Final Report of:

Subsurface Exploration
Bridge and Retaining Walls
Relocated Shumway Hollow Road Over CSXT Railroad
Project SCI-823-6.81 Portsmouth Bypass (PID 19415)
Scioto County, Ohio

Prepared for:

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DLZ Job No. 0121-3070.03

October 1, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND RETAINING WALLS

RELOCATED SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

PROJECT SCI-823-6.81 PORTSMOUTH BYPASS (PID 19415)

SCIOTO COUNTY, OHIO

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1.0 INTRODUCTION

This report includes the findings of evaluations for foundations and retaining walls for the structure at the relocated Shumway Hollow Road over the CSXT Railroad. The retaining walls evaluated include mechanically stabilized earth (MSE) and drilled shaft retaining walls. The findings included in this report pertain to the structure at relocated Shumway Hollow Road over the CSXT railroad only and supercede recommendations presented in previous reports dated November 20, 2006 and April 19, 2007. The findings of other structure evaluations will be submitted in separate documents. This document presents updated recommendations for foundations and retaining walls at the forward abutment location.

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

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

2.0 GENERAL PROJECT INFORMATION

The currently proposed structure is shown on the provided plan and profile drawing in Appendix I. This portion of the project consists of constructing a single-span bridge on relocated Shumway Hollow Road over the CSXT Railroad. It is anticipated that the proposed rear abutment will be founded on a fill section, contained using an MSE wall. It is also anticipated that the forward abutment will be founded on the slope of a soil/rock cut near station 37+90 (Shumway Hollow stationing) using a drilled shaft foundation / retaining wall system.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the fill at station 36+70 (rear abutment) will be approximately 43 feet. It is understood that the forward abutment will be founded on the slope of a soil/rock cut near station 37+90. The proposed roadway grade at the structure varies from approximate elevation 660 to 662.

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

3.0 FIELD EXPLORATION

The field exploration consisted of drilling a total of six structural borings. Borings B-24 through B-27 were drilled for the currently proposed bridge plan, essentially consisting of proposed Shumway Hollow Road over CSXT Railroad, as shown on the structural site plan in Appendix I. Structure borings, B-24 through B-27 were drilled between January 17 and 30, 2007. Structure borings TR-27 and TR-28 were drilled on August 25, 2004 and February 2, 2005, respectively for a previous design configuration. The boring logs are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

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

4.0 FINDINGS

4.1 Geology of the Site

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

The genesis of the soils varies across the site. Residual and colluvial soils are found on the ridge tops and the hillsides near the site. These soils are generally thin to moderately deep, covering moderate to steep slopes. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. The area of soils of the Minford Complex generally overlie a layer of sand and gravel which is directly above bedrock. In this area, the Minford Complex is characterized by clays of high plasticity and high moisture content. Although borings drilled for this structure did not encounter soils of the Minford Complex, several other borings drilled for the Shumway Hollow / SR 823 interchange did encounter these soils.

Bedrock within the structure area is primarily sandstone of the Logan Formation of Mississippian Age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the west of the structure, roughly above elevation 860.

4.2 Field Reconnaissance

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

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

Reconnaissance of the site at the bottom of the cut confirmed the presence of a very soft and wet environment at the proposed rear abutment location. Drainage channels have been established along the railroad cut, which currently run near the rear abutment location. These drainage paths have deposited approximately 3 to 5 feel of soil, as confirmed by the borings drilled for the rear abutment.

4.3 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix II. Laboratory test results are presented on the boring logs and also in Appendix III.

4.3.1 Soil Conditions

Borings B-24 and B-25 were drilled for the rear abutment of the currently proposed structure. Similarly, borings B-26 and B-27 were drilled for the forward abutment of the currently proposed structure. Boring TR-27 was also considered in the evaluation of the rear abutment location. Similarly, boring TR-28 was considered in the evaluation of the forward abutment location.

Borings TR-27 and B-24 through B-27 encountered surficial material consisting of 5 to 8 inches of topsoil while boring TR-28 encountered 8 inches of asphalt concrete pavement. All borings encountered native cohesive and granular soil deposits below the surficial material. The cohesive deposits consisted mainly of medium stiff to very stiff silt and clay (A-6a), medium stiff clay (A-6b), and medium stiff to hard sandy silt (A-4a), while the granular soil deposits consisted mainly of loose to dense coarse and fine sand (A-3a) and medium dense sand (A-3). Boring B-26 encountered a relatively thin soft silt and clay (A-6a) layer (approximately 2-foot thick) above the sandstone. The native soil deposits were 3.0 feet thick at the rear abutment and between 16.5 and 17.5 feet thick at the forward abutment. It should be noted that the presence of organic material was noted in boring B-24, drilled at the rear abutment location.

4.3.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all the borings and was confirmed by coring between 10 and 20 feet of rock in each boring. The bedrock consisted of medium hard to hard, slightly to highly weathered, slightly fractured sandstone. A layer of severely weathered rock, ranging in thickness between 1.5 to 3 feet was encountered above the more competent cored bedrock in borings B-24, B-25, and TR-28. The amount of rock recovered in each core run varied between 50 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 12 and 100 percent with an average of 80 percent indicating "good" rock.

