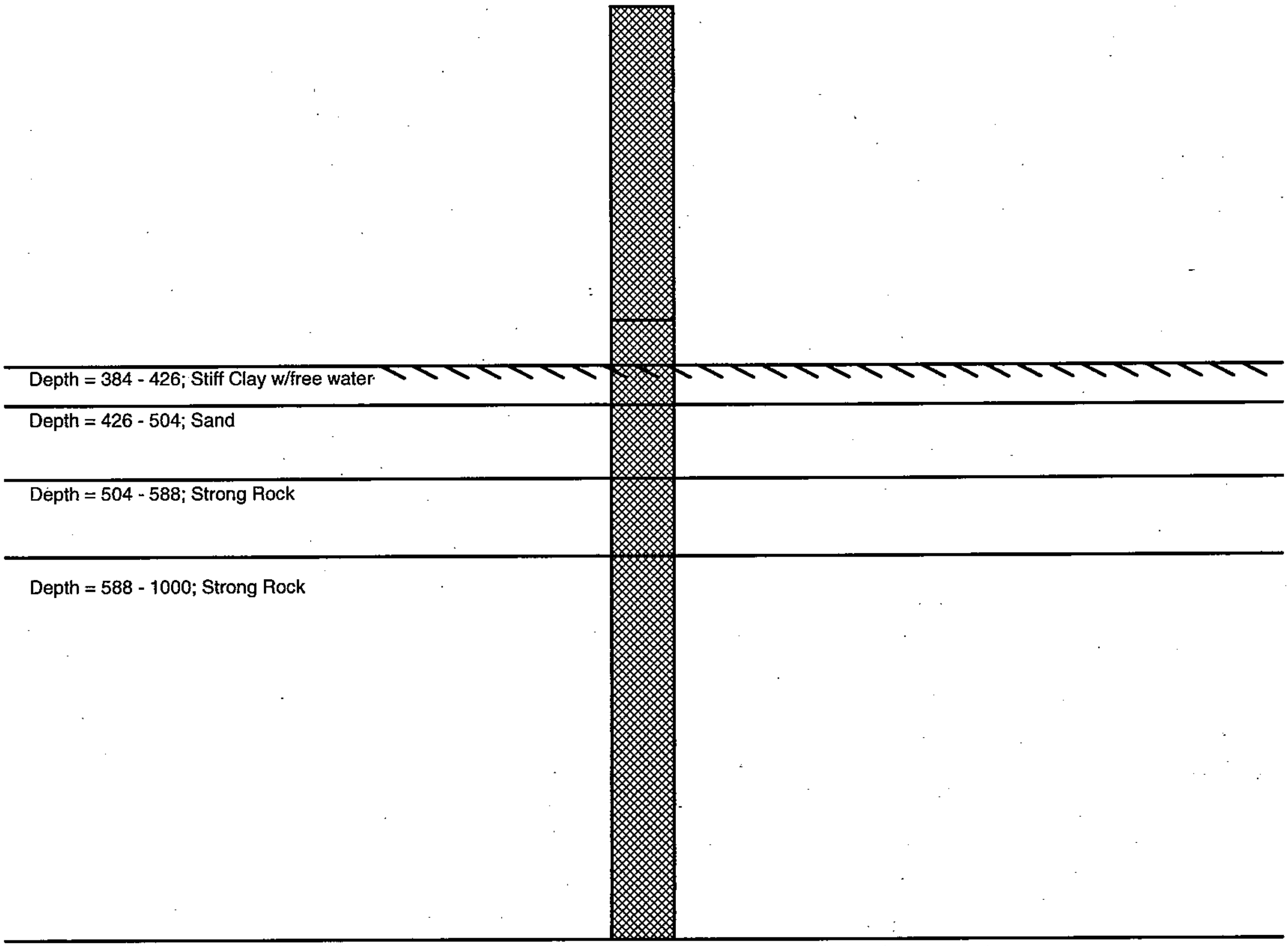
Ramp B Lat Analysis - Pier 2.lpo

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = pile-head shear force lbs
Type 4 = Deflection and Moment, S = pile-head slope, radians
Type 5 = Deflection and Slope, R = rotational stiffness of pile-headin-lbs/rad

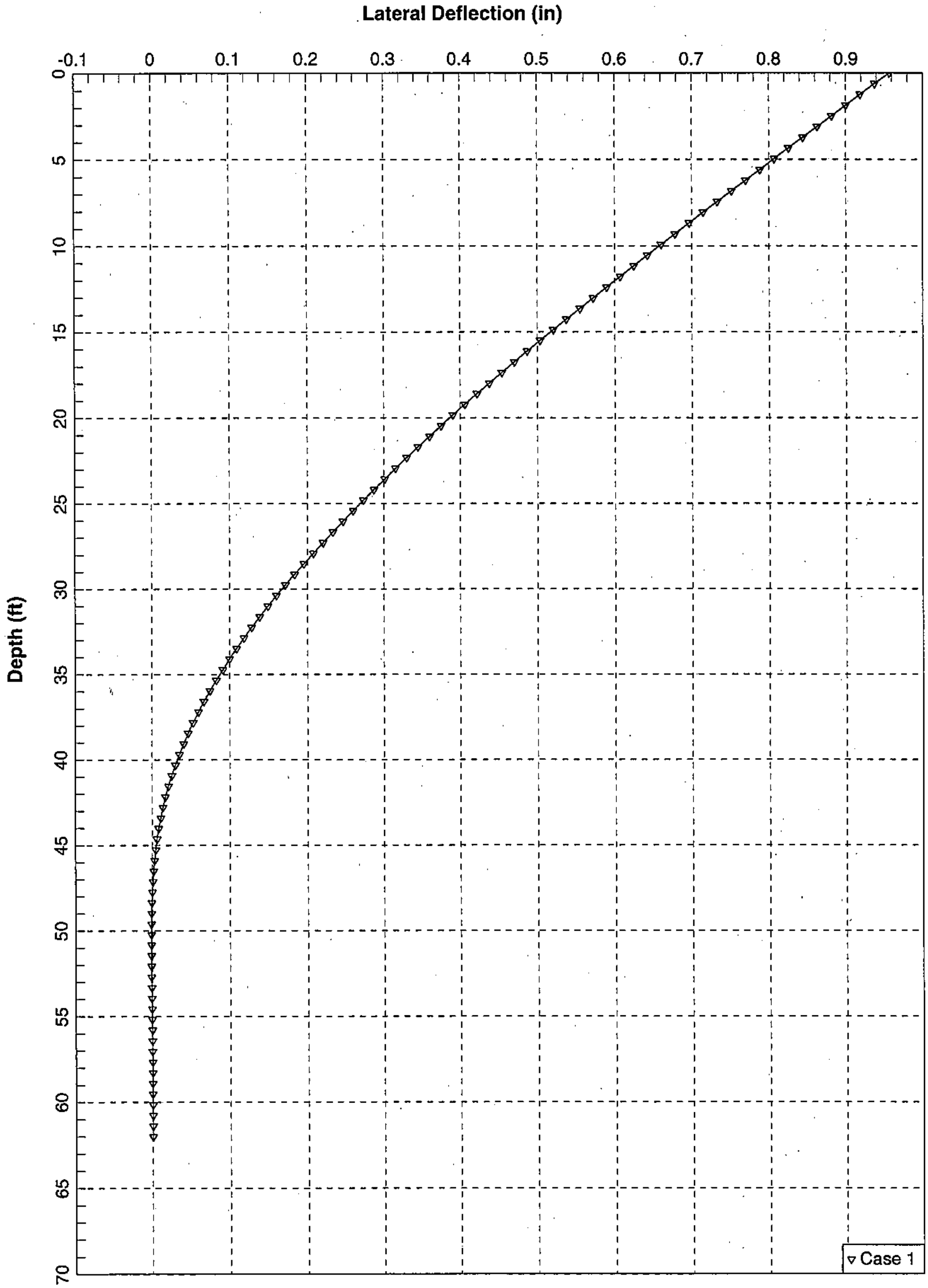
Load Type	Boundary Condition 1		Boundary Condition 2		Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V=	75500.	M=	0.000	1765000.	.9565566	3.5504E+07	-348367.

The analysis ended normally.

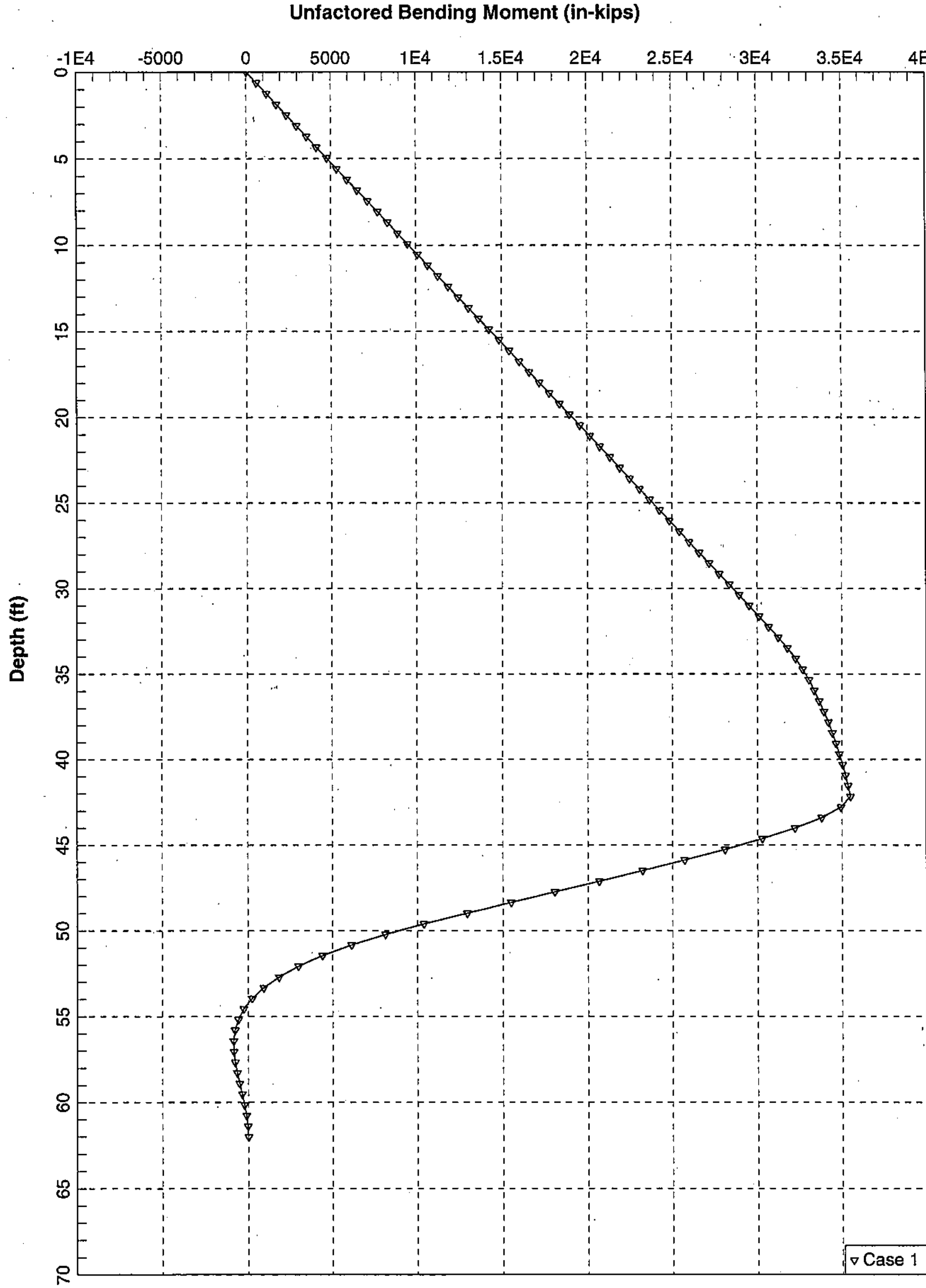
Sheet 48 of 51
EWT 6-18-07



sheet 44 of 51
GWT 6-18-07



sheet 50 of 51
CWT 6-18-07



Sheet 51 of 51
GWT 6-18-07