Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
US 52 Ramp A Over Ohio River Road (To Northbound SR 823)
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

STRUCTURAL ENGINEERING
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June 1, 2007

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

US 52 RAMP A OVER OHIO RIVER ROAD (TO NORTHBOUND SR 823)

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

SCIOTO COUNTY, OHIO

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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 A of the Portsmouth bypass project. The findings included in this report pertain to US 52 Ramp A to northbound SR 823 only. The findings of other structure evaluations for the Portsmouth bypass project will be submitted in separate documents.

This project consists in part of constructing a bridge for proposed US 52 Ramp A over Ohio River Road (CR 503). The structure as planned, is a two-span structure 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 plan, which is 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 27.9 and 34.1 feet, respectively. These heights are based upon the maximum difference between the proposed grade of US 52 Ramp A and the existing grade as per the revised profile for Ramp A, received May 15, 2007. In addition, it is understood that the MSE walls will be placed at approximate stations 39+17 and 41+50 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 six structure borings for the proposed bridge and MSE walls. Borings B-33 through B-36 were drilled for the currently proposed structure, as indicated on the structure site plan. These borings were drilled between January 26 and February 1, 2007. Borings TR-62 and TR-76 were drilled for a previous design configuration. These borings were drilled between March 18 and 30, 2005. The bedrock encountered in boring TR-66, drilled for Ramp B, was also considered in the analyses of the drilled shaft foundations. 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. Laboratory test results are presented on the boring logs and also in Appendix III.

4.2.1 Soil Conditions

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

Borings B-33 and B-36 were drilled for the rear and forward abutments, respectively, of the approved structure configuration. Similarly, borings B-34 and B-35 were drilled for the pier of the currently proposed structure. Boring TR-76 was also considered in the evaluation of the rear abutment location, while boring TR-62 was considered in the evaluation of the forward abutment location.

All borings, except boring TR-62, encountered 2 to 5 inches of topsoil underlain by natural soils. Boring TR-62 encountered 3 inches of aggregate base at the ground surface. Borings B-33, B-34, TR-62, and TR-76 encountered natural cohesive soil deposits below the surface material, while borings B-35 and B-36 encountered natural granular soil deposits. The natural cohesive deposits consisted of hard Silt and Clay (A-6a), stiff to hard Sandy Silt (A-4a), and very stiff Silt (A-4b) , while the granular soil deposits consisted mainly of medium dense to very dense Sandy Silt (A-4a), and very dense Gravel with Sand and Silt (A-2-4). The native soil deposits extended to depths ranging between approximately 3.0 and 13.0 feet below the ground surface, where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. The bedrock consisted of soft to hard, slightly to highly weathered, argillaceous sandstone. Severely weathered, argillaceous sandstone was encountered in borings B-33, B-34, B-35, and TR-62 above the competent sandstone. The amount of rock recovered in each core run varied between 92 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 60 and 86 percent with an average of 73 percent indicating “fair” to “good” rock.

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 strength of the tested cores is shown in Table 1. Anticipating the possibility that the foundations will need to be designed for lateral loading, the elastic modulus of selected cores was also measured. The results of these tests are presented in Appendix III.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-33	20.8-21.3	149.7	9,284
B-34	20.1-20.7	155.4	5,450
B-35	8.4-8.9	157.2	10,892
B-36	9.6-10.0	158.3	9,260
TR-62	9.3-9.7	154.4	10,794
TR-76	19.6-20.0	140.2	11,036

4.2.3 Groundwater Conditions

Seepage was not observed in any of the borings drilled for this structure. There were no measurable water levels in the borings prior to rock coring. Measurable water levels were present in all borings upon the completion of coring between approximate depths of 1.2 and 10.5 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 pier. Additionally, it is understood that MSE walls will be used to contain the abutments and hold back the roadway embankment. Recommendations for the piles, drilled shafts, spread footings, and MSE walls are presented in the following sections.

5.1 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. Based upon subsurface conditions, it was assumed that deep foundations would be used for the stability analyses and settlement calculations for the proposed MSE walls.

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 B-33)	Stiff to Hard Sandy Silt (A-4a)	120	1750	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 11.0-foot thick soil layer at this location. In contrast, competent bedrock was encountered within 3.0 feet of the existing ground surface at the forward abutment.

An embankment height of 27.9 feet, as shown on the structure site plan, 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 30.9 feet was assumed for the analyses. Additionally, the soil profile and properties encountered by boring B-33 were assumed the analyses of the rear abutment MSE wall.

Boring B-33 encountered stiff Sandy Silt (A-4a) to an approximate depth of 8.0 feet below the ground surface. Below this layer, very dense Gravel with Sand and Silt (A-2-4), and severely weathered sandstone was encountered to an approximate depth of 14.5 feet below the ground surface, at the top of cored bedrock.

Initially, analyses were undertaken to ascertain 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 is 2.3, 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. Consequently, it is recommended that the existing foundations soils be overexcavated to an approximate depth of 4.3 feet below the bottom of the proposed leveling pad, corresponding to an approximate elevation of 550.4 (based on boring B-33). 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 have indicated that a minimum reinforcement length of 0.7 times the full height (H+D) or 21.6 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 4 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 0.7 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.

Therefore, if the recommended overexcavation is preformed, the anticipated settlement of the proposed MSE wall at the rear abutment is assumed to be negligible.

If overexcavation of the soft / compressible soils at the rear abutment is performed as recommended, it will not be necessary to monitor the settlement of the proposed embankments as previously thought.

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) ***Undercut with Compacted Granular Fill Foundation***

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) $q_{all} = 3,667$ *psf*
Allowable Bearing Capacity – Drained Condition (With overexcavation) $q_{all} = 6,853$ psf
Global Stability *(Without Overexcavation)* Factor of Safety – Undrained Condition = 2.0 Factor of Safety – Drained Condition = 1.7 Factor of Safety – Drained Seismic Condition = 1.6
Estimated Settlement of MSE volume Maximum Total Settlement = 4.0 inches (Without Overexcavation) Differential Settlement = 0.7% (maximum allowable is 1.0% ODOT BDM 204.6.2.1) Maximum Total Settlement (With Overexcavation) - Negligible
Full Height of MSE Wall = 30.9 feet (including embedment depth) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability, 0.7(H+D) = 21.6 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 $\varphi'=34°$, 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, $\varphi'=30°$.

5.1.3 MSE Wall Evaluations and Recommendations – Forward Abutment

An embankment height of 34.1 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 37.1 feet was assumed for the analyses. Additionally, the soil profile and properties encountered by boring B-36 were assumed for the analyses of the forward abutment MSE wall.

Boring B-36 encountered Sandy Silt (A-4a) to a depth of 3.0 feet below the ground surface. Below the thin soil layer, slightly weathered sandstone was encountered to a depth of 13.0 feet below the ground surface, upon termination of the boring.

Consequently, at the forward abutment, 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 26.0 feet is required for stability of the proposed MSE wall at the forward abutment location.

5.2 Bridge Foundation Recommendations

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

SEE BDM 202.2.3.2.a.
STEEL 'H' PILES DESIGN LOADS
HAVE BEEN REVISED

Table 4-Summary of Foundation Recommendations

Structural Element	Structure / Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	B-33	558.4	HP 12x53-driven	547.4 ✓	70 tons
			CIP Piles-prebored	537.4*	Pile Capacity^{++}
			Drilled Shafts	537.4*	40 ksf^{+++}
Pier	Left / B-34	558.6	Spread Footings	542.6	40 ksf
			HP 12x53-driven	544.6	70 tons
			CIP Piles-prebored	537.6*	Pile Capacity^{++}
			Drilled Shaft	525.6**	40 ksf^{+++}
	Right / B-35	558.4	Spread Footings	555.8	40 ksf
			Drilled Shafts	536.9**	40 ksf^{+++}
Forward Abutment	B-36	558.5	CIP Piles	549.4*	Pile Capacity^{++}
			Drilled Shaft	549.4*	40 ksf^{+++}

* Includes 5-foot socket into competent rock, assumes no significant lateral loads.

** Drilled shaft tip elevation reflects 19-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

From comments by OSE, which are based on preliminary borings, it is understood that driven H-piles are preferred to support the abutments of the proposed structure. However, additional borings drilled for the currently proposed structure indicate that approximately 11.0 feet of overburden is present at the rear abutment location.

For MSE wall stability purposes, an overexcavation of the existing foundation soils is recommended at the rear abutment location. Consequently, if this is performed, the anticipated settlement will be negligible. As a result, no appreciable downdrag forces will be applied to the piles.

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, at a depth of approximately 11.0 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 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 Pier

The currently proposed structure utilizes an integral straddle bent pier. The proposed bottom of footing / pile cap is assumed to be approximately elevation 552.78, as shown on the structure site plan presented in Appendix I. Due to the variation in subsurface conditions, recommendations for left and right pier foundations are presented separately.

Borings drilled for the pier encountered weathered bedrock at approximate elevations 545.6 and 556.4 for the left and the right substructures, respectively.

5.2.2.1 Pier – Left Foundation

Boring B-34 was drilled for the currently proposed left pier foundation. Boring B-34 encountered approximately 13.0 feet of Silt (A-4b) and Sandy Silt (A-4a) overlying bedrock. Highly weathered, argillaceous sandstone was encountered in this boring below the soil to a depth of 19.7 feet, where more competent bedrock was encountered.

At the proposed foundation elevation of 552.78, boring B-34 encountered very stiff Sandy Silt. 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 left pier. This footing should be founded at or below elevation 542.6, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf).

Consideration should be given to the means and extent of the excavation, which would be required for the use of spread footings. An excavation approximately 16.0 feet deep would be required. To avoid the closure of the adjacent Ohio River Road and disruption of existing utilities, significant shoring would be required.

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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 left pier. It is anticipated that piles could be driven to a depth of 14 feet, corresponding to an elevation of 544.6 (as per boring B-34). Depending on the elevation and configuration of the pile cap, driven piles may be of concern at this location. Typically, seven to ten feet of pile embedment is desirable for the lateral support of driven piles.

If sufficient embedment cannot be achieved with driven H-piles piles the use of prebored CIP piles could also 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. At this time, lateral loading and uplift is not anticipated to be a concern for this type of foundation. However, if these forces are determined to be significant, longer socket lengths may be required.

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 drilled shaft could support the 54-inch column and limit deflections at the top of the column to approximately 0.6 inches. Analyses also indicate that a 19-foot deep rock socket will be required to resist the lateral loading. Based upon boring B-34, this corresponds to a bearing elevation of 525.6. 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 19-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 – Right Foundation

Boring B-35 was drilled for the currently proposed right pier foundation. Boring B-35 encountered only 2.0 feet of Gravel with Sand and Silt (A-2-4) overlying bedrock. Moderately weathered, argillaceous sandstone was encountered below the soil to the completion depth of 12.5 feet.

Based upon the conditions encountered in boring B-35, it is recommended that spread footings, founded on rock be considered to support the right pier. This footing should be founded at or below elevation 555.7, and may be designed based upon an allowable bearing pressure of 40 ksf (20 tsf). It should be noted that competent bedrock was encountered above the proposed bottom of footing, as indicated on the structure site plan in Appendix I. Consequently, excavations approximately 2.9 feet into competent bedrock will be required to construct the footing as shown on the structure site plan.

Given the column arrangement being utilized for the straddle bent pier, a single drilled shaft has been considered for the support of the right 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 drilled shaft could support the 54-inch column and limit deflections at the top of the column to approximately 0.5 inches. Analyses also indicate that a 19-foot deep rock socket will be required to resist the lateral loading. Based upon boring B-35, this corresponds to a bearing elevation of 536.9. 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 19-foot deep rock socket is required for stability under lateral loading. It may be increased if necessary for axial loading.

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.

5.2.3 Forward Abutment

Boring B-36, drilled for the currently proposed forward abutment, indicates that approximately 3.0 feet of overburden is present at forward abutment location. Below the soil, slightly weathered argillaceous sandstone was encountered to the completion depth of 13.0 feet.

The rear Fwd. 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 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 2. 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 A over Ohio River Road traverses a gently to moderately sloping area. Consequently, the placement of fill will be required to construct the approach embankments at the abutments. The maximum fill anticipated is approximately 35 feet, near the proposed forward abutment. In addition, excavations up to 16 feet deep are anticipated for the 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 2 to 5 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 not encountered in any of the borings and no groundwater was noted prior to adding drill 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.

Steven J. Riedy DAA

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams

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

sjr

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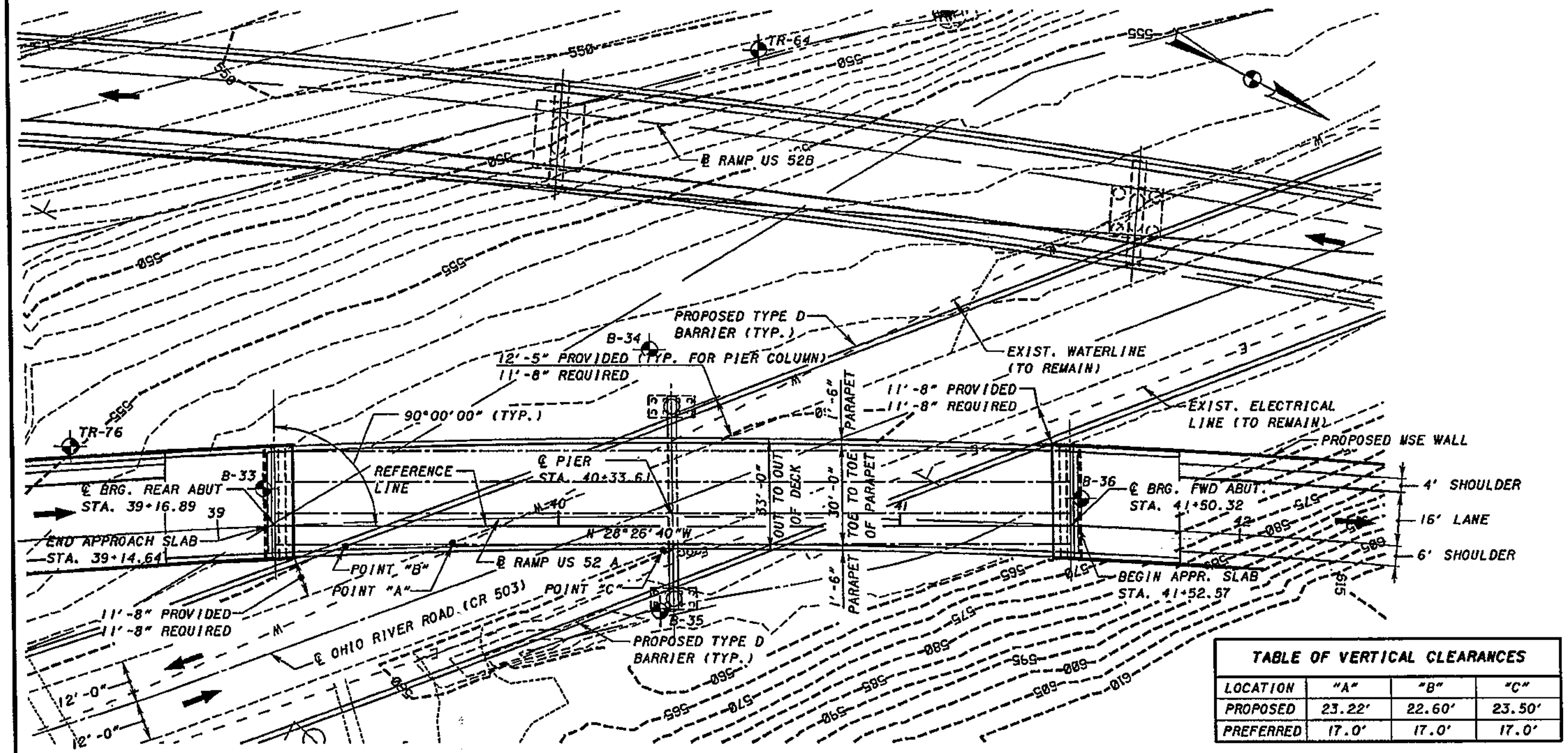
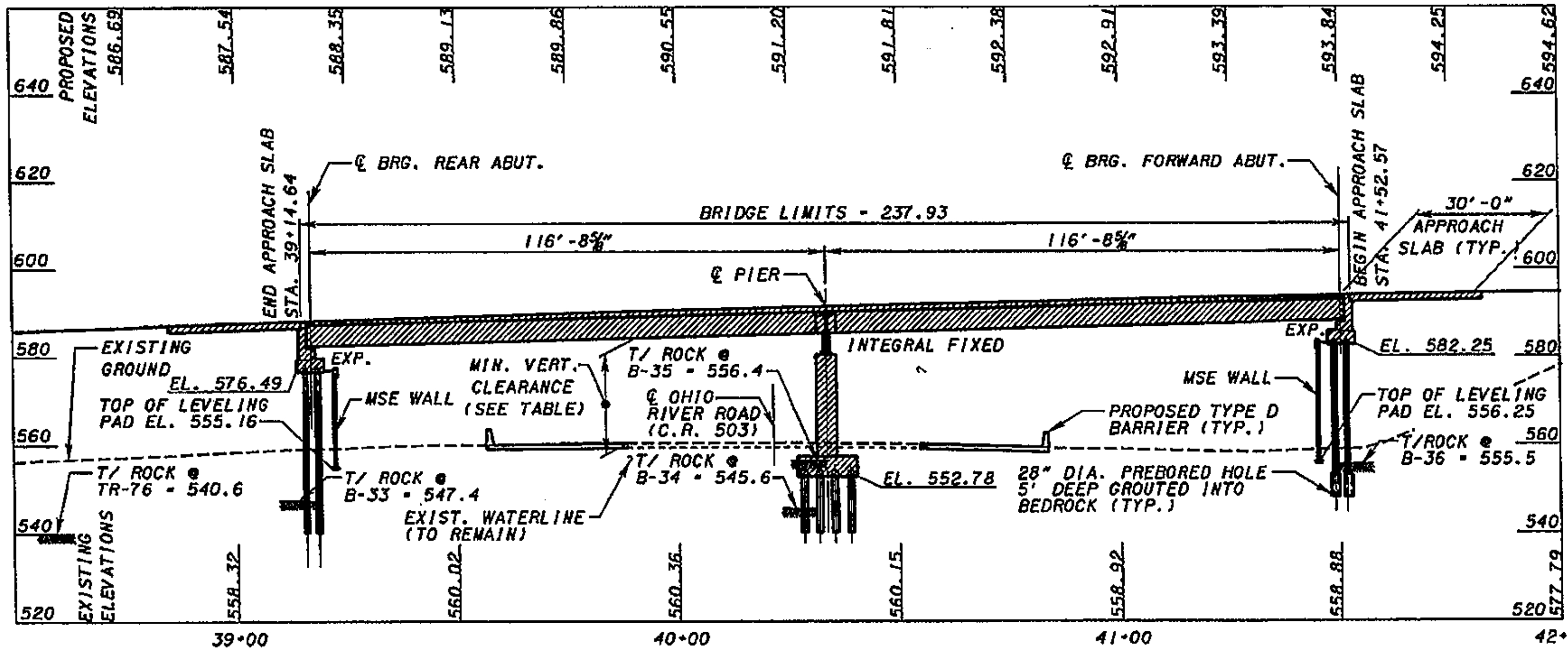
APPENDIX I

Structure Plan and Profile Drawing - 11"x17"



PLAN

TABLE OF VERTICAL CLEARANCES			
LOCATION	"A"	"B"	"C"
PROPOSED	23.22'	22.60'	23.50'
PREFERRED	17.0'	17.0'	17.0'



ELEVATION ALONG ℄ RAMP US 52 A

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-62	43+02.06	93.34 LT.
TR-64	40+58.96	138.58 LT.
TR-76	38+59.24	27.09 LT.

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

TRAFFIC DATA

(ROUTE)

CURRENT YEAR ADT (2010) - 6,700
DESIGN YEAR ADT (2030) - 10,500
CURRENT YEAR ADTT (2010) - 938
DESIGN YEAR ADTT (2030) - 1,470

PROPOSED STRUCTURE

TYPE: 2 SPAN CONTINOUS 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: 116'-8⅝", 116'-8⅝" C/C BEARINGS
ROADWAY: 30' 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: 0.056 FT/FT ACROSS LANE
ALIGNMENT: Dc = 2°15'00" CURVE TO THE RIGHT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)
LATITUDE:
LONGITUDE:

HORIZONTAL CURVE DATA:

P.I. = STA. 44+40.37
△ = 34°25'56.25'
Dc = 2°15'00"
R = 2546.48'
T = 789.05'
L = 1530.32'
E = 119.45'
e_{max} = 0.056
P.C. = STA. 36+51.32
P.T. = STA. 51+81.64

900' VERT. CURVE DATA

P.V.I. = STA. 41+00.00
P.V.I. EL. = 599.46
G_1 = +4.95%
G_2 = -0.88%

ESTIMATED PILE LENGTHS
REAR ABUTMENT

ORDER LENGTH - 50'
ESTIMATED LENGTH - 45'

ESTIMATED PILE LENGTHS
FORWARD ABUTMENT

ORDER LENGTH - 40'
ESTIMATED LENGTH - 35'

ESTIMATED PILE LENGTHS
PIER

ORDER LENGTH - 20'
ESTIMATED LENGTH - 15'

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 16" DIA. CIP PILES AND HAVE A MAXIMUM CAPACITY OF 90 TONS

DESIGN AGENCY
Systems
DATE 12/5/06
REVIEWED JRC
STRUCTURE FILE NUMBER
DRAWN MTN
REVISED
DESIGNED MSL
CHECKED PJP

SCIOTO COUNTY
STA. 39+14.64
STA. 41+52.57

PRELIMINARY SITE PLAN
BRIDGE US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 77366

1 / 6

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APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Seven (7) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

Granular Soils – Compactness

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

Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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.

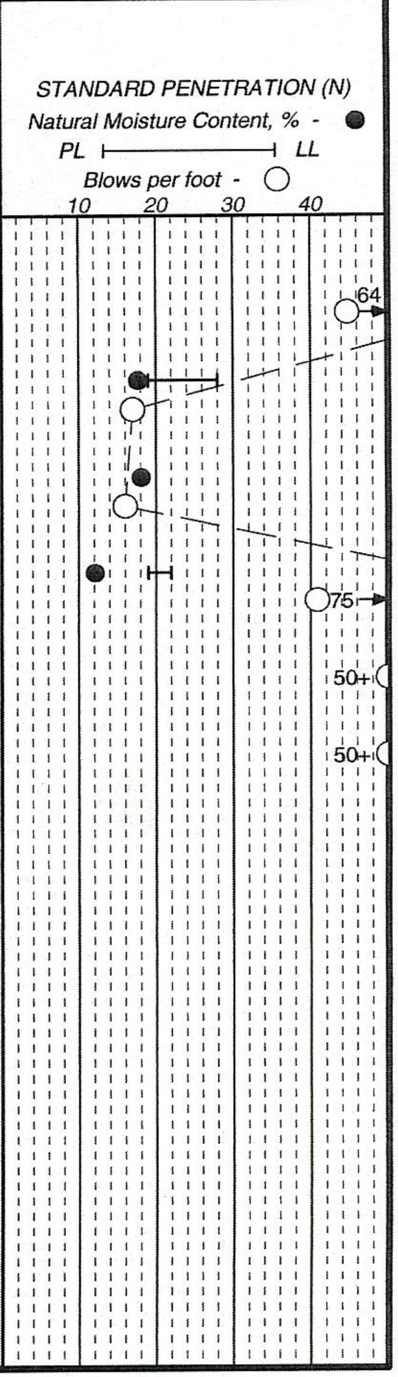
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-33 | Location: Sta. 39+14.0, 11.2 ft. LT of US 52 Ramp A BL | Date Drilled: 02/01/07

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.4												
0.4	558.0						Topsoil - 5"						
		22 32 32	18	1		3.0	Very stiff brown SANDY SILT (A-4a), little to some gravel, trace to little clay; contains sandstone fragments; dry to damp.						
5		4 5 12	18	2		2.25		13	12	--	10	48	17
		7 8 8	18	3		1.5	@ 6.0' stiff, moist.						
8.0	550.4												
10		38 48 27	18	4		1.5	Very dense brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; dry to damp.	26	26	--	17	26	5
11.0	547.4	48 50/5	4	5			Severely weathered brown SANDSTONE, argillaceous.						
		50/5	5	6									
14.5 15	543.9						Soft to medium hard brown and gray SANDSTONE; very fine to medium grained, highly weathered, argillaceous, laminated, highly fractured, contains clay seams. @ 14.7'-15.1', lost recovery.						
20.0	538.4	Core 120"	Rec 116"	RQD 60%	R1		Medium hard to hard gray SANDSTONE; fine grained, slightly weathered, micaceous, argillaceous, thinly bedded, slightly fractured. @ 20.8'-21.3', qu = 9,284 psi.						
24.5 25	533.9						Bottom of Boring - 24.5'						
30													



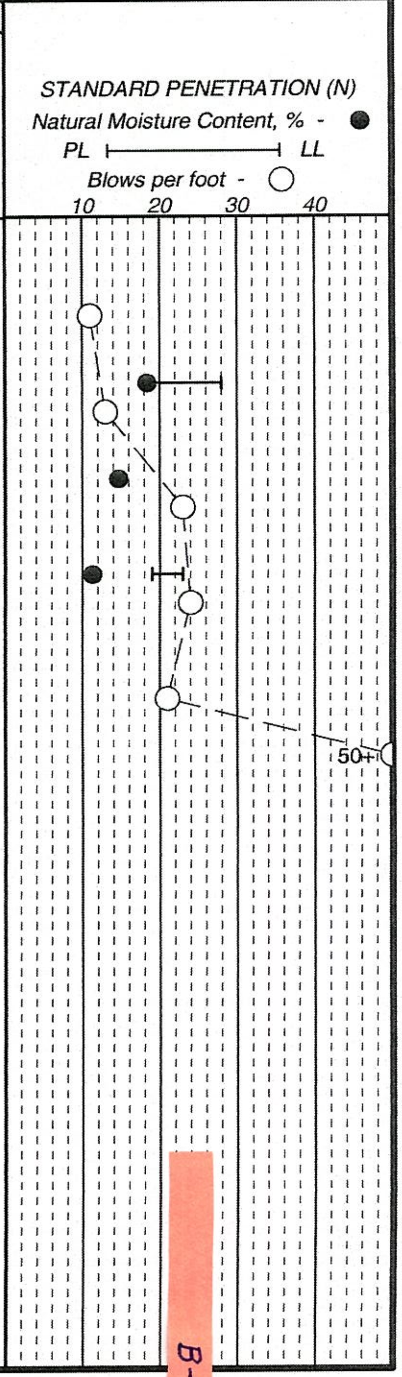
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-34
Location: Sta. 40+27.4, 49.7 ft. LT of US 52 Ramp A BL
Date Drilled: 01/26/07

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.6												
0.4	558.2						Topsoil - 5"						
		5 6 5	18	1		3.0	Very stiff brown SILT (A-4b), some fine to coarse sand, little clay, trace gravel; damp to moist.						
		3 4 9	18	2		2.0		6	12	--	12	54	16
5 5.5	553.1												
		8 8 15	18	3		2.5	Very stiff brown SANDY SILT (A-4a), little gravel, trace clay; contains sandstone fragments; damp to moist.						
10		10 11 13	18	4		--		13	23	--	21	36	7
		8 10 11	18	5		--							
13.0	545.6						Severely weathered brownish gray SANDSTONE.						
14.0	544.6	50/5	3	6			Soft to medium hard brown and gray SANDSTONE; very fine to medium fine grained, highly weathered, argillaceous, laminated, highly fractured, contains clay seams, iron stained.						
15		Core 120"	Rec 120"	RQD 65%	R1								
19.7 20	538.9						Medium hard to hard gray SANDSTONE; fine grained, slightly weathered, argillaceous, micaceous, laminated to thinly bedded, slightly fractured. @ 20.0'-20.1', clay seam. @ 20.1'-20.7', qu = 5,450 psi, Er = 326,649 psi.						
24.0	534.6						Bottom of Boring - 24.0'						
25													
30													



B-34

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

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

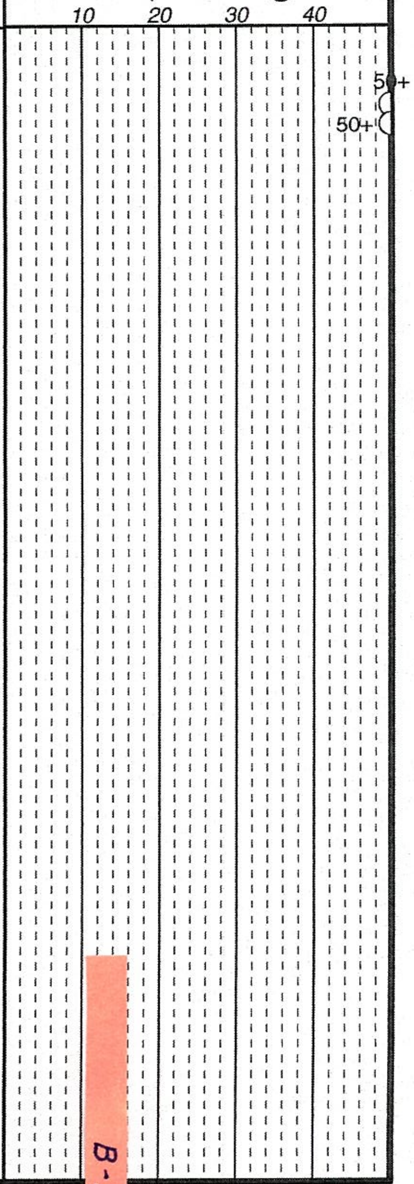
LOG OF: Boring B-35 *Location:* Sta. 40+29.1, 28.3 ft. RT of US 52 Ramp A BL *Date Drilled:* 02/01/07

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.4												
0.4	558.0						Topsoil - 5"						
		18 50/4	10	1			Very dense gray GRAVEL WITH SAND AND SILT (A-2-4); contains sandstone fragments; damp.						
2.0	556.4	50/3	1	2			Severely weathered gray SANDSTONE.						
2.5	555.9	Core 120"	Rec 120"	RQD 85%	R1		Medium hard to hard gray SANDSTONE; fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly bedded, slightly to moderately fractured. @ 7.5'-8.2', contains clay seams. @ 8.4'-8.9', qu = 10,892 psi, Er = 1,616,616 psi. @ 10.2'-10.4', clay seam.						
12.5	545.9						Bottom of Boring - 12.5'						

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

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



B-35

FILE: 0121-3070-03 [6/1/2007 11:40 AM]

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

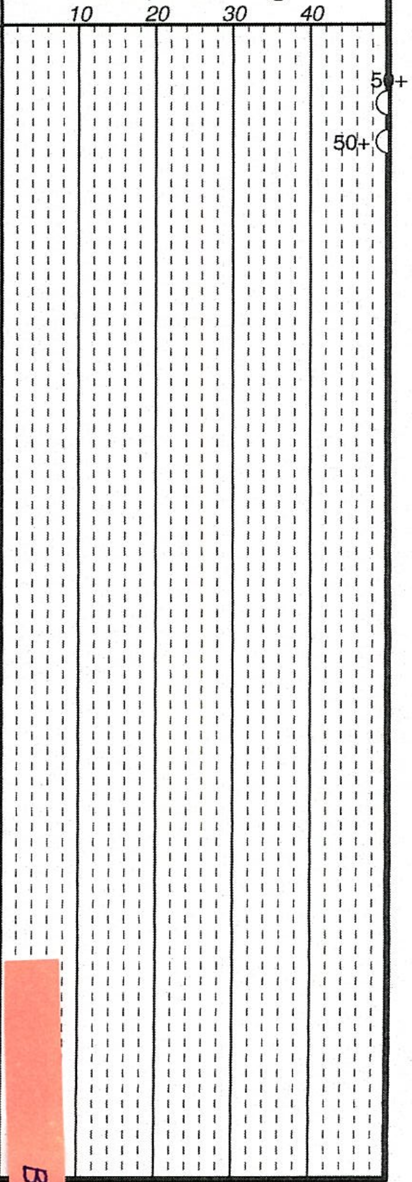
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-36 | *Location:* Sta. 41+53.2, 8.2 ft. LT of US 52 Ramp A BL | *Date Drilled:* 01/31/07

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.5												
0.3	558.2						Topsoil - 4"						
		18 50/5	4	1			Very dense gray SANDY SILT (A-4a), little clay; contains sandstone fragments; dry.						
		50/4	2	2									
3.0	555.5	Core 120"	Rec 110"	RQD 86%	R1		Medium hard to hard gray SANDSTONE; fine grained, slightly weathered, argillaceous, micaceous, thinly bedded, slightly to moderately fractured, iron staining. @ 3.0'-4.1', lost recovery. @ 6.7'-7.5', high angle fracture. @ 8.6'-8.7', clay seam. @ 9.6'-10.0', qu = 908 psi.						
13.0	545.5						Bottom of Boring - 13.0'						

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



B-36

FILE: 0121-3070-03 [6/1/2007 11:40 AM]

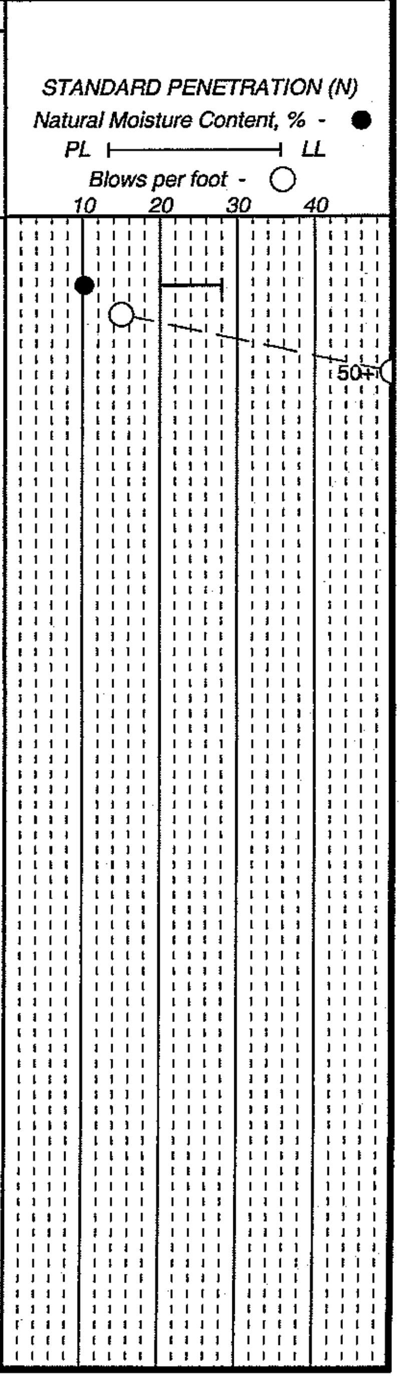
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-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.						
5													
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.						
10													
15													
16.0	543.1						Bottom of Boring - 16.0'						
20													
25													
30													



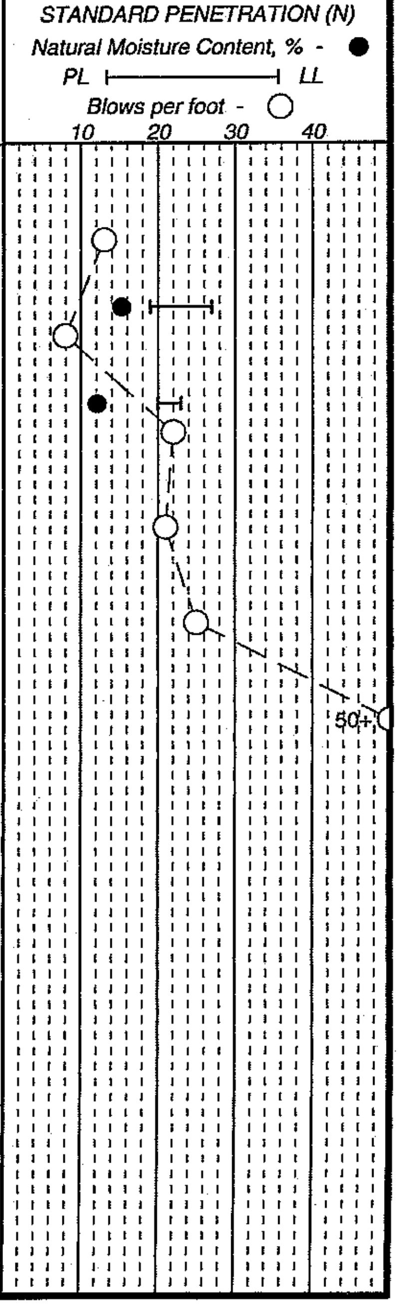
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-76 | *Location:* Sta. 38+59.2, 27.1 ft LT of US 52 Ramp A BL | *Date Drilled:* 3/30/05

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.2	555.1 554.9						Topsoil - 2"						
3.0	552.1	4 6 7	12	1		4.5+	Hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp.						
5		2 3 5	10	2		4.0	Very stiff to hard brown SANDY SILT (A-4a), trace to little clay, little gravel; damp.	15	10	--	11	47	17
8.0	547.1	2 7 15	17	3		–		16	23	--	21	30	10
10		5 9 12	16	4			Medium dense brown SANDY SILT (A-4a), trace clay, little gravel; damp.						
		11 12 13	18	5			@ 11.0', contains sandstone fragments						
14.5	540.6	1 6 50/5	17	6			Severely weathered brown SANDSTONE.						
15.0 20	540.1	Core 120"	Rec 120"	RQD 64%	R1		Hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, medium bedded, moderately fractured. @ 16.4'-16.8', 17.3', 18.4', filled fractures. @ 19.4' gray @ 20.9'-21.3', fractured. @ 19.6', qu = 11,036 psi.						
25.0	530.1						Bottom of Boring - 25.0'						
30													



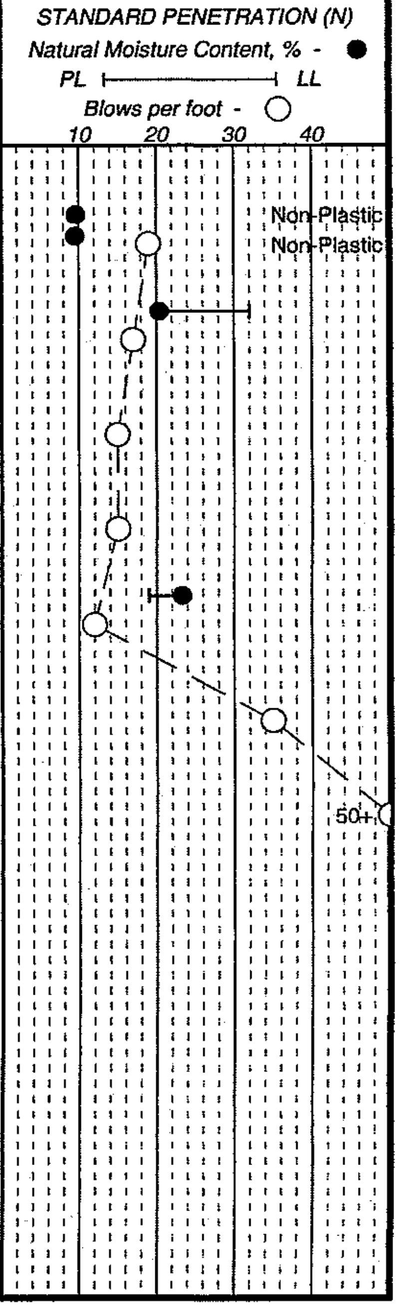
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

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

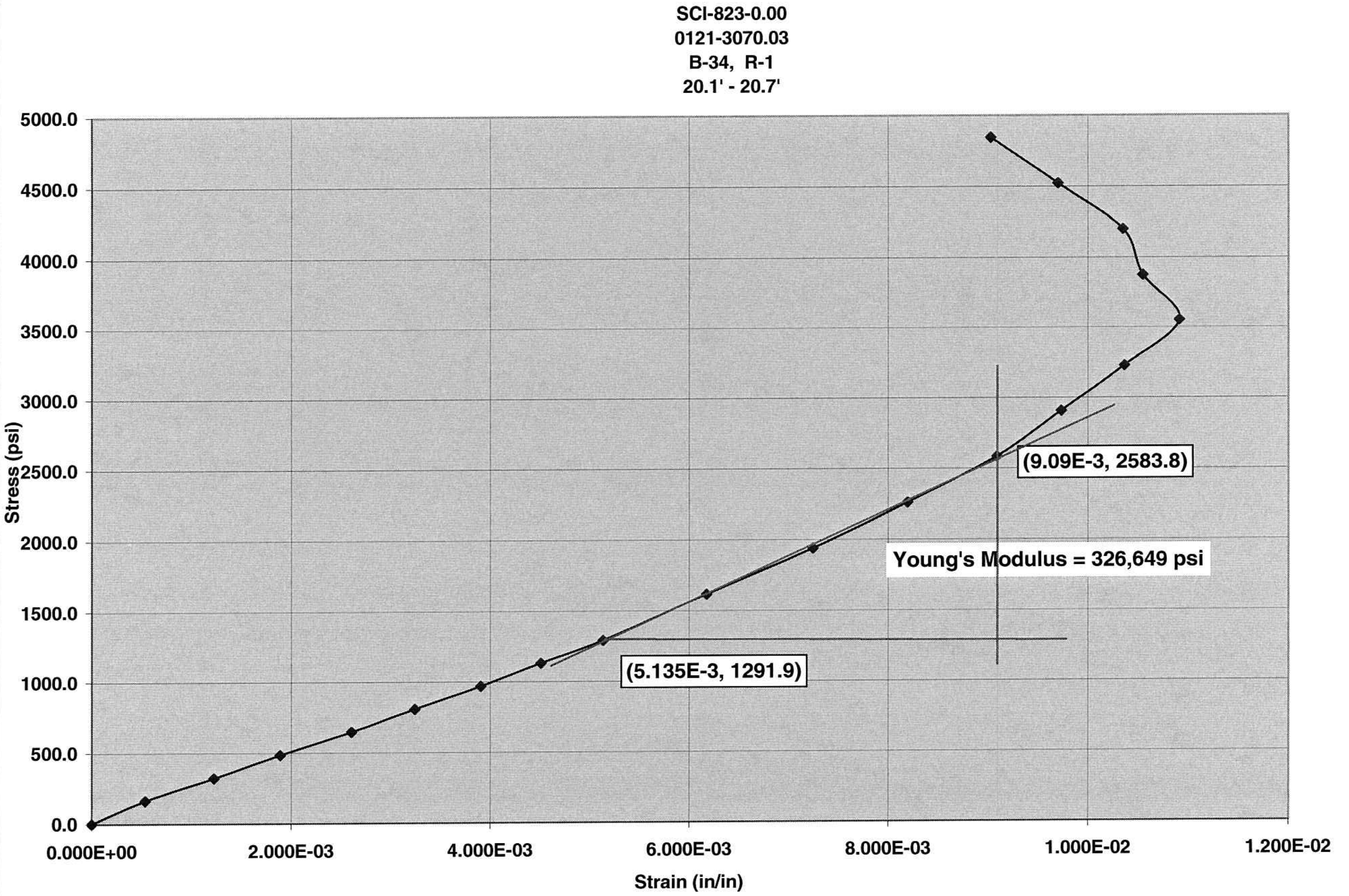
WATER OBSERVATIONS: Water seepage at: 16.0'-17.0'
Water level at completion: 14.0' (prior to coring)
8.0' (inside hollowstem augers)

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						Medium dense gray GRAVEL WITH SAND (A-1-b), trace silty clay; damp.						
1.3	548.5	11 10 9	12	1A				44	28	--	19	9	
				1B			Medium dense brown COARSE AND FINE SAND (A-3a), some silt, little gravel, trace clay; damp.	19	27	--	27	21	6
3.0	546.8	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													
5.5	544.3	10 7 8	10	3		2.0	Stiff to very stiff brown SILT (A-4b), little clay, little fine sand, trace coarse sand; contains sandstone fragments; damp.						
		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													
		11 18 50/3	13	7									
17.0	532.8						Severely weathered brown SANDSTONE.						
17.5	532.3	Core 120"	Rec 120"	RQD 58%	R1		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. @ 22.8', qu = 11,463 psi.						
20													
25													
27.5	522.3						Bottom of Boring - 27.5'						
30													

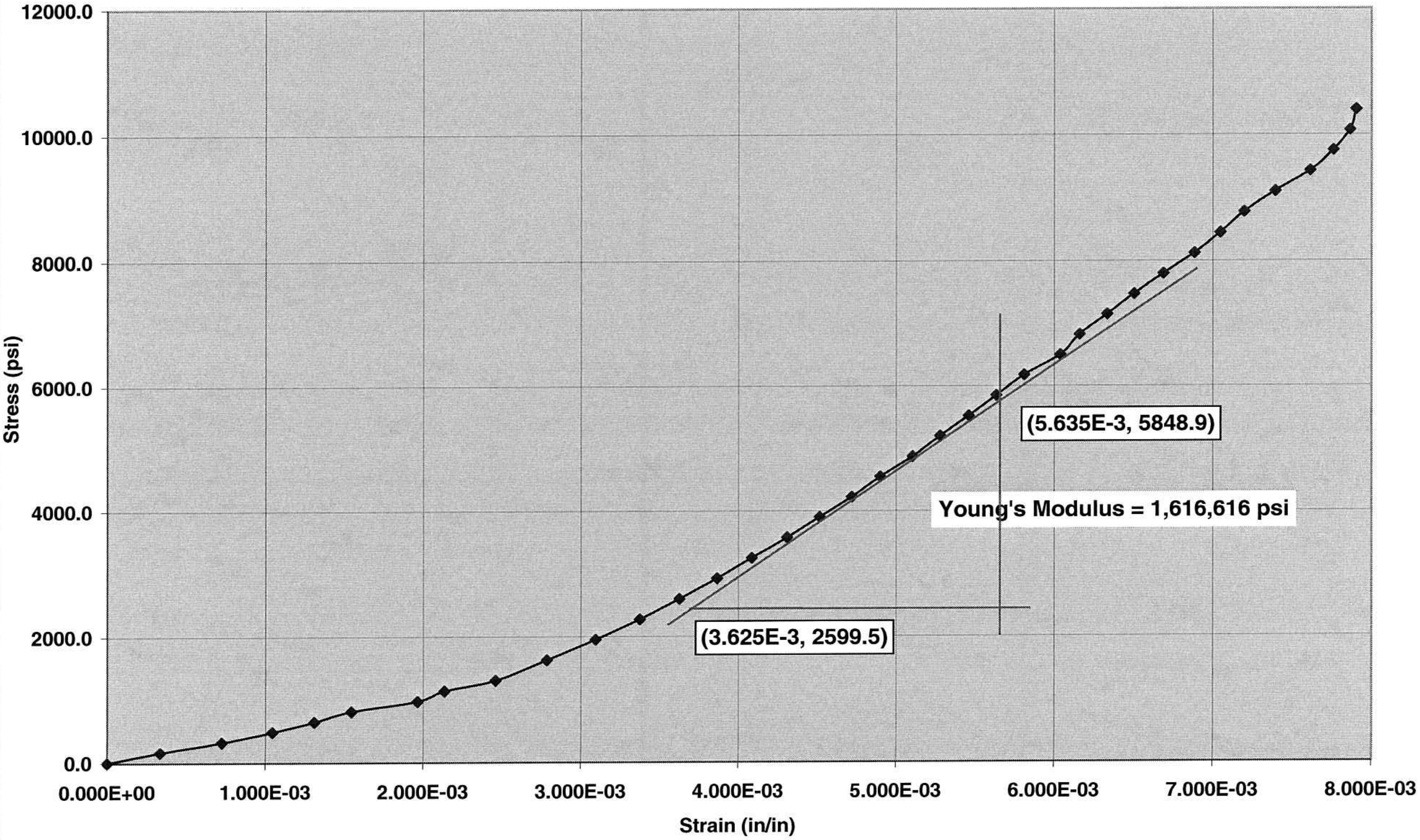


APPENDIX III

Laboratory Test Results



01-307003 B-34 R-1 Chart 1



SCI-823-0.00
0121-3070.03
B-35, R-1
8.4' - 8.9'

01-307003 B-35 R-1 Chart 1

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

DLZ Project No.: 0121-3070.03

Client: TranSystems

Project Name: SCI-823-0.00

Date: 2/21/07

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-33	1	20.8'-21.3'	1.970	1.980	1.978	1.977	4.643	4.642	4.639	4.641	2.347	0.0082441	559.95	149.7	28,510	**9,284**
			1.974	1.982	1.980											
B-34	1	20.1'-20.7'	1.987	1.985	1.987	1.986	4.661	4.659	4.664	4.661	2.347	0.0083552	589.07	155.4	16,890	**5,450**
			1.986	1.987	1.986											
B-35	1	8.4'-8.9'	1.981	1.979	1.983	1.980	4.550	4.554	4.555	4.553	2.299	0.0081118	578.35	157.2	33,550	**10,892**
			1.981	1.978	1.980											
B-36	1	9.6'-10.0'	1.982	1.986	1.983	1.981	3.717	3.719	3.720	3.719	1.877	0.0066309	476.21	158.3	28,770	**9,260**
			1.979	1.980	1.977											

DLZ

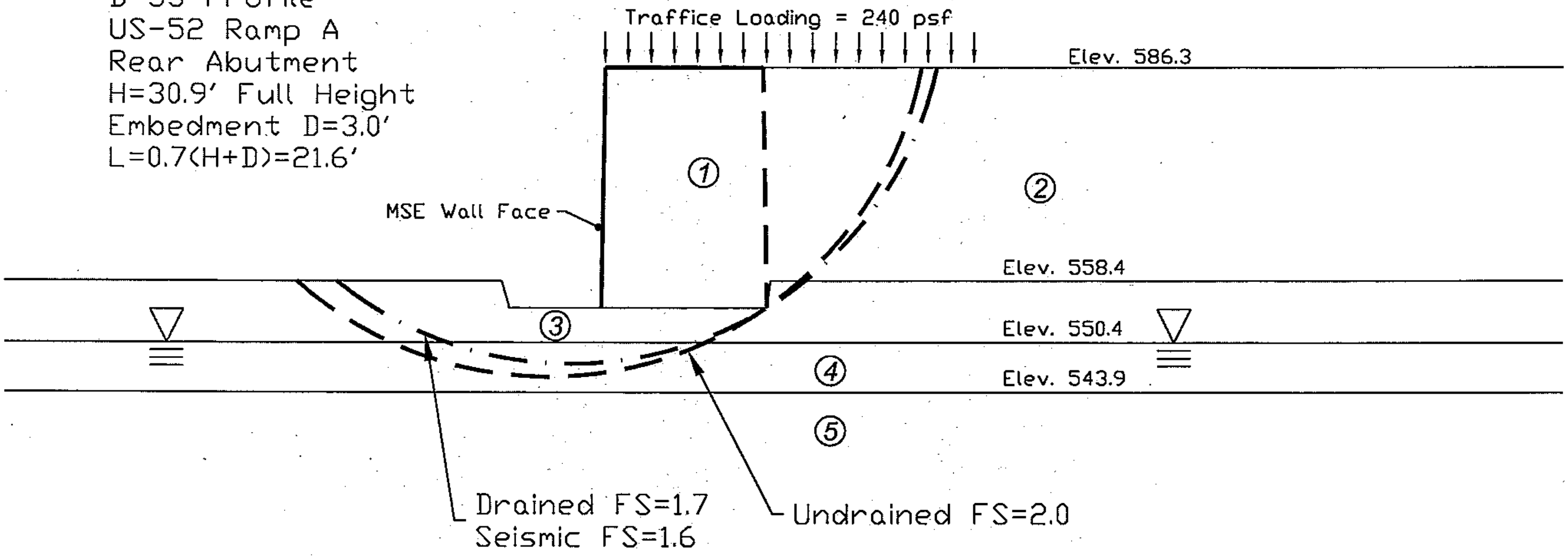
Engineers * Architects * Scientists

APPENDIX IV

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

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	Φ (deg)	C' (psf)	Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Very Stiff	Sandy Silt	1750	0	0	29	120
Material 4	M. Dense	Gravel	0	30	0	30	125
Material 5		BEDROCK	5000	45	5000	45	150

MSE Stability Analysis
B-33 Profile
US-52 Ramp A
Rear Abutment
H=30.9' Full Height
Embedment D=3.0'
L=0.7(H+D)=21.6'



Sheet 1 of 37

US-52 Ramp A over Ohio River Road
BASED ON BORING B-33 PROFILE

MSE STABILITY ANALYSIS

SCI-823-0. 00

PROJECT NO. 0121-3070. 03 | CALC: SJR | DATE 5/29/07

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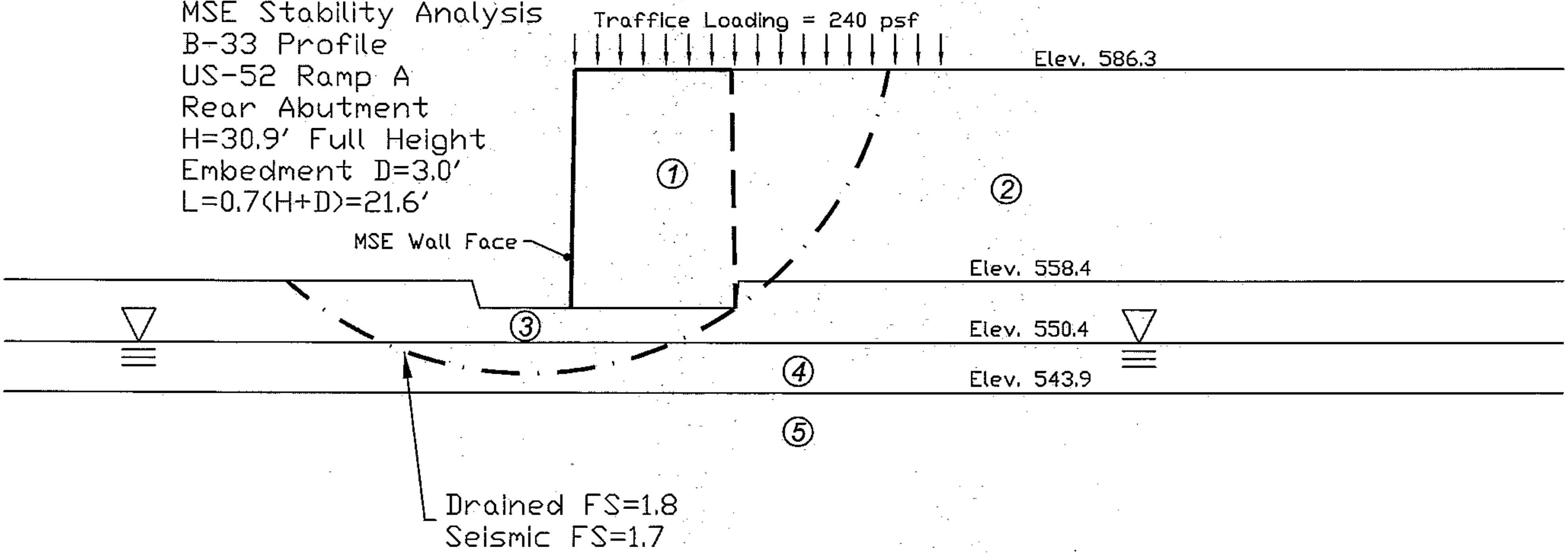
			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	φ (deg)	C' (psf)	φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Granular Fill	0	34	0	34	120
Material 4	M. Dense	Gravel	0	30	0	30	125
Material 5		BEDROCK	5000	45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

MSE Stability Analysis
B-33 Profile
US-52 Ramp A
Rear Abutment
H=30.9' Full Height
Embedment D=3.0'
L=0.7(H+D)=21.6'



Sheet 2 of 37

US-52 Ramp A over Ohio River Road
BASED ON BORING B-33 PROFILE
WITH UNDERCUT

MSE STABILITY ANALYSIS

SCI-823-0. 00

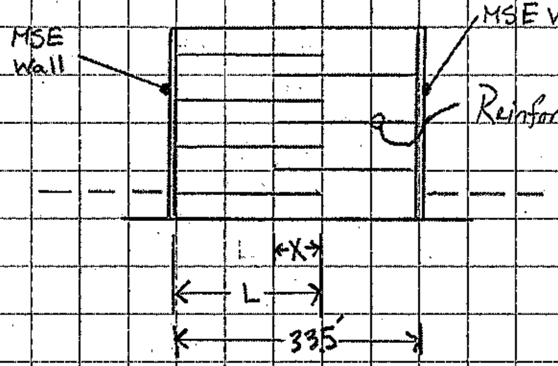
PROJECT NO. 0121-3070. 03 | CALC: SJR | DATE 5/29/07

CLIENT TranSystems Corp
PROJECT SCI-823 Portsmouth Bypass
SUBJECT MSE Wall Bearing Capacity
Modification of Ka

PROJECT NO. 0121-3070.03
SHEET NO. 3 OF 37
COMP. BY S/R DATE 5-29-07
CHECKED BY DAA DATE 6/1/07

* From Revised Profile dated 5-17-07

1.) Analyze Rear Abutment MSE wall as back-to-back walls.
* 90° Turn back from Abutment Wall face.
* 33.5' – Ramp A width as per most current cross-sections
* H = 27.9' + 3.0' = 30.9' Including Embedment



X = Overlap of Reinforcing Straps
L = 0.7·H = 0.7(30.9) = 21.6'

X = (2·L) – 33.5 = 9.7'

Overlap as a ratio of hieght

$$\frac{9.7'}{30.9'} = 0.3 \cdot H$$

IF overlap ≥ 0.3 H → Use $K_a = 0$ for external stability.

↳ See attached text ; Ref FHWA-NHI-00-043

2.) Analyze Forward Abutment MSE wall as back-to-back walls
* 90° Turn back from Abutment wall face
* 33.5' – Ramp A width
* H = 34.1 + 3.0' = 37.1' Including Embedment

X = Overlap of Reinforcing Straps
L = 0.7H = 0.7 (37.1) = 26.0'

$$X = (2 \cdot L) - 33.5' = 18.5' \Rightarrow \frac{18.5}{37.1} = 0.5 \cdot H \text{ overlap}$$

IF overlap ≥ 0.3·H → Use $K_a = 0$ for external stability.

↳ See attached text ; Ref FHWA-NHI-00-043

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:

  $$D = H_1 \tan(45° - \varphi/2) \qquad (55)$$

  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:

  $$D > H_1 \tan(45° - \varphi/2) \approx 0.5\ H_1 \qquad (56)$$

  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.

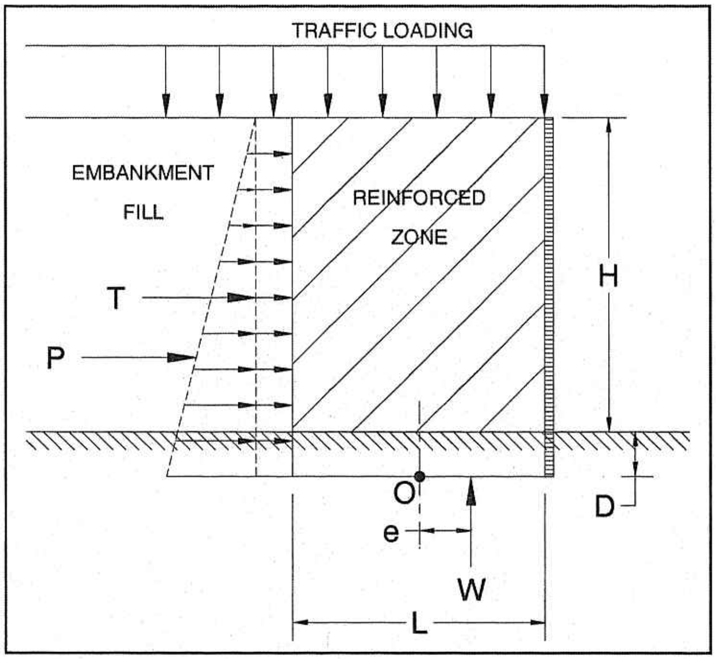
DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	5 OF 37
Item	MSE Wall Bearing Capacity	COMP. BY	SJK DATE 5-29-07
Rear Abutment, Based upon boring B-33		CHECKED BY	DAA DATE 6/1/07

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	=	1750	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	=	21.63	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	=	30.9	ft	
H	=	27.9	ft	Height of wall
Ka	=	**0.00**		**Ka = 0.0*K, Due to overlap**
Γ Pa	=	10.3	ft	Moment arm
Γ Wt	=	15.45	ft	Moment arm
B'	=	21.63	ft	
γ'	=	57.6	pcf	
W_t		5,191	lb/ft of wall	Weight from traffic
W_{mse}	=	80,204	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 3,948 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 9,168 psf

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

q_{ALL} = 3,667 psf

Factor of Safety = **2.32** **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} = 14,889 psf

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

q_{ALL} = 5,956 psf

Factor of Safety = **3.77** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 0.00 ft

Kern

e < L/6 = 3.61 ft

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	6 OF 37
Item	MSE Wall Stability	COMP. BY	SAK DATE 5-29-07
Rear Abutment, Based upon boring B-33		CHECKED BY	DAA DATE 6-1-07

Analysis assumes overlapping soil reinforcement

Based upon existing foundation soils

STABILITY OF MSE WALL

Assumptions:

1. Estimated height of embankment; H=27.9'
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

$H+D$ = 30.9 feet

γ_{mse} = 120 pcf

L = 21.63 feet

L factor = 0.70

ϕ = 30 deg

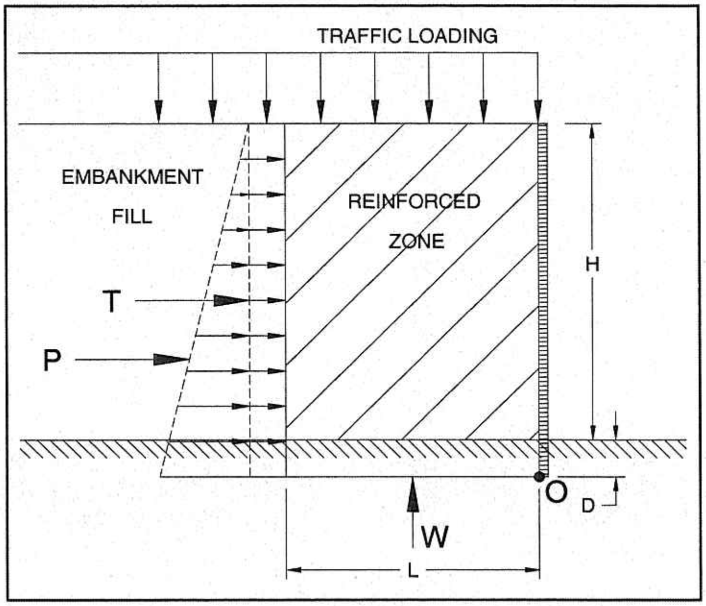
Foundational Soil Properties

c	=	1750	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



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

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

P_a = 0 lbs per foot of wall

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

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

P_r = 29,675 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 37,853 lbs per foot of wall

Use Drained Value

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

Calculated FS = ###### $\quad$ Required FS = 1.50

FS = ∞ Due to $K_a = 0.0$

FS > 1.5

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).

* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 867,407 lb-ft

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

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

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

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

Calculated FS = ###### $\quad$ Required FS = 2.00

FS = ∞

FS > 2.0 Due to $K_a = 0.0$

Resistance Against Overturning is **OK**

DLZ SUBJECT

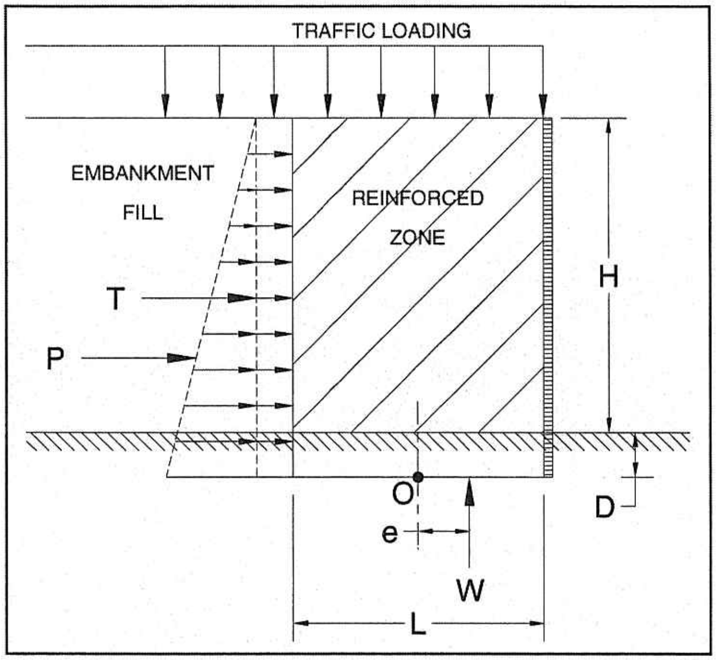
Client TranSystems Corp
Project SCI-823 Portsmouth Bypass
Item MSE Wall Bearing Capacity
Rear Abutment, Based upon boring B-33

JOB NUMBER 0121-3070.03
SHEET NO. 7 OF 37
COMP. BY SJK DATE 5-29-07
CHECKED BY DAA DATE 6/1/07

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}



Effective Bearing Pressure

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

σ_V = 3,948 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 17,133 psf

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

q_{ALL} = 6,853 psf

Factor of Safety = **4.34** **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} = 17,133 psf

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

q_{ALL} = 6,853 psf

Factor of Safety = **4.34** **OK**

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

Use $\phi' = 30°$ for soil below undercut.
Ref: AASHTO Std Spec for Highway bridges, 5.8.2

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	21.63	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	=	30.9	ft	
H	=	27.9	ft	Height of wall
Ka	=	**0.00**		**Ka = 0.0*K, Due to overlap**
Γ Pa	=	10.3	ft	Moment arm
Γ Wt	=	15.45	ft	Moment arm
B'	=	21.63	ft	
γ'	=	57.6	pcf	
W_t		5,191	lb/ft of wall	Weight from traffic
W_{mse}	=	80,204	lb/ft of wall	Weight from MSE wall

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 = 3.61 ft

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	8 OF 37
Item	MSE Wall Stability	COMP. BY	SJK DATE 5-29-07
Rear Abutment, Based upon boring B-33		CHECKED BY	DAA DATE 6/1/07

Analysis assumes overlapping soil reinforcement

Based upon undercut of existing foundation soils

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=27.9'
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

$H+D$ = 30.9 feet
γ_{mse} = 120 pcf
L = 21.63 feet
L factor = 0.70
ϕ = 30 deg

Foundational Soil Properties

c = 0 psf Cohesion
ϕ' = 30 deg Friction angle
ω_T = 240 psf Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

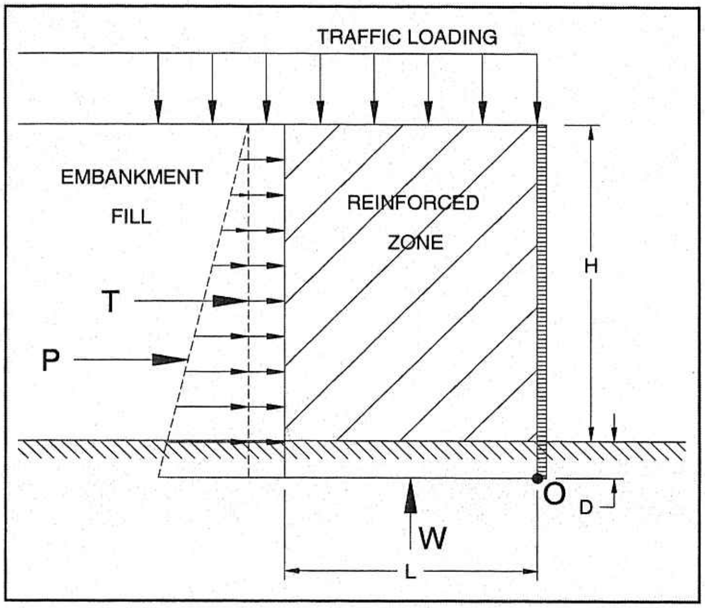
P_a = 0 lbs per foot of wall

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

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

P_r = 31,280 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**

$FS = \infty$
$FS > 1.5$ Due to $K_a = 0.0$

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 867,407 lb-ft

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

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

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

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

Calculated FS = ######

Required FS = 2.00

Resistance Against Overturning is **OK**

$FS = \infty$
$FS > 2.0$ Due to $K_a = 0.0$

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	9 OF 37
Item	MSE Wall Stability	COMP. BY	SJK DATE 5-29-07
Forward Abutment Wall, Based on B-36		CHECKED BY	DAA DATE 6/1/07

Analysis assumes overlapping soil reinforcement

Based upon MSE wall leveling pad founded on bedrock

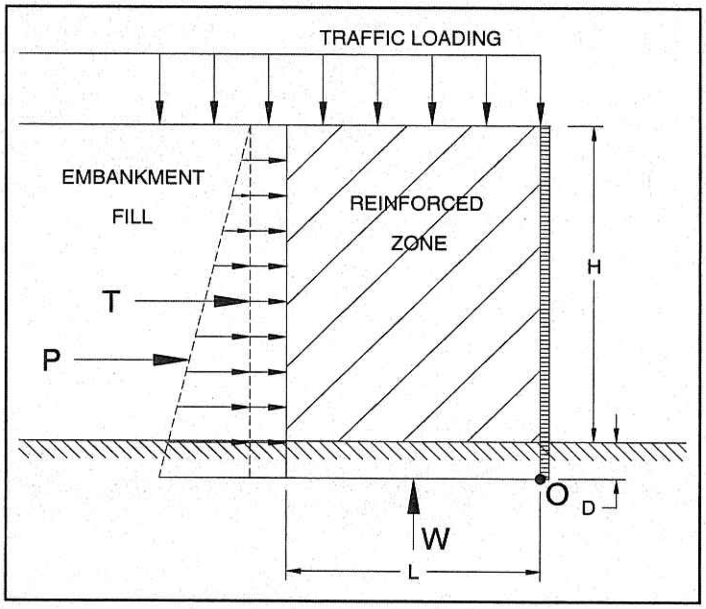
STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=37.1'
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			Foundational Soil Properties			
H+D =	37.1	feet	c =	0	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	34	deg	Friction angle
L =	26.044	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 = 0.45

P_r = 52,177 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**

$FS = \infty$
$FS > 1.5$ Due to $K_a = 0.0$

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 1,509,897 lb-ft

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

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

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

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

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

Resistance Against Overturning is **OK**

$FS = \infty$
$FS > 2.0$ Due to $K_a = 0.0$

DLZ

SUBJECT

Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	10 OF 37
Item	MSE Wall Stability	COMP. BY	SJK DATE 5-29-07
Forward Abutment Wall, Based on B-36		CHECKED BY	DAA DATE 6/1/07

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=37.1'

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

$H+D$ = 37.1 feet

γ_{mse} = 120 pcf

L = 26.044 feet

L factor = 0.70

ϕ = 30 deg

Foundational Soil Properties

c = 0 psf Cohesion

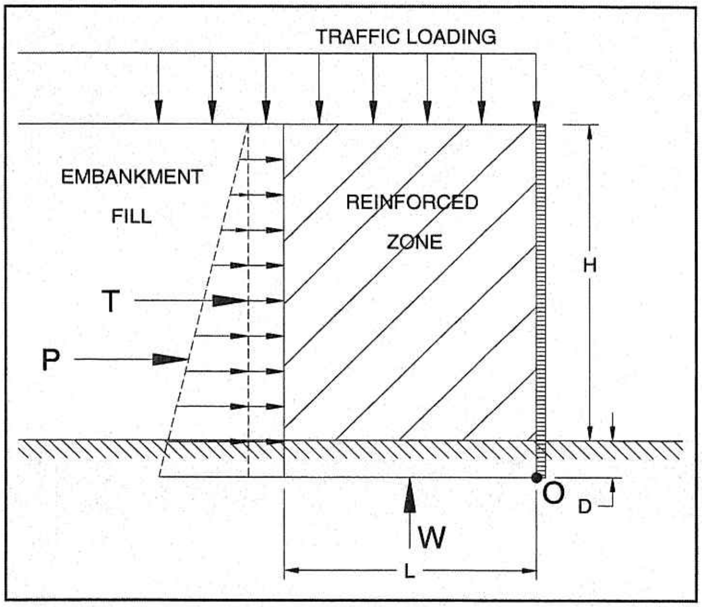
ϕ' = 34 deg Friction angle

ω_T = 240 psf Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



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

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

P_a = 30,191 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 = 52,177 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 = **1.73**

Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).

* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 1,509,897 lb-ft

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

$\Sigma M_{overturning}$ = 391,534 lb-ft

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

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

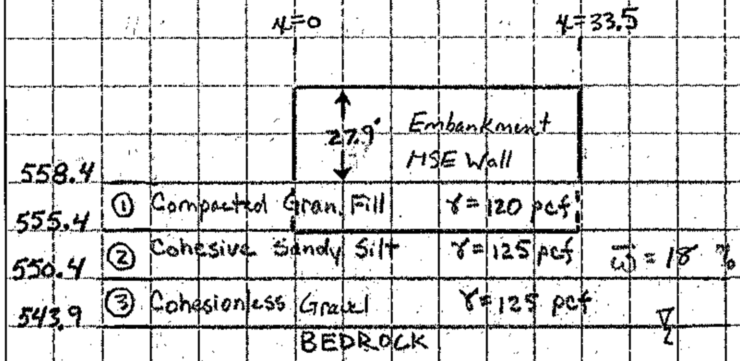
Calculated FS = **3.86**

Required FS = 2.00

Resistance Against Overturning is **OK**

CLIENT TranSystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Consolidation Parameters
US-52 Ramp A over Ohio River Road
PROJECT NO. 0121-3070.03
SHEET NO. 11 OF 37
COMP. BY SJK DATE 4-11-07
CHECKED BY DAA DATE 6/1/07

Most Critical Soil Profile was encountered in boring B-33.
At B-33, maximum MSE wall height is approximately 29.5'.



* Assume 90° Turnback
* Assume Soils are normally Consolidated

} Assume Incompressible
$\bar{\omega} = 18\ \%$ * } $C_c = 0.18$ $e_o = 0.56$ [FHWA-NHI-00-045]
* } $\bar{N} \approx 75$ $\bar{N}' \approx 75 \rightarrow C' = 300$ [FHWA]
↳ [$C_c = 0.007$, $e_o = 1.0$] - See Calculation Below.

*Consolidation Parameters are estimated from FHWA NHI-00-045 for; cohesive soils based upon moisture and, cohesionless soils based upon an average SPT N-Values.

The computer program EMBANK requires inputs for C_c, C_r and e_o. To evaluate the settlement of the granular layers we must calculate equivalent consolidation parameters from C'.

$$\frac{1}{C'} = \frac{C_c}{1+e_o}$$ Say $e_o = 1.0$ in this case.

$$\frac{1}{C'} = \frac{C_c}{1+1.0} \longrightarrow C' = \frac{2.0}{C_c} \qquad \boxed{C_c = \frac{2}{C'}}$$

When $C' = 300$, $C_c = \frac{2}{300} = 0.006\bar{6}$ use 0.007

---

* Soil not Saturated

$$e_o = \frac{G_s\ \gamma_w}{\gamma_d} - 1 \qquad \text{where } \gamma_d = \frac{\gamma}{1+w} = \frac{125\text{ pcf}}{1+.18} = 106\text{ pcf}$$

$$e_o = \frac{(2.65)(62.4)}{106} - 1 \qquad e_o = 0.56$$

Differential Settlement $$DS = \frac{\Delta\delta}{L} = \frac{(3.84" - 2.52")(1/12")}{16.75'} = 0.007$$

$$DS = 0.7\% < 1.0\% \longrightarrow \boxed{OK}$$

Sheet 12 of 37 SJK 5-31-07
DAA 6/1/07

US 52 Ramp A MSE wall Settlement Analysis

```
ÚÄÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿
³            INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION            ³
³                                                                               ³
³ Project Name   : SCI-823 US-52 Ramp A Client          : TranSystems Corp      ³
³ File Name      : 52A                  Project Manager : Nix                   ³
³ Date           : 04/12/07             Computed by     : sjr                   ³
³                                                                               ³
³                                                                               ³
³                         Settlement for X-Direction                            ³
³                                                                               ³
³ Embank. slope, x direc. =     0.10 (ft)  Height of fill H     =   27.90 (ft)  ³
³                y direc. =     0.10 (ft)  Unit weight of fill =  120.00 (pcf)  ³
³ Embankment top width    =    33.50 (ft)  p load/unit area     = 3348.00 (psf) ³
³ Embankment bottom width =    33.70 (ft)  Foundation Elev.     =  558.40 (ft)  ³
³ Ground Surface Elev.    =   558.40 (ft)                                       ³
³ Water table Elev.       =   543.90 (ft)  Unit weight of Wat. =   62.40 (pcf)  ³
³                                                                               ³
³                                                                               ³
³          LAYER               COEFFICIENT           UNIT    SPECIFIC   VOID    ³
³      N§.  TYPE   THICK.  COMP.  RECOMP.  SWELL.   WEIGHT   GRAVITY   RATIO    ³
³                   (ft)                            (pcf)                       ³
³                                                                               ³
³      1  INCOMP.  3.0    -----   -----   -----     120.00    ----     ----     ³
³      2   COMP.   5.0    0.180   0.018   0.000     125.00    2.65     0.56     ³
³      3   COMP.   6.5    0.007   0.000   0.000     125.00    2.65     1.00     ³
³                                                                               ³
³            SUBLAYER                         SOIL STRESSES                     ³
³     N§.     THICK.        ELEV.               INITIAL        MAX.PAST PRESS.  ³
³              (ft)         (ft)                 (psf)             (psf)        ³
³                                                                               ³
³     1  INCOMP.                                                                ³
³     2        5.00        552.90              672.50            672.50         ³
³     3        6.50        547.15             1391.25           1391.25         ³
³                                                                               ³
³                                                                               ³
³            X =     0.10✓   X =      6.80     X =     13.50✓  X =     20.20    ³
³ Layer   Stress   Sett.    Stress   Sett.    Stress   Sett.   Stress   Sett.   ³
³          (psf)   (in.)     (psf)   (in.)     (psf)   (in.)    (psf)   (in.)   ³
³                                                                               ³
³   1   INCOMP.  INCOMP.  INCOMP.  INCOMP.                                      ³
³   2    855.03    2.47   1586.40    3.64   1666.49    3.75   1666.49    3.75   ³
³   3    834.71    0.06   1346.57    0.08   1529.55    0.09   1529.55    0.09   ³
³                ------            ------            ------            ------   ³
³                  2.52              3.72              3.84              3.84   ³
³                                                                               ³
³                                                                               ³
³            X =    26.90    X =     33.60                                      ³
³ Layer   Stress   Sett.    Stress   Sett.                                      ³
³          (psf)   (in.)     (psf)   (in.)                                      ³
³                                                                               ³
³   1   INCOMP.  INCOMP.                                                        ³
³   2   1586.40    3.64    855.03    2.47                                       ³
³   3   1346.57    0.08    834.71    0.06                                       ³
³                ------            ------                                       ³
³                  3.72              2.52                                       ³
³                                                                               ³
³                                                                               ³
³                                                                               ³
ÀÄÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

Use ℄ of Ramp
y = 16.85'

CLIENT Tran Systems Corp.
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Allowable Uplift in driven piles
Pier 2 - Left
PROJECT NO. 0121-3070.03
SHEET NO. 13 OF 37
COMP. BY SJK DATE 5-29-07
CHECKED BY DAA DATE 6/1/07

* From Driven analysis ;

Ultimate skin friction = 67.3 kips

Allowable uplift in driven piles = $\frac{67.3^{k}}{3.0}$ = 22.4 kips per pile.

If uplift forces are present, use 20^{k} allowable uplift in the pier 2, left foundation only.

DRIVEN 1.0

Sheet 14 of 37

GENERAL PROJECT INFORMATION

SJK 05-31-07
DAA 7/1/07

Filename:
Project Name: SCI-823
Project Date: 05/29/2007
Project Client: TranSystems Corp.
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: H Pile - HP12X53
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	13.00 ft
	- Driving/Restrike	13.00 ft
	- Ultimate:	13.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	14.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel

Sheet 15 of 37
SAK 5-31-07 / DAA 6/1/07

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	1165.00 psf	0.05 Kips
9.01 ft	Cohesive	N/A	N/A	1165.00 psf	41.68 Kips
13.99 ft	Cohesive	N/A	N/A	1210.94 psf	67.27 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	17.74 Kips
13.99 ft	Cohesive	N/A	N/A	N/A	17.74 Kips

ULTIMATE - SUMMARY OF CAPACITIES

sheet 16 of 37
SJK 05-31-07/
DAA 6/1/07

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.05 Kips	17.74 Kips	17.78 Kips
9.01 ft	41.68 Kips	17.74 Kips	59.42 Kips
13.99 ft	67.27 Kips	17.74 Kips	85.01 Kips

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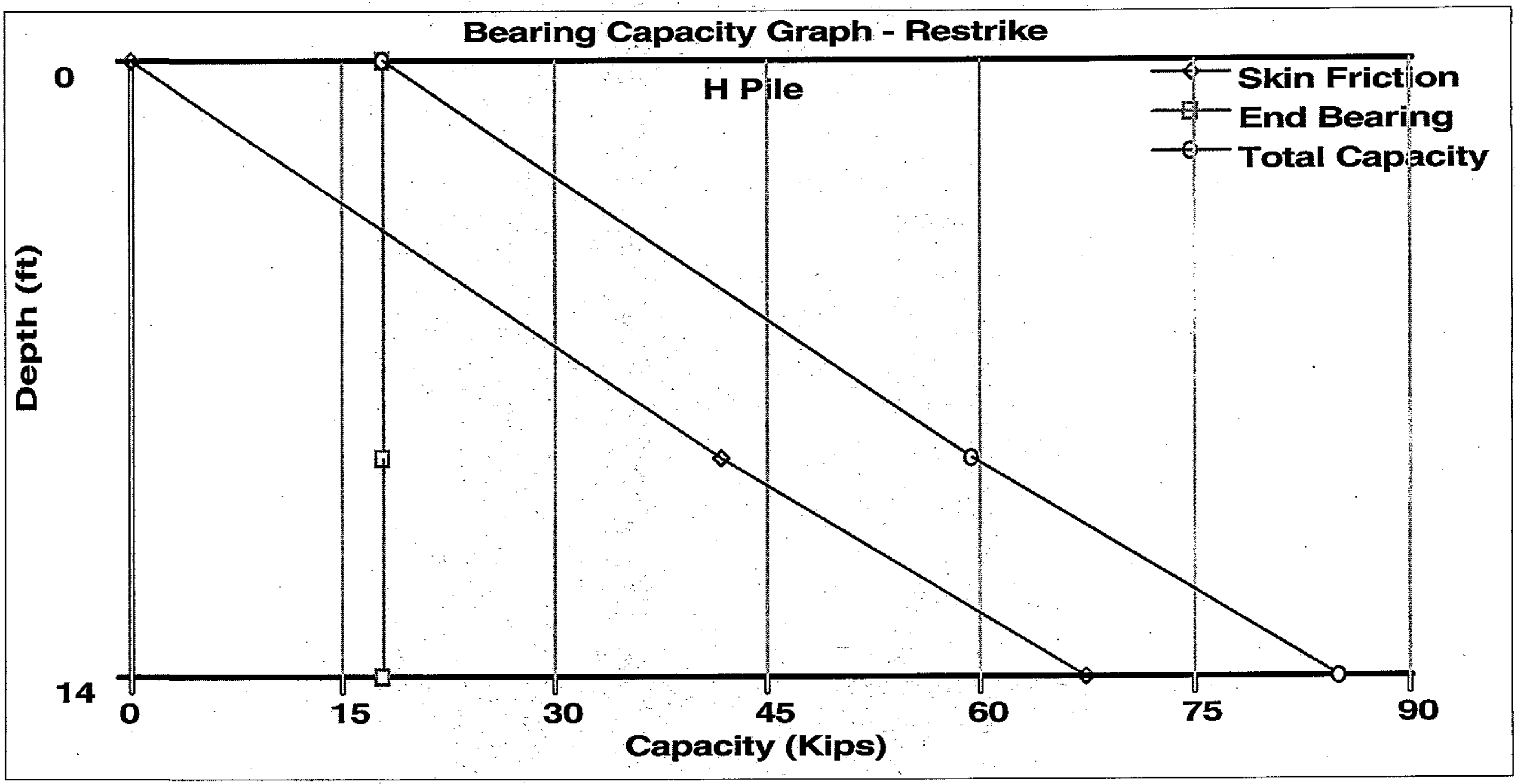

Sheet 17 of 37
SAK 05-31-07
DAA 6/1/07

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ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Transystems Corp/ODOT D-9
PROJECT SCT-823 Portsmouth Bypass
SUBJECT Drilled shaft End Bearing & Skin Friction
US 52 Ramp A

PROJECT NO. 0121-3070-03
SHEET NO. 18 OF 37
COMP. BY SAK DATE 2/27/07
CHECKED BY DAA DATE 4-20-07

*From Rock core testing $q_u = 5,450$ psi (minimum value) = 37.58 MPa

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

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

FHWA-IF-99-025
Eq. 11.6

$$= 4.83\left[37.58\right]^{0.51}$$

$$= 30.7 \text{ MPa}$$

$$= 640 \text{ Ksf}$$

$$q_a = \frac{q_{max}}{FS} = \frac{640}{3} = 213.5 \text{ Ksf}$$

Say $q_a = 80$ Ksf

For Design Use $\underline{q_a = 40 \text{ Ksf}}$ * Reduction for Argillaceous Rock

2) Side Friction: (smooth Rock socket)

$$f_{max} = 0.65\, P_a \left[\frac{q_u}{P_a}\right]^{0.5} \leq 0.65\, P_a \left[\frac{f'_c}{P_a}\right]^{0.5}$$

$$= 0.65\,(14.7)\left(\frac{5450}{14.7}\right)^{0.5} \leq 0.65 * 14.7\left(\frac{4500}{14.7}\right)^{0.5}$$

$$184 \text{ psi} \leq 167 \text{ psi}$$

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

$$f_{all} = \frac{167}{3} = 55.7 \text{ psi} = 8,016 \text{ psf}$$

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

For Design Use $f_{all} = 3,750$ psf *Reduction for Argillaceous Rock

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CLIENT TranSystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT US 52 Ramp A - Pier 2
Laterally Loaded Drilled Shafts
PROJECT NO. 0121-3070.03
SHEET NO. 19 OF 37
COMP. BY SJK DATE 5-29-07
CHECKED BY EWT DATE 6-1-07

*Loads have been provided from TranSystems based upon preliminary designs. → Service ONLY

Factored Lateral Load = 27.02^{k} AASHTO Case III OR VI loading.

↳ Load applied at top of column/drilled shaft for pier 2.

Assumed that lateral loading applied at elevation 580, 22' above ground surface. Profile encountered in boring B-34 deemed to be most critical. Consequently this profile is assumed for the laterally loaded drilled shaft analyses.

Factored axial load from Case I.

$$P = 1.3\left[(1.67)(191.46^{k}) + (1.0)(684.44^{k})\right] = 1305.4^{k}$$

It is understood that a 54" column is being considered to support the bent for Ramp A, Pier 2.

* From Pier Cap to 1' below the ground surface:

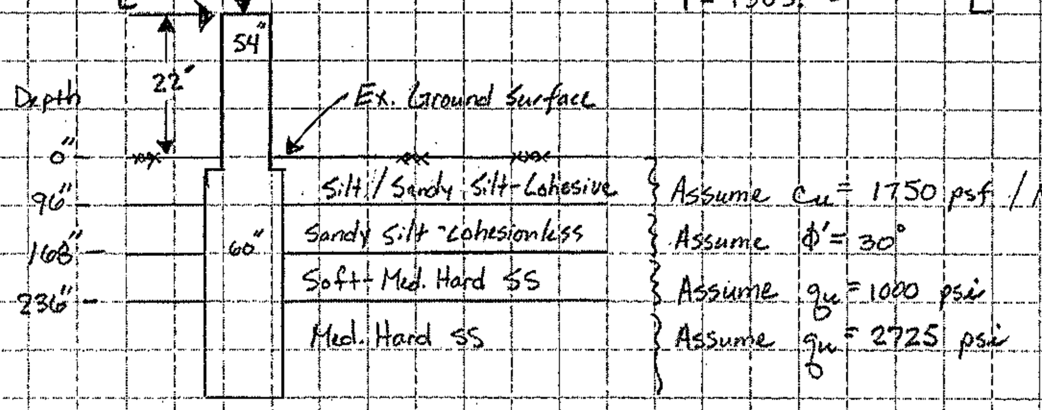
Assume: 54" dia. R.C. Shaft
$\rho = 1.4\%$ 8-#18 bars (OR equivalent)
$f_y = 60$ ksi $f_c' = 4.5$ ksi
$E_s = 29 \times 10^6$ psi $E_c = 3.824 \times 10^6$ psi

* From 1' below ground surface to rock socket

Assume: 60" dia R.C. Shaft
$\rho = 1.7\%$ 12-#18 bars (OR equivalent)
$f_y = 60$ ksi $f_c' = 4.5$ ksi
$E_s = 29 \times 10^6$ psi $E_c = 3.824 \times 10^6$ psi

Profile Assumed for analyses (B-34)

$P = 1305.4^{k}$ $L = 27.02^{k}$



DLZ

ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Transystems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT US52 Ramp A Pier 1
Laterally Loaded Drilled Shafts

PROJECT NO. 0121-3070.03
SHEET NO. 20 OF 37
COMP. BY SAK DATE 5-29-07
CHECKED BY GWT DATE 6-1-07

* Using the afore-mentioned assumptions, it is possible to resist lateral and axial loads using a column / drilled shaft system.

* Column diameter (above ground surface) 54" reinforced concrete. $\rho = 1.4\%$

* Drilled Shaft diameter (below ground surface) 60" r.c. $\rho = 1.7\%$

* 19' Rock socket should be provided to resist lateral loading.

↳ From B-34, 19' Rock socket corresponds to elevation 525.6.

* Pile head deflection (at pier cap) = 0.56"

* Using factored loads, From LPile analysis, we obtain;

$V_{max} = 138.6^{k}$

$M_{max} = 880.3$ k-ft

SAK

Sheet 21 of 37 SAK 5-29-07 EWT 6-1-07

Factored Lateral Loads for drilled shaft analysis **US 52 Ramp A** Pier 1

Assumes loading from Pier 1 Left (most critical for pier structure)

						Loads				
AASHTO Load Case	γ	W	WL	LF	T	**W**	**WL**	**LF**	**T**	**Factored Load**
I	1.3	0.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	0.00
IB	1.3	0.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	0.00
II	1.3	1.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	20.23
III	1.3	0.3	1.0	1.0	0.0	15.56	4.97	11.15	0.00	27.02
IV	1.3	0.0	0.0	0.0	1.0	15.56	4.97	11.15	0.00	0.00
V	1.25	1.0	0.0	0.0	1.0	15.56	4.97	11.15	0.00	20.23
VI	1.25	0.3	1.0	1.0	1.0	15.56	4.97	11.15	0.00	27.02
VII	1.3	0.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	0.00
VIII	1.3	0.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	0.00
IX	1.2	1.0	0.0	0.0	0.0	15.56	4.97	11.15	0.00	20.23

Vertical rxns (kips)

	Pier 1 Left column	Pier 1 RIght column
DL from superstructure	684.44	599.44
LL+I (from superstructure)	191.46	157.34

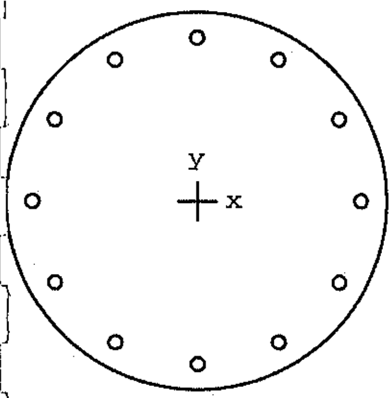
Horizontal (in longitudinal direction of bridge)) rxns (kips)

	Pier 1 Left column	Pier 1 RIght column
Long. Force	11.15	9.77
WL (wind on LL)	4.97	4.35
Wind (W)	15.56	13.62
Thermal (T)	0	0

no thermal force due to strip seal exp. joints at rear and forward abutment
(integral pier will not have to carry any thermal forces due to expansion/contraction)

Remember....these effects are based on precursory analyses of the integral pier for Ramp A (2-span option)

Sheet 23 of 37 SAR 5-31-07
EWT 6-1-07



DS - Socket
60 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Design

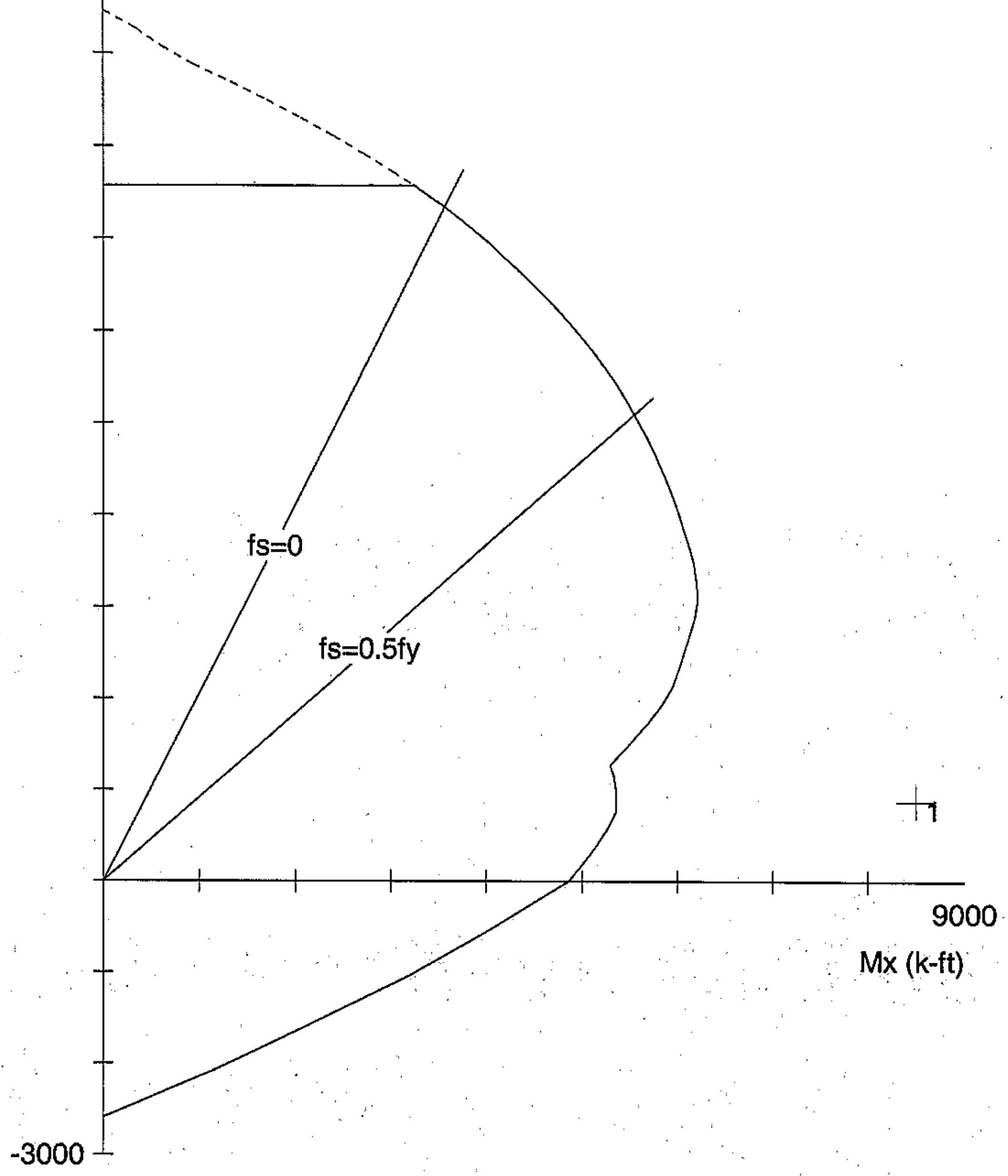
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 05/31/07

Time: 16:08:55



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

File: C:\PROGRA~1\PCACOL\60INCHA.COL

Project:

Column: Engineer:

f'c = 4.5 ksi fy = 60 ksi Ag = 2827.43 in^2 12 #18 bars

Ec = 3824 ksi Es = 29000 ksi As = 48.00 in^2 Rho = 1.70%

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 = 11.14 in Clear cover = 3.00 in

Confinement: Tied phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

Sheet 24 of 37 SJR 5-31-07
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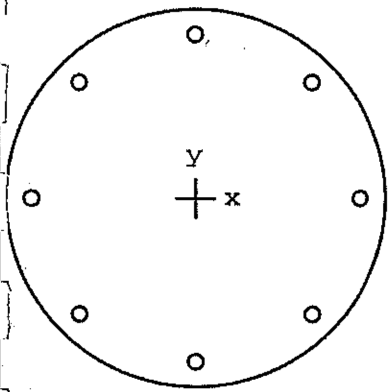


Column

54 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Design

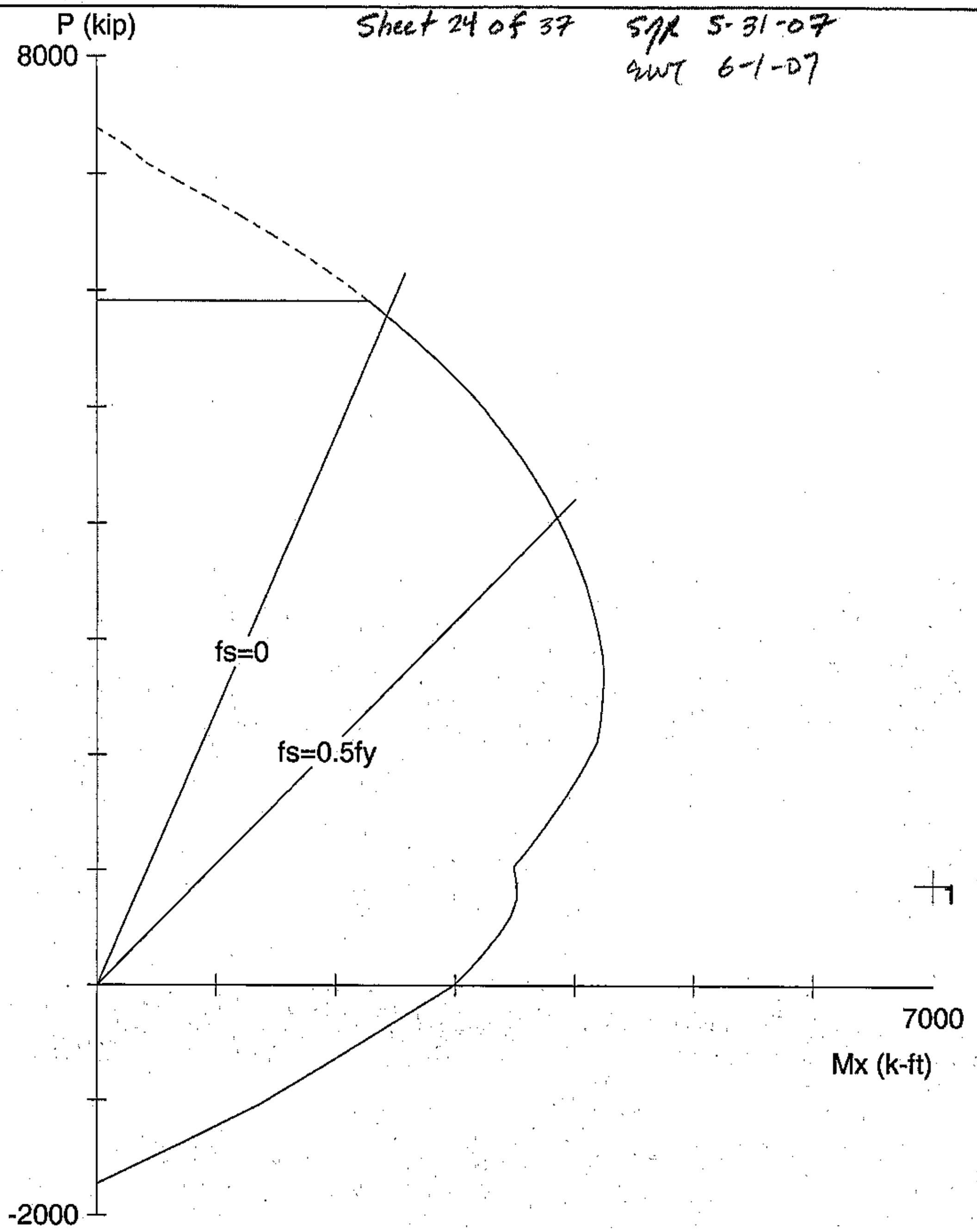
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 05/31/07

Time: 16:07:02



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

File: C:\PROGRA~1\PCACOL\54INCHA.COL

Project:

Column: Engineer:

f'c = 4.5 ksi fy = 60 ksi Ag = 2290.22 in^2 8 #18 bars

Ec = 3824 ksi Es = 29000 ksi As = 32.00 in^2 Rho = 1.40%

fc = 3.825 ksi e_rup = Infinity Xo = 0.00 in Ix = 417393 in^4

e_u = 0.003 in/in Yo = 0.00 in Iy = 417393 in^4

Beta1 = 0.825 Clear spacing = 15.63 in Clear cover = 2.50 in

Confinement: Tied phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

Sheet 25 of 37 SJR 5-31-07
SWT 6-1-07

```
===============================================================================

                LPILE Plus for Windows, Version 5.0 (5.0.5)

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

                    (c) Copyright ENSOFT, Inc., 1985-2004
                            All Rights Reserved

===============================================================================


This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations:       M:\proj\0121\3070.03\Stability Analyses\MSE Walls\US
52\Ramp A\Joint-Final\rear abutment\
Name of input data file:      Ramp A Lat Analysis_Prelim Design.lpd
Name of output file:          Ramp A Lat Analysis_Prelim Design.lpo
Name of plot output file:     Ramp A Lat Analysis_Prelim Design.lpp
Name of runtime file:         Ramp A Lat Analysis_Prelim Design.lpr


-------------------------------------------------------------------------------
                          Time and Date of Analysis
-------------------------------------------------------------------------------

            Date:  May 31, 2007     Time:  17:42:41


-------------------------------------------------------------------------------
                                Problem Title
-------------------------------------------------------------------------------

New LPILE Plus 5.0 Data File


-------------------------------------------------------------------------------
                               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:
```

Sheet 26 of 37 SMR 5-31-07
QWT 6-1-07

```
- 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                                    =     660.00 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         Pile       Moment of       Pile       Modulus of
           X         Diameter      Inertia        Area       Elasticity
           in           in          in**4         Sq.in      lbs/Sq.in
-----  ---------  -----------  -----------  ----------  -----------
  1       0.0000   54.00000000  417393.0000   2290.0000     3823700.
  2     276.0000   54.00000000  417393.0000   2290.0000     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  4 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 =     360.000 in

Layer  2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =     360.000 in
Distance from top of pile to bottom of layer =     432.000 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  3 is strong rock (vuggy limestone)
Distance from top of pile to top of layer    =     432.000 in
Distance from top of pile to bottom of layer =     500.400 in

Layer  4 is strong rock (vuggy limestone)
Distance from top of pile to top of layer    =     500.400 in
Distance from top of pile to bottom of layer =    1000.000 in
```

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GWT 6-1-07

```
(Depth of lowest layer extends  340.00 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        Depth X    Eff. Unit Weight
 No.           in         lbs/in**3
-----       ----------  ---------------
  1           324.00        .06900
  2           360.00        .06900
  3           360.00        .06900
  4           432.00        .06900
  5           432.00        .08100
  6           500.40        .08100
  7           500.40        .08100
  8          1000.00        .08100


---------------------------------------------------------------------------
                          Shear Strength of Soils
---------------------------------------------------------------------------

Distribution of shear strength parameters with depth
defined using  8 points

Point     Depth X      Cohesion c     Angle of Friction     E50 or     RQD
 No.        in         lbs/in**2           Deg.              k_rm       %
-----    --------     -----------    -----------------     ------    ------
  1      324.000       12.15000             .00             ------    ------
  2      360.000       12.15000             .00             ------    ------
  3      360.000         .00000           30.00             ------    ------
  4      432.000         .00000           30.00             ------    ------
  5      432.000     1000.00000             .00             ------    ------
  6      500.400     1000.00000             .00             ------    ------
  7      500.400     2725.00000             .00             ------    ------
  8     1000.000     2725.00000             .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
```

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EWT 6-1-07

```
-----------------------------------------------------------------------------
            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     =       27020.000 lbs
Bending moment at pile head =             .000 in-lbs
Axial load at pile head     =     1305400.000 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                         =       54.0000 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               =             8
Area of Single Bar                       =       4.00000 In**2
Number of Rows of Reinforcing Bars       =             5
Cover Thickness (edge to bar center)     =         3.000 In

Unfactored Axial Squash Load Capacity    =      10557.70 Kip

Distribution and Area of Steel Reinforcement

 Row          Area of        Distance to
Number     Reinforcement   Centroidal Axis
               In**2             In
------     -------------   ---------------
   1         4.000000           24.0000
   2         8.000000           16.9706
   3         8.000000             .0000
   4         8.000000          -16.9706
   5         4.000000          -24.0000


Axial Thrust Force =   1305400.00  lbs

    Bending       Bending       Bending      Maximum      Neutral Axis
    Moment       Stiffness     Curvature     Strain        Position
    in-lbs        lb-in2        rad/in       in/in          inches
  ------------- ------------- ------------- ------------- -------------
      1768625.  1.768625E+12     .00000100     .00016562     165.62183
```

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```
    8837326.  1.767465E+12     .00000500      .00027505   55.00926590
   15852860.  1.761429E+12     .00000900      .00038572   42.85811234
   18240467.  1.403113E+12     .00001300      .00047054   36.19545364
   21176442.  1.245673E+12     .00001700      .00055530   32.66472244
   23731709.  1.130081E+12     .00002100      .00063584   30.27828598
   26064042.  1.042562E+12     .00002500      .00071351   28.54052353
   28247168.  9.740403E+11     .00002900      .00078898   27.20609665
   30343427.  9.194978E+11     .00003300      .00086325   26.15923691
   32372318.  8.749275E+11     .00003700      .00093663   25.31445694
   34350153.  8.378086E+11     .00004100      .00100946   24.62087631
   36290781.  8.064618E+11     .00004500      .00108215   24.04780197
   38175651.  7.790949E+11     .00004900      .00115371   23.54517746
   40049657.  7.556539E+11     .00005300      .00122621   23.13607407
   41875550.  7.346588E+11     .00005700      .00129762   22.76528549
   43701248.  7.164139E+11     .00006100      .00137089   22.47359848
   45474060.  6.996009E+11     .00006500      .00144233   22.18973923
   47227203.  6.844522E+11     .00006900      .00151428   21.94604874
   48692554.  6.670213E+11     .00007300      .00158411   21.70009232
   49905845.  6.481279E+11     .00007700      .00165343   21.47308731
   51071027.  6.305065E+11     .00008100      .00172062   21.24216843
   52220983.  6.143645E+11     .00008500      .00178827   21.03844070
   53354718.  5.994912E+11     .00008900      .00185638   20.85819626
   53858898.  5.791279E+11     .00009300      .00191851   20.62913132
   54213000.  5.588969E+11     .00009700      .00197551   20.36607742
   54560142.  5.401994E+11     .00010100      .00203288   20.12753677
   56868971.  4.341143E+11     .00013100      .00247582   18.89940262
   58209620.  3.615504E+11     .00016100      .00294029   18.26267624
   59345159.  3.107076E+11     .00019100      .00343395   17.97881699
   59541791.  2.694199E+11     .00022100      .00392966   17.78126907

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 =    58346.96504
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
Modulus of Elasticity of Reinforcement    =       29000. Kip/In**2
Number of Reinforcing Bars                =           12
Area of Single Bar                        =      4.00000 In**2
Number of Rows of Reinforcing Bars        =            7
Cover Thickness (edge to bar center)      =        3.000 In

Unfactored Axial Squash Load Capacity     =     13511.33 Kip

Distribution and Area of Steel Reinforcement

 Row          Area of         Distance to
Number     Reinforcement    Centroidal Axis
               In**2              In
------     -------------    ---------------
   1         4.000000           27.0000
   2         8.000000           23.3827
   3         8.000000           13.5000
```

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SWT 6-1-07

```
     4          8.000000           .0000
     5          8.000000        -13.5000
     6          8.000000        -23.3827
     7          4.000000        -27.0000


 Axial Thrust Force =    1305400.00  lbs

      Bending          Bending        Bending       Maximum      Neutral Axis
      Moment          Stiffness      Curvature      Strain        Position
      in-lbs           lb-in2         rad/in        in/in          inches
   ------------- ------------- ------------- ------------- -------------
        2832115.  2.832115E+12     .00000100     .00013926     139.26212
       14140749.  2.828150E+12     .00000500     .00026089   52.17739105
       20634675.  2.292742E+12     .00000900     .00036470   40.52249908
       25559841.  1.966142E+12     .00001300     .00045823   35.24837494
       29806268.  1.753310E+12     .00001700     .00054603   32.11956024
       33732515.  1.606310E+12     .00002100     .00063060   30.02872467
       37484106.  1.499364E+12     .00002500     .00071337   28.53481293
       41109232.  1.417560E+12     .00002900     .00079475   27.40528107
       44656511.  1.353228E+12     .00003300     .00087557   26.53255463
       48144899.  1.301213E+12     .00003700     .00095628   25.84545135
       51554974.  1.257438E+12     .00004100     .00103589   25.26569366
       54945061.  1.221001E+12     .00004500     .00111670   24.81548309
       58267100.  1.189124E+12     .00004900     .00119646   24.41745758
       61554316.  1.161402E+12     .00005300     .00127675   24.08969879
       64820160.  1.137196E+12     .00005700     .00135862   23.83541107
       68025168.  1.115167E+12     .00006100     .00143938   23.59645844
       70688543.  1.087516E+12     .00006500     .00151730   23.34308624
       73019717.  1.058257E+12     .00006900     .00159357   23.09520721
       74376300.  1.018853E+12     .00007300     .00166415   22.79651642
       75664812.  9.826599E+11     .00007700     .00173246   22.49942780
       76935045.  9.498154E+11     .00008100     .00180119   22.23690033
       78187049.  9.198476E+11     .00008500     .00187037   22.00435638
       79445660.  8.926479E+11     .00008900     .00194197   21.81987762
       80644560.  8.671458E+11     .00009300     .00201094   21.62303925
       81359259.  8.387552E+11     .00009700     .00207435   21.38500214
       81837375.  8.102710E+11     .00010100     .00213520   21.14055634
       84864077.  6.478174E+11     .00013100     .00260903   19.91626740
       86978629.  5.402399E+11     .00016100     .00309398   19.21726227
       88067783.  4.610879E+11     .00019100     .00363190   19.01515961
       88067783.  3.984968E+11     .00022100     .00419901   19.00005341

 Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 =    86568.84551
 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    =       27020.000 lbs
 Specified moment at pile head         =            .000 in-lbs
 Specified axial load at pile head     =     1305400.000 lbs

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

   Depth      Deflect.    Moment        Shear       Slope      Total     Flx. Rig.   Soil Res
     X           y          M             V           S        Stress       EI          p
```

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6-1-07

```
     in         in       lbs-in       lbs        Rad.   lbs/in**2 lbs-in**2  lbs/in
 --------  ---------  ---------  ---------  ---------  --------- ---------  ---------
    0.000   .561388   1.35E-05     27020.    -.001791    570.044  1.77E+12      0.000
    6.600   .549570   1.94E+05     27020.    -.001790    582.577  1.77E+12      0.000
   13.200   .537758   3.88E+05     27020.    -.001789    595.111  1.77E+12      0.000
   19.800   .525954   5.81E+05     27020.    -.001787    607.643  1.77E+12      0.000
   26.400   .514165   7.75E+05     27020.    -.001785    620.174  1.77E+12      0.000
   33.000   .502395   9.69E+05     27020.    -.001782    632.704  1.77E+12      0.000
   39.600   .490649   1.16E+06     27020.    -.001778    645.232  1.77E+12      0.000
   46.200   .478932   1.36E+06     27020.    -.001773    657.757  1.77E+12      0.000
   52.800   .467248   1.55E+06     27020.    -.001767    670.280  1.77E+12      0.000
   59.400   .455602   1.74E+06     27020.    -.001761    682.799  1.77E+12      0.000
   66.000   .443999   1.94E+06     27020.    -.001754    695.314  1.77E+12      0.000
   72.600   .432444   2.13E+06     27020.    -.001747    707.826  1.77E+12      0.000
   79.200   .420941   2.32E+06     27020.    -.001739    720.333  1.77E+12      0.000
   85.800   .409496   2.52E+06     27020.    -.001729    732.835  1.77E+12      0.000
   92.400   .398112   2.71E+06     27020.    -.001720    745.332  1.77E+12      0.000
   99.000   .386795   2.90E+06     27020.    -.001709    757.824  1.77E+12      0.000
  105.600   .375550   3.10E+06     27020.    -.001698    770.309  1.77E+12      0.000
  112.200   .364381   3.29E+06     27020.    -.001686    782.788  1.77E+12      0.000
  118.800   .353293   3.48E+06     27020.    -.001673    795.260  1.77E+12      0.000
  125.400   .342291   3.67E+06     27020.    -.001660    807.725  1.77E+12      0.000
  132.000   .331380   3.87E+06     27020.    -.001646    820.182  1.77E+12      0.000
  138.600   .320563   4.06E+06     27020.    -.001631    832.631  1.77E+12      0.000
  145.200   .309847   4.25E+06     27020.    -.001616    845.072  1.77E+12      0.000
  151.800   .299235   4.44E+06     27020.    -.001600    857.504  1.77E+12      0.000
  158.400   .288733   4.64E+06     27020.    -.001583    869.927  1.77E+12      0.000
  165.000   .278346   4.83E+06     27020.    -.001565    882.340  1.77E+12      0.000
  171.600   .268077   5.02E+06     27020.    -.001547    894.743  1.77E+12      0.000
  178.200   .257932   5.21E+06     27020.    -.001527    907.135  1.77E+12      0.000
  184.800   .247915   5.40E+06     27020.    -.001508    919.517  1.77E+12      0.000
  191.400   .238031   5.59E+06     27020.    -.001487    931.887  1.77E+12      0.000
  198.000   .228286   5.78E+06     27020.    -.001466    944.246  1.77E+12      0.000
  204.600   .218682   5.98E+06     27020.    -.001444    956.593  1.77E+12      0.000
  211.200   .209226   6.17E+06     27020.    -.001421    968.927  1.77E+12      0.000
  217.800   .199923   6.36E+06     27020.    -.001398    981.248  1.77E+12      0.000
  224.400   .190775   6.55E+06     27020.    -.001374    993.557  1.77E+12      0.000
  231.000   .181789   6.74E+06     27020.    -.001349   1005.851  1.77E+12      0.000
  237.600   .172969   6.93E+06     27020.    -.001323   1018.132  1.77E+12      0.000
  244.200   .164320   7.12E+06     27020.    -.001297   1030.398  1.77E+12      0.000
  250.800   .155846   7.31E+06     27020.    -.001270   1042.649  1.77E+12      0.000
  257.400   .147552   7.50E+06     27020.    -.001243   1054.885  1.77E+12      0.000
  264.000   .139443   7.68E+06     27020.    -.001214   1067.106  1.77E+12      0.000
  270.600   .131524   7.87E+06     27020.    -.001185   1079.311  1.77E+12      0.000
  277.200   .123798   8.06E+06     27020.    -.001156    841.902  1.77E+12      0.000
  283.800   .116271   8.25E+06     27020.    -.001125    850.775  1.77E+12      0.000
  290.400   .108947   8.44E+06     27020.    -.001094    859.636  1.77E+12      0.000
  297.000   .101832   8.62E+06     27020.    -.001062    868.483  1.77E+12      0.000
  303.600   .094928   8.81E+06     27020.    -.001029    877.318  1.77E+12      0.000
  310.200   .088242   9.00E+06     27020.    -.000996    886.139  1.77E+12      0.000
  316.800   .081778   9.19E+06     27020.    -.000962    894.947  1.77E+12      0.000
  323.400   .075541   9.37E+06     27020.    -.000928    903.740  1.77E+12      0.000
  330.000   .069534   9.56E+06     25138.    -.000892    912.519  1.77E+12   -570.269
  336.600   .063763   9.72E+06     21359.    -.000856    920.113  1.77E+12   -574.783
  343.200   .058232   9.86E+06     17554.    -.000820    926.511  1.77E+12   -578.250
  349.800   .052944   9.97E+06     13730.    -.000783    931.706  1.77E+12   -580.624
  356.400   .047901   1.01E+07   9893.967    -.000745    935.693  1.77E+12   -581.856
  363.000   .043107   1.01E+07   7746.933    -.000708    938.470  1.77E+12    -68.761
  369.600   .038562   1.02E+07   7296.039    -.000670    941.091  1.77E+12    -67.874
  376.200   .034268   1.02E+07   6854.357    -.000632    943.556  1.77E+12    -65.969
  382.800   .030226   1.03E+07   6428.180    -.000593    945.870  1.77E+12    -63.175
  389.400   .026437   1.03E+07   6022.963    -.000555    948.039  1.77E+12    -59.618
  396.000   .022902   1.04E+07   5643.318    -.000516    950.070  1.77E+12    -55.426
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  402.600    .019623  1.04E+07  5293.008  -.000477    951.972  1.77E+12    -50.728
  409.200    .016601  1.04E+07  4974.945  -.000438    953.753  1.77E+12    -45.654
  415.800    .013836  1.05E+07  4691.188  -.000399    955.425  1.77E+12    -40.333
  422.400    .011329  1.05E+07  4442.938  -.000360    956.998  1.77E+12    -34.895
  429.000    .009081  1.05E+07  4230.533  -.000321    958.483  1.77E+12    -29.470
  435.600    .007094  1.06E+07   -19276.  -.000281    959.892  1.77E+12  -7093.745
  442.200    .005367  1.03E+07   -60396.  -.000242    946.713  1.77E+12  -5366.738
  448.800    .003893  9.77E+06   -90954.  -.000205    922.494  1.77E+12  -3893.455
  455.400    .002661  9.09E+06 -1.13E+05  -.000170    890.263  1.77E+12  -2661.140
  462.000    .001653  8.29E+06 -1.27E+05  -.000137    852.551  1.77E+12  -1652.819
  468.600    .000849  7.42E+06 -1.35E+05  -.000108    811.432  1.77E+12   -848.733
  475.200    .000227  6.51E+06 -1.39E+05 -8.20E-05    768.558  1.77E+12   -227.388
  481.800   -.000234  5.59E+06 -1.39E+05 -5.94E-05    725.207  1.77E+12    233.626
  488.400   -.000557  4.68E+06 -1.36E+05 -4.03E-05    682.328  1.77E+12    556.970
  495.000   -.000765  3.79E+06 -1.32E+05 -2.44E-05    640.586  1.77E+12    765.058
  501.600   -.000880  2.94E+06 -1.21E+05 -1.19E-05    600.409  1.77E+12   2397.202
  508.200   -.000922  2.19E+06 -1.05E+05 -2.30E-06    565.152  1.77E+12   2512.231
  514.800   -.000910  1.55E+06   -88523.  4.69E-06    535.053  1.77E+12   2480.089
  521.400   -.000860  1.02E+06   -72605.  9.50E-06    510.045  1.77E+12   2343.637
  528.000   -.000785  5.96E+05   -57814.  1.25E-05    489.850  1.77E+12   2138.468
  534.600   -.000695  2.61E+05   -44509.  1.41E-05    474.047  1.77E+12   1893.322
  541.200   -.000598  7875.394   -32880.  1.46E-05    462.133  1.77E+12   1630.691
  547.800   -.000502 -1.74E+05   -22986.  1.43E-05    469.955  1.77E+12   1367.532
  554.400   -.000410 -2.96E+05   -14790.  1.34E-05    475.710  1.77E+12   1116.033
  561.000   -.000325 -3.69E+05 -8188.699  1.22E-05    479.172  1.77E+12    884.386
  567.600   -.000249 -4.04E+05 -3034.419  1.07E-05    480.817  1.77E+12    677.517
  574.200   -.000183 -4.09E+05   844.027  9.23E-06    481.070  1.77E+12    497.769
  580.800   -.000127 -3.93E+05  3626.818  7.73E-06    480.299  1.77E+12    345.501
  587.400  -8.06E-05 -3.62E+05  5491.701  6.32E-06    478.818  1.77E+12    219.615
  594.000  -4.33E-05 -3.21E+05  6605.850  5.05E-06    476.886  1.77E+12    118.005
  600.600  -1.39E-05 -2.75E+05  7120.406  3.94E-06    474.710  1.77E+12     37.921
  607.200   8.71E-06 -2.27E+05  7167.219  3.00E-06    472.457  1.77E+12    -23.735
  613.800   2.58E-05 -1.80E+05  6857.337  2.25E-06    470.252  1.77E+12    -70.169
  620.400   3.84E-05 -1.36E+05  6280.866  1.66E-06    468.190  1.77E+12   -104.520
  627.000   4.76E-05   -97157.  5507.871  1.22E-06    466.343  1.77E+12   -129.721
  633.600   5.45E-05   -63639.  4590.064  9.20E-07    464.763  1.77E+12   -148.402
  640.200   5.97E-05   -36584.  3563.057  7.33E-07    463.487  1.77E+12   -162.812
  646.800   6.41E-05   -16619.  2449.048  6.34E-07    462.545  1.77E+12   -174.766
  653.400   6.81E-05 -4267.272  1259.821  5.95E-07    461.963  1.77E+12   -185.606
  660.000   7.20E-05     0.000     0.000  5.87E-07    461.762  1.77E+12   -196.158

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              =       .56138794 in
Computed slope at pile head       =     -.00179055
Maximum bending moment            =       10563233. lbs-in
Maximum shear force               =  -138626.85694 lbs
Depth of maximum bending moment   =       435.60000 in
Depth of maximum shear force      =       475.20000 in
Number of iterations              =               7
Number of zero deflection points  =               2
```

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--------------------------------------------------------------------------------
                        Summary of Pile-Head Response(s)
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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   Boundary      Boundary         Axial     Pile Head   Pile-Head    Pile Head
Type   Condition     Condition        Load      Deflection   Moment       Shear
          1             2             lbs          in        in-lbs        lbs
---- ------------ ------------ ----------- ----------- ----------- -----------
  1  V=    27020. M=      0.000    1305400.    .5613879  1.0563E+07    -138627.

The analysis ended normally.
```

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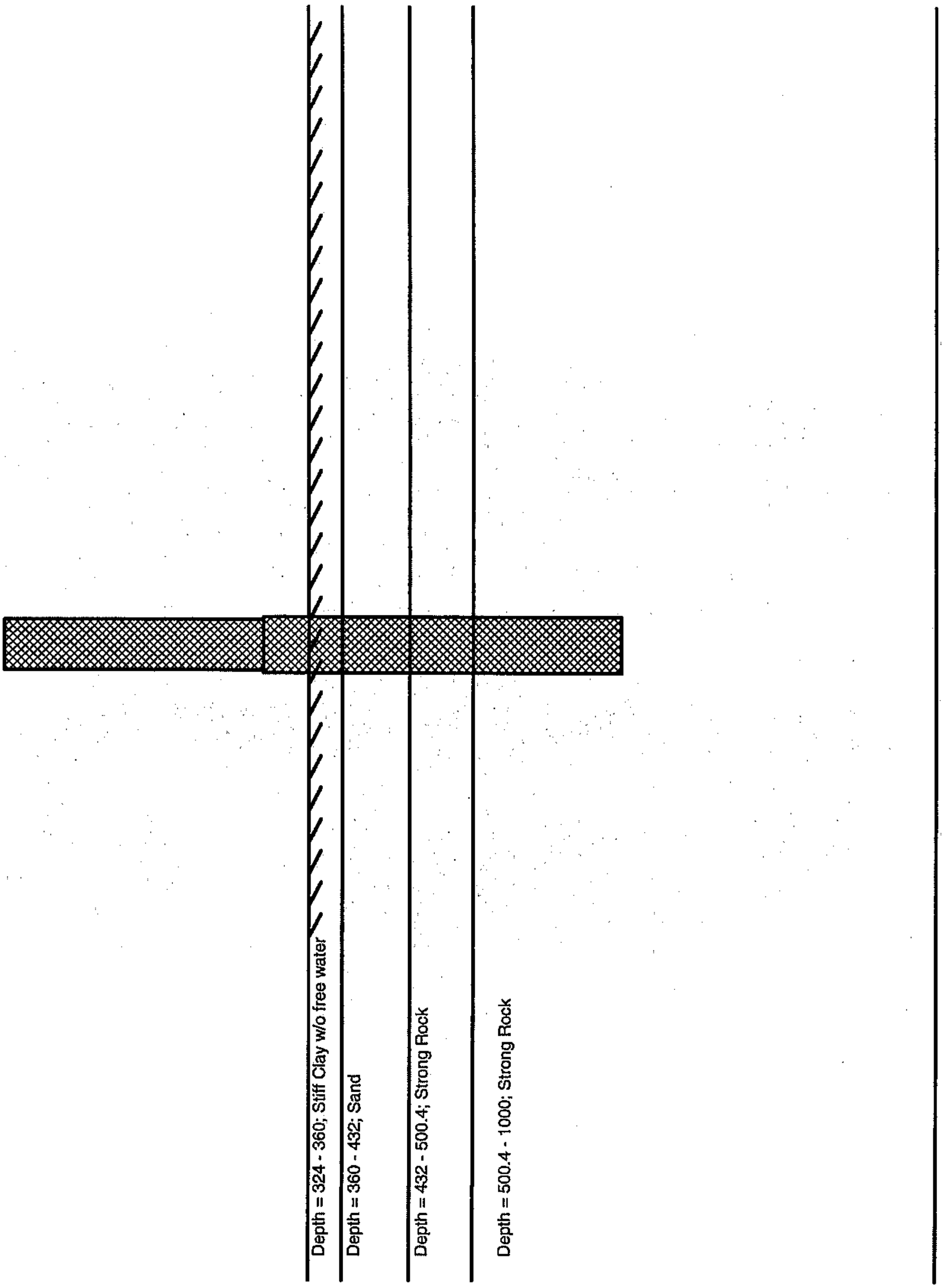

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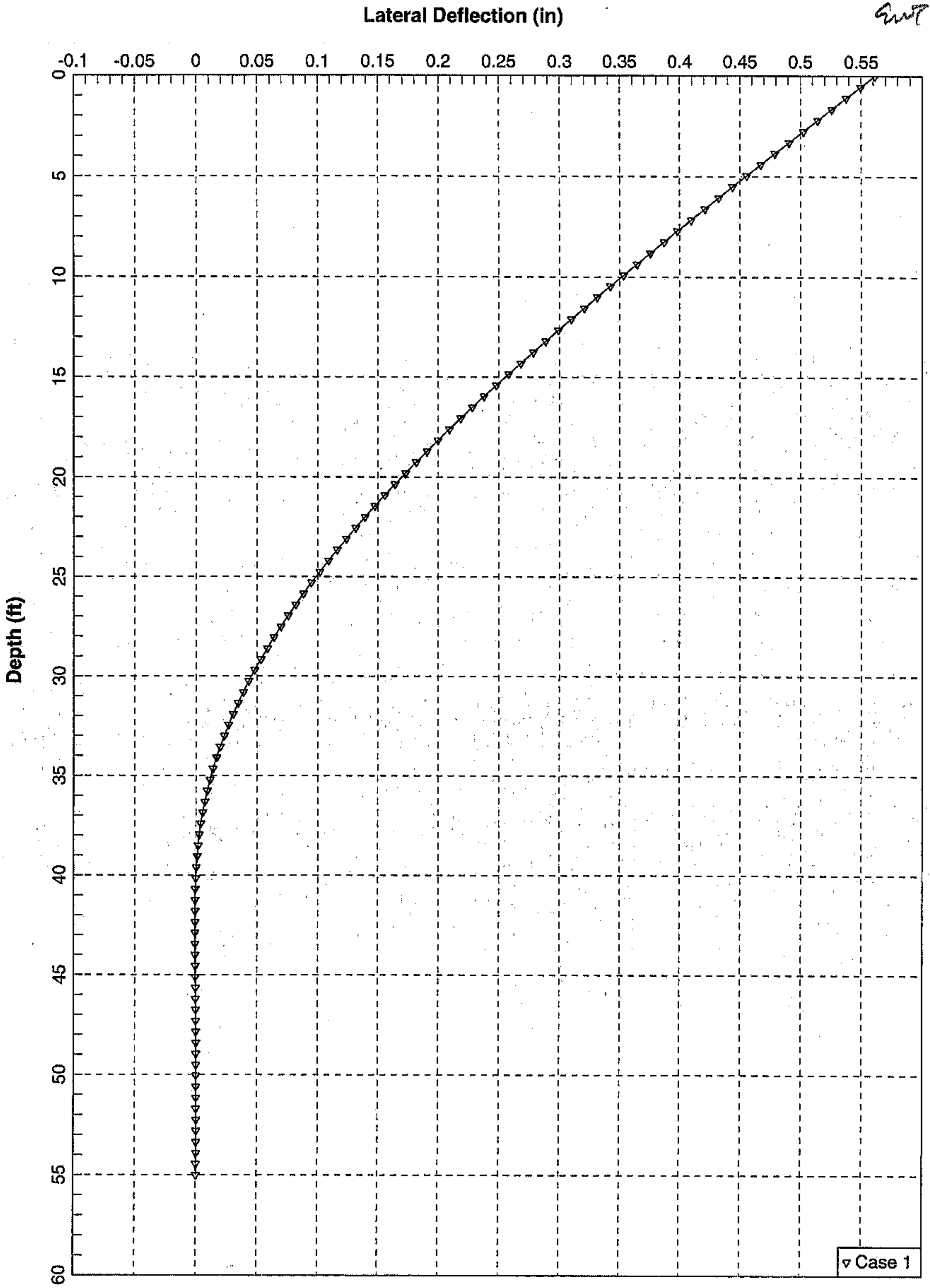

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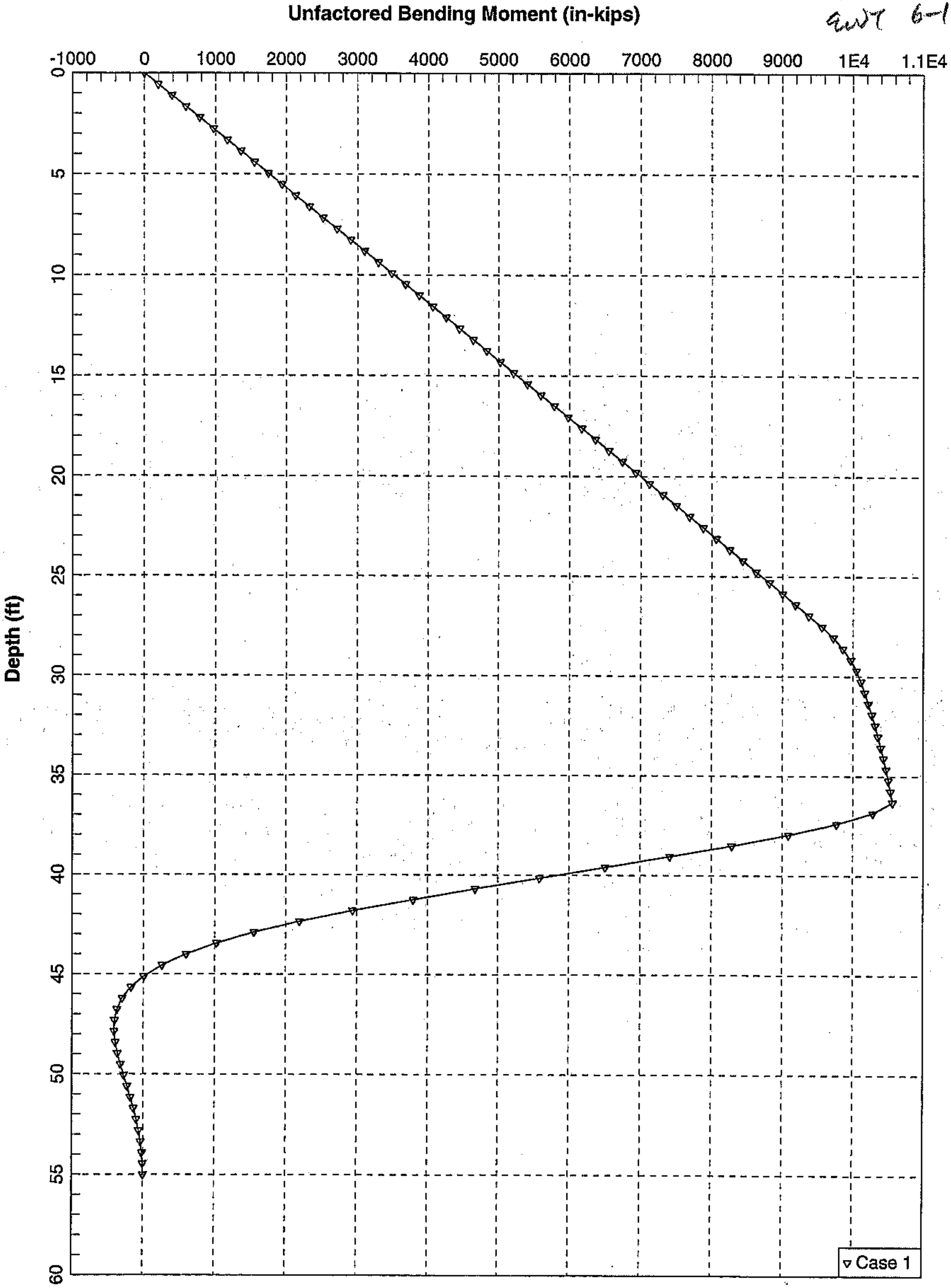

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