Unconfined compressive strength of tested cores ranged between 9,952 psi and 13,148 psi. The tested cores correspond to samples at depths between 10.0 feet and 32.5 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1. Also, 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-24	10.0-10.5	157.7	9,952
B-25	10.5-11.0	155.8	10,295
B-26	22.5-23.0	140.5	10,454
	32.0-32.5	146.9	12,453
B-27	21.5-22.0	136.6	13,148
	32.0-32.5	143.5	12,949

4.3.3 Groundwater Conditions

Minor seepage was encountered at depths between 8.5 and 18.5 feet below the ground surface in borings B-26, B-27, and TR-28 only. Measurable water levels prior to rock coring were encountered at depths between 14.3 and 36.1 in borings B-26 and B-27 only. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels were present in all borings except TR-27 upon the completion of coring between approximate depths of 5.5 and 35.8 feet. Boring TR-27 collapsed at a depth of 6.0 feet and did not have a measurable water level. It should be noted that the final water levels include drilling water and consequently may not be representative of the actual groundwater conditions.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General Information

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

It is understood that maintenance of traffic issues at the intersection of SR 335 and relocated Shumway Hollow Road have limited the type of the foundation and retaining wall systems that may be considered at the forward abutment location. It has been expressed that at least one lane of SR 335 must remain open during the entire construction process. In accordance with this, several retaining wall options have been eliminated due to the impact of excavations or required construction limits. If this requirement is modified, alternative foundation and retaining wall system recommendations can be provided upon request.

The current configuration includes the retention of approximately 20 feet of fill at the forward abutment. Consequently, the currently proposed drilled shaft foundations will have to be designed to resist the lateral loading of the fill material. Recommendations for this drilled shaft foundation / retaining wall system at the forward abutment location are included in the following sections. The following sections also contain additional recommendations and information for the design of the proposed structural foundations and MSE wall. Table 2 summarizes the site conditions and foundation recommendations. Calculations are presented in Appendix IV.

Table 2 - Summary of Foundation Recommendation

Structural Element	Structure / Boring	Existing Ground Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-24	625.9	CIP Piles	615.0 *	Maximum Allowable Capacity
			Drilled Shafts	615.0 *	80 ksf++
			Spread Footings	MSE Fill	4 ksf
	Right / B-25	625.0	CIP Piles	615.0 *	Maximum Allowable Capacity
			Drilled Shafts	615.0 *	80 ksf++
			Spread Footings	MSE Fill	4 ksf
Forward Abutment	Left / B-26	660.2	Drilled Shafts	615.0+	80 ksf++
	Right TR-28 / B-27	659.7 656.8	Drilled Shafts	615.0+	80 ksf++

* Includes 5-foot socket into competent rock.

+ Minimum tip elevation 615 for drilled shafts subject to lateral loading (forward abutment wall).

++ End bearing capacity only.

5.2 Bridge Foundation Recommendations

5.2.1 Rear Abutment (Sta. 36+70)

Spread footings bearing in the MSE wall fill could be considered to support the rear abutment. As per the Bridge Design Manual 204.6.2.1, an allowable bearing capacity of 4 ksf may be used to design the footings. The MSE wall, as proposed, will be founded on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be negligible.

However, it should be noted that the proposed rear abutment lies in close proximity to a drainage ditch, which runs essentially parallel to the railroad tracks. The area where the MSE wall is currently proposed is prone to frequent flooding. Consequently, over time there is a risk of migration of the select granular fill from the reinforced zone.

As an alternative to spread footings, CIP piles could be used to support the rear abutment. 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 steel CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment granular fill. Pile sleeves should also be used to mitigate down drag effects from compaction and to protect the pile during the embankment and MSE wall construction. The maximum allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration.

At this time, excessive lateral loading and uplift is not anticipated to be a concern at the rear abutment. However, if these forces are significant, longer socket lengths may be required. Due to the relatively small rigidity of the steel CIP piles compared to drilled shafts, the steel CIP piles are anticipated to provide low resistance to lateral earth pressures that can be induced in high embankment fills. Therefore, the prebored and socketed steel pipe pile foundation system may not be suitable if significant lateral loads are present.

As mentioned above, drilled shafts may also be considered for the support of the rear abutment. Drilled shafts, socketed a minimum of 5 feet into competent rock are recommended to support the proposed rear abutment. This corresponds to an approximate bearing elevation of 615 at the rear abutment. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance, ignoring any end bearing capacity.

5.2.2 Forward Abutment (Sta. 37+90)

The forward abutment of the proposed structure lies on the eastern slope of a railroad cut, which is approximately 33 feet deep. Based upon provided drawings and the available subsurface information, it is anticipated that approximately 20 feet of fill will be retained at the forward abutment location. Based upon these conditions, it is recommended that a drilled shaft foundation / retaining wall system be used to support the structure and retain the fill at the forward abutment location. The drilled shafts will need to be designed to resist the lateral loading from the fill material as well as any loads from the proposed structure.

From the borings, it is anticipated that competent bedrock will be encountered within 3 to 5 feet below the soil-rock interface at the proposed centerline of bearing for the forward abutment. This corresponds to an approximate elevation of 637. However, based upon field reconnaissance, it is anticipated that the degree of weathering and fractures present in the rock located at the centerline of bearing for the forward abutment will be more severe than that encountered in the borings drilled for the proposed structure, at the crest of the slope.

It is believed that significant fracturing of the rock is present at the location of the proposed abutment wall. Because of this fracturing, the resistance to lateral loading, provided by the upper bedrock could be decreased. Analyses determined that the weight of the rock wedge providing passive resistance to lateral earth pressures above elevation 630 is inadequate. Small lateral movements of the upper bedrock (passive side) could occur due to the laminated nature of the bedrock positioned on the steep rock slope. If such movements are realized, the lateral resistance will need to be provided by deeper drilled shaft sockets. Below elevation 630, although the rock is laminated, it is assumed that it will generally behave as a confined unit. This is due to the elevation relative to the bottom of the adjacent rock cut. Consequently, it is recommended that the lateral capacity provided by the drilled shaft rock socket above elevation 630 be ignored. Furthermore, it is recommended that the socket extend a minimum of 15 feet below elevation 630 to elevation 615 to resist the lateral loading. The drilled shafts should be straight (not belled) and may be designed based on an allowable end bearing pressure of 80 ksf (40 tsf). It should be noted that the required socket length cited here is based upon geotechnical considerations only. Additional socket length may be required for structural purposes such as sufficient reinforcement development length.

If sufficient axial capacity cannot be obtained with a reasonable shaft diameter, the shafts could be designed as friction-type shafts. As discussed earlier, at a minimum, the rock socket should extend to elevation 615. If designed as friction-type shafts, the shafts should be designed such that design loads are carried entirely by the socket resistance, ignoring any end bearing capacity. Any resistance provided from the rock socket above elevation 630 should also be ignored. An allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used to design the shafts.

It is understood that only very small deflections could be tolerated by the integral-type abutment proposed at the forward abutment location. However, the drilled shaft foundation / retaining wall system proposed for the forward abutment would deflect under the influence of lateral earth pressure from the backfill materials. Based upon discussions with TranSystems, it is understood that deflections of the drilled shaft foundation / retaining wall prior to making the attachment to the superstructure should be limited to 1/8 of an inch or less, which is typically cited as being within acceptable construction tolerances. Considering the top of rock and the proposed grade at the forward abutment location, it is assumed that 14 feet of fill will be placed behind the drilled shaft foundation / retaining wall system prior to making the connection to the superstructure and placing the remaining fill. Based upon this assumption, it is evident that a deflection-based design, to limit the amount of deflection, is appropriate for the proposed drilled shaft foundation / retaining wall system. Considering the amount of allowable deflection and the compaction effort required for the backfill materials, it is recommended that the at-rest condition be assumed for analyses that include lateral earth pressures.

Originally, a tangent drilled shaft foundation / retaining wall system was considered as a possible configuration. However, this type of shaft layout is more difficult to construct due to the strict tolerance required to avoid overlap of the shafts and conflicts between the temporary casing and the adjacent shafts. Consequently, a configuration utilizing a drilled shaft spacing wider than the shaft diameter was assumed for the analyses. To complete the retaining wall, the void space between shafts will have to be filled, such as with bentonite-cement grout or with some form of lagging.

Several LPILE analyses were performed to determine a preliminary configuration (diameter, reinforcement ratio, and spacing) for the proposed drilled shaft foundation / retaining wall. Based upon the prescribed deflection criterion and discussions with TranSystems, the use of 48-inch diameter, reinforced concrete drilled shafts on a 60-inch center–to-center spacing is recommended to support the structure and retain the approach embankment fill. As discussed above, a permanent lagging will be required for this drilled shaft system. It is understood that the design of lagging will be provided by others.

It is understood that the structural design of the drilled shafts will be determined by others. However, an estimate of the longitudinal reinforcing steel was required in order to model the rock-structure interaction while using a non-linear bending stiffness in the LPILE program. If final design uses a reinforcement ratio, diameter, or spacing that differs significantly from that which was assumed, DLZ should be informed so that the model and recommendations may be revised as necessary. A sample LPILE output file is presented in Appendix IV. Additionally, a summary of the unfactored shear forces and bending moments generated in the drilled shafts from the lateral earth pressures for various configurations is also presented in Appendix IV.

5.2.3 Drilled Shaft Foundations: General 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. 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.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although the borings did not encounter significant seepage, water could flow into the drilled shafts during installation particularly below the drainage channel level (rear abutment) and within wet zones that may be present in the rock or soil. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

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

5.3 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

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

5.3.1 MSE Walls: General Information

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

The parameters required to perform the stability analyses are presented in Table 3, below. As outlined in section 5.3.2, it is recommended that the existing soils at the rear abutment location be removed to the top of rock and replaced with compacted granular fill. Consequently, the properties of the compacted granular fill are assumed for the foundation soil used in the stability and bearing capacity calculations of the MSE wall. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. Additionally, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 3 - Soil Parameters Used in MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear Abutment)	Compacted Granular Fill	120	0	34	0	34

5.3.2 MSE Wall Evaluations and Recommendations

The MSE fill at the rear abutment is understood to have a maximum height of approximately 43 feet. Borings drilled at the rear abutment first encountered bedrock at 3.0 to 5.0 feet below the ground surface. Based on the field reconnaissance of the site, very soft and wet soils were observed at the proposed rear abutment location. The conditions at the site may vary greatly depending upon the amount of recent precipitation. Consequently, it is recommended that soils overlying bedrock be removed, and the leveling pad be placed on bedrock. Additionally, provisions for diverting water away from the proposed MSE wall should be made to prevent any scour of the MSE wall materials.

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

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

Table 4 - MSE Retaining Wall Parameters and Results of Analyses (Rear Abutment) *Borings B-24 & B-25*

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi = 30°$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45
Allowable Bearing Capacity – Drained Condition q_{all} = 12,129 psf (Assumes Compacted Granular Fill Foundation on Rock)
Global Stability Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock) Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock) Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)
Estimated Settlement of MSE volume Total settlement = negligible (Assumes Compacted Granular Fill on Rock) Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 43.0 feet Approximate Embedment Depth = 0.0 feet (Bedrock) Minimum Length of Reinforcement for External Stability = (0.7)(H+D) or 30.1 feet

5.4 General Earthwork Recommendations

Only boring B-24 encountered organic material in the borings drilled for the proposed structure. However, since organic or very soft soils may be encountered at locations other than where the borings were drilled, the contractor should be prepared to perform overexcavation of any poor soils at other locations and replace the overexcavated soil with compacted engineered fill as needed. Additionally, all topsoil, organic soil within 3 feet of subgrade level, and vegetation should also be removed prior to placing fill or pavement materials. For satisfactory performance of the proposed MSE wall, it is recommended that the existing soils be overexcavated to the top of rock and replaced with compacted granular fill. The area of overexcavation should extend beyond the edge of the MSE wall/select granular footprint by a distance equal to the depth of the aggregate base or 3 feet, whichever is greater.

Durable sandstone is evident at the rear abutment location in the rock cut. Significant rock excavation to accommodate the reinforcing straps of the MSE retaining wall is not anticipated at this time. However, if necessary, 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.

5.5 Groundwater Considerations

Minor seepage was encountered in borings TR-28, B-26, and B-27. In these borings, seepage was first encountered at depths ranging from 8.5 to 14.0 feet below the ground surface. Groundwater was noted prior to adding drilling water in boring B-27 at a depth of 14.4 feet. Representative, final water levels could not be obtained due to the use of water during rock coring operations. The use of drilling water in rock coring operations also masked any seepage zones that may be present in the rock. Excavations for shafts extending below the soil-rock interface may encounter significant seepage through fractured zones in the rock.

Based on the field reconnaissance of the site, very soft and wet soils were observed at the proposed rear abutment location. Consequently, the contractor should anticipate significant seepage in the excavations for the proposed MSE retaining wall. The contractor should also be prepared to deal with any unexpected seepage, precipitation, or water flow that may enter any excavations.

5.6 Scour Analysis

Particle-size analyses were performed on samples collected in the area of the rear abutment for possible scour analysis. The flow line elevation in the existing drainage channel is reported to be approximate elevation 623.1. Table 5 presents the sample locations and the D_{50} and D_{85} sizes from the particle size analyses. The samples tested are considered representative of the alluvial material, which has been deposited in the area of the proposed rear abutment.

Table 5 – Particle Size Data

Boring Number	Ground Surface Elevation ft. (at boring)	Sample Depth (below ground surface)	ODOT Classification	D_{85} (mm)	D_{50} (mm)
B-24	625.9	1.0-2.5	A-6a	0.0337	0.0100
B-25	625.0	1.0-2.5	A-6a	0.0255	0.0061

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
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sjr

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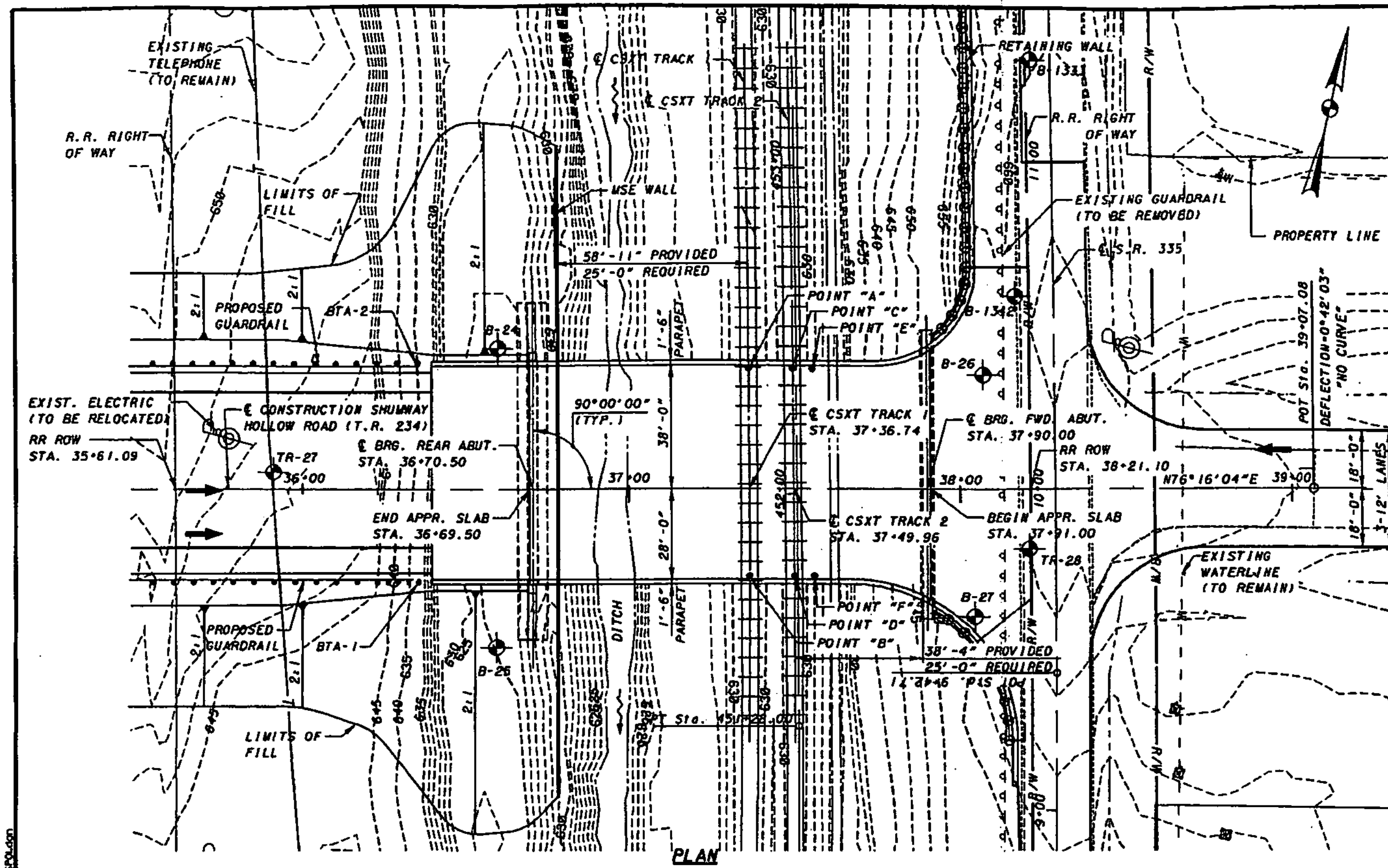
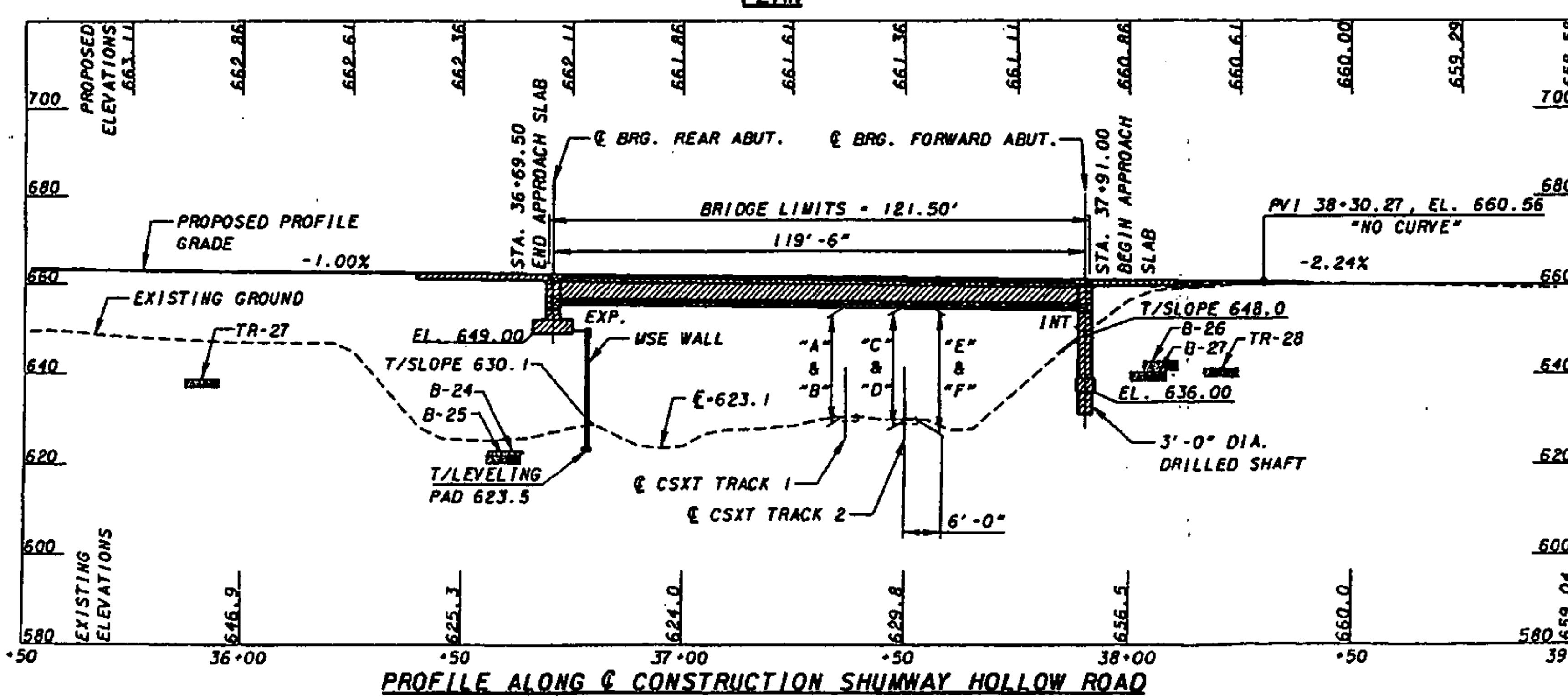
APPENDIX I

Structure Plan and Profile Drawing – 11"x17"



PLAN



PROFILE ALONG ℄ CONSTRUCTION SHUMWAY HOLLOW ROAD

GUARDRAIL POST		
LOCATION	STATION	OFFSET
REAR ABUT	36+35.37	38' LEFT
REAR ABUT	36+35.24	28' RIGHT

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

TRAFFIC DATA

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

TABLE OF VERTICAL CLEARANCES						
LOCATION	"A"	"B"	"C"	"D"	"E"*	"F"*
PROPOSED	24.16'	23.49'	24.30'	23.52'	24.29'	23.46'
REQUIRED	23.0'	23.0'	23.0'	23.0'	23.0'	23.0'

* 6'-0" FROM ℄ CSXT TRACK 2

NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

FOUNDATION DATA:
REAR ABUTMENT SHALL BE ON SPREAD FOOTING WITH AN ALLOWABLE BEARING CAPACITY OF 4 KSF. FORWARD ABUTMENT SHALL DRILLED SHAFTS WITH AN ALLOWABLE BEARING CAPACITY OF 40 KSF.

LEGEND

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

BORING INFORMATION	
BORING ID	T/ROCK ELEV.
B-24	622.9
B-25	622.0
B-26	642.7
B-27	640.3
TR-27	638.8
TR-28	644.2

FOR BORINGS B-1333 AND B-1312 SEE RETAINING WALL SITE PLAN.

PROPOSED STRUCTURE

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

SPANS: 119'-6" C/C BEARINGS

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

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

SKEW: NONE

CROWN: VARIES

ALIGNMENT: TANGENT

WEARING SURFACE: MONOLITHIC CONCRETE

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

LATITUDE: 38° 50' 30" N
LONGITUDE: 82° 51' 00" W

DESIGNED USL · CHECKED PJP · DRAWN CAS · REVIEWED MTN · REVISED JRC · DATE 10/12/06 · STRUCTURE FILE NUMBER 7336934

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

SITE PLAN
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID 19415

1/4

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Six (6) Borings
Log of Rock Cut – Eastern Slope

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

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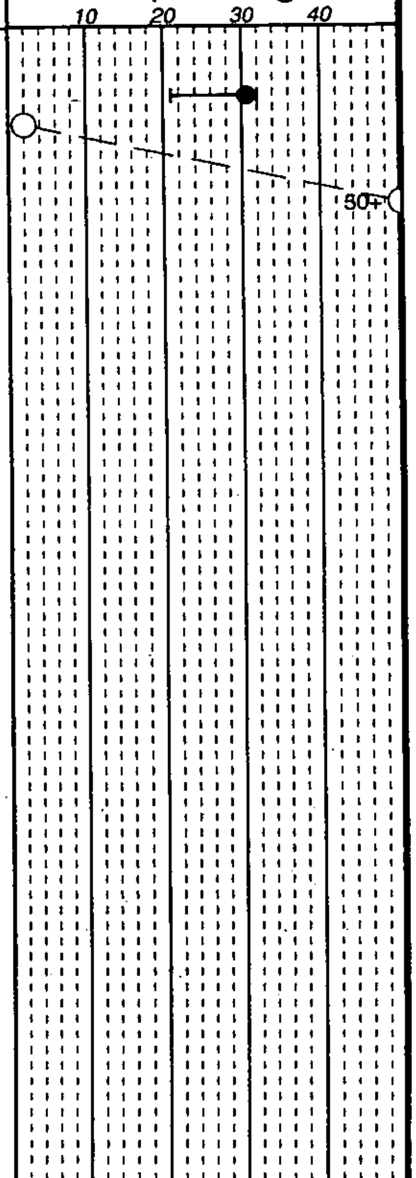
Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring B-24	Location: Sta. 36+59.8, 43.4 ft. LT of Rel. Shumway Hollow CL	Date Drilled: 1/17/07

WATER OBSERVATIONS:
Water seepage at: None
Water level at completion: None (prior to coring)
6.6' (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	625.9						Topsoil - 7"						
0.6	625.3	WOH 1 1	18	1		1.0	Medium stiff to stiff brown SILT AND CLAY (A-6a), trace fine sand; contains organic material; damp to moist.	0	1	--	2	65	33
3.0	622.9	9 50/3	5	2			Severely weathered gray SANDSTONE, argillaceous, micaceous.						
5.0	620.9	Core 120"	Rec 120"	RQD 90%	R1		Hard gray SANDSTONE; fine grained, slightly weathered, argillaceous, micaceous, thinly laminated, slightly fractured. @ 5.2', 5.4', 6.9', 7.9', 11.3', 12.3', 12.6', 13.8', low angle clay filled fractures. @ 10.0'-10.5', qu = 9,952 psi.						
15.0	610.9						Bottom of Boring - 15.0'						

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



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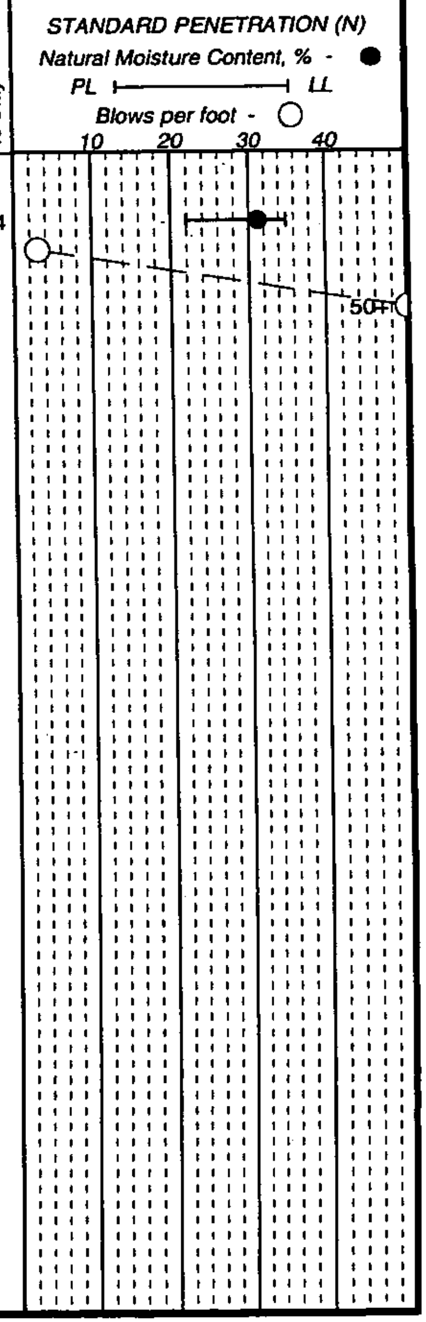
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-25 | Location: Sta. 36+59.3, 49.2 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 1/17/07

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

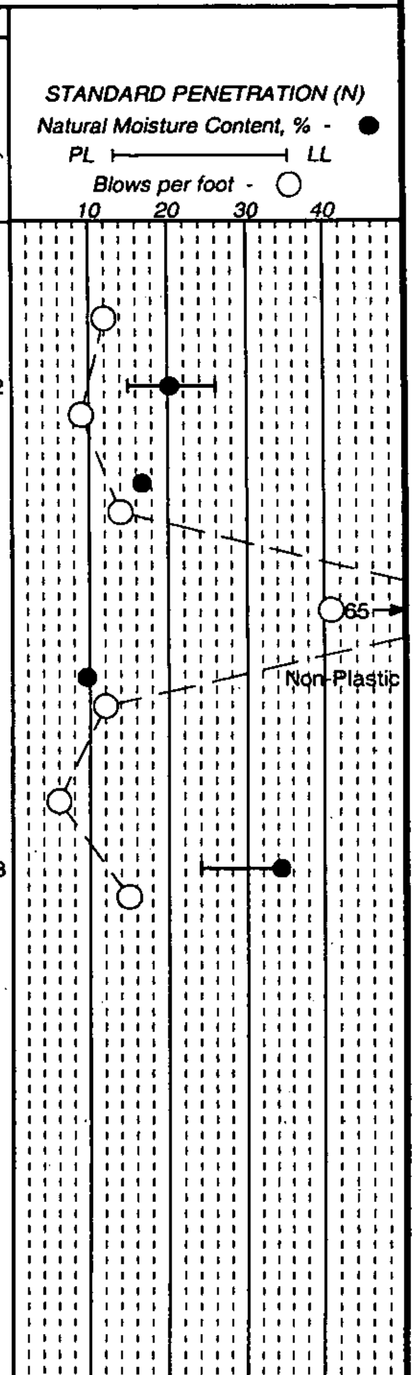
Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	625.0												
0.5	624.5						Topsoil - 6"						
		1 1 2	18	1		1.25	Stiff brown SILT AND CLAY (A-6a), trace fine sand, trace gravel; damp to moist.	2	0	--	2	53	44
3.0	622.0	50/2	1	2			Severely weathered gray SANDSTONE, argillaceous, micaceous.						
4.5	620.5	Core 120"	Rec 120"	RQD 85%	R1		Hard gray SANDSTONE interbedded with SILTSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thinly laminated, slightly fractured. @ 4.5'-5.0', 6.2'-7.0', broken zones. @ 10.5'-11.0', qu = 10,295 psi. @ 11.6', 11.7', 13.5', low angle clay filled fractures.						
14.5	610.5						Bottom of Boring - 14.5'						



Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring B-26	Location: Sta. 38+06.7, 35.3 ft. LT of Rel. Shumway Hollow CL	Date Drilled: 01/30/07

WATER OBSERVATIONS: Water seepage at: 13.5'-15.0'
Water level at completion: 36.1' (prior to coring)
18.3' (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	660.2												
0.7	659.5						Topsoil - 8"						
		WOH 5 7	3	1		1.25	Medium stiff to stiff brown SANDY SILT (A-4a), trace to little clay; damp.						
5		1 4 5	18	2		0.75		3	3	--	32	30	32
5.5	654.7	4 7 7	18	3			Medium dense brown FINE SAND (A-3), trace coarse sand, trace silty clay; damp.						
10		45 37 28	18	4			@ 8.5'-10.0', very dense.						
		5 6 6	18	5			@ 11.0', trace clay.	0	7	--	88	6	
15		7 3 3	12	6			@ 13.5', loose, wet.						
15.5	644.7	WOH 6 9	18	7		0.5	Soft to medium stiff brown SILT AND CLAY (A-6a), trace fine sand; moist to wet.	0	0	--	0	56	43
17.5	642.7						Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured.						
20													
		Core 120"	Rec 120"	RQD 100%	R1		@ 22.5'-23.0', qu = 10,454 psi, Er = 2,484,015 psi.						
25													
30							@ 27.8'-27.9', broken zone.						



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

LOG OF: Boring B-26 | Location: Sta. 38+06.7, 35.3 ft. LT of Rel. Shumway Hollow CL | Date Drilled: 01/30/07

WATER OBSERVATIONS: Water seepage at: 13.5'-15.0'
Water level at completion: 36.1' (prior to coring)
18.3' (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	STANDARD PENETRATION (N)
30	630.2	Core 120"	Rec 120"	RQD 95%	R2		Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 32.0'-32.5', qu = 12,453 psi.							
37.5	622.7						Bottom of Boring - 37.5'							

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

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

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

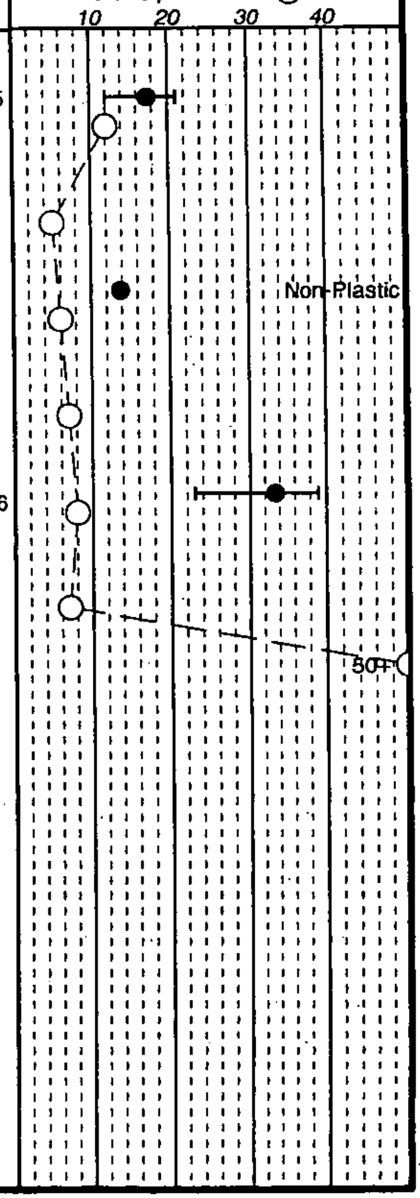
LOG OF: Boring B-27 | Location: Sta. 38+04.1, 39.7 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 01/29/07 to 01/30/07

WATER OBSERVATIONS:
Water seepage at: 8.5'-11.6', 13.0'-15.5'
Water level at completion: 14.4' (prior to coring)
35.8' (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	656.8												
0.4	656.4						Topsoil - 5"						
		1 5 7	18	1		1.75	Stiff brown SANDY SILT (A-4a), some clay, trace gravel; moist.	1	4	--	33	36	25
3.0	653.8						Loose brown COARSE AND FINE SAND (A-3a), trace to little clay, trace silt; damp.						
		4 2 3	18	2									
		5 3 3	18	3				0	7	--	80	13	
		3 3 4	18	4			@ 8.5'-10.0', wet.						
11.6	645.2	3 5 3	18	5A 5B		0.75	Medium stiff brown SILTY CLAY (A-6b), trace fine to coarse sand; moist.	0	1	--	1	52	46
13.0	643.8						Loose brown COARSE AND FINE SAND (A-3a), trace clay; wet.						
		5 4 3	18	6									
		50/3	3	7			@ 16.0', contains rock fragments.						
16.5	640.3						Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 16.5'-18.5', broken zone. @ 17.1'-17.5', lost recovery.						
		Core 120"	Rec 119"	RQD 80%	R1		@ 21.5'-22.0', qu = 13,148 psi, Er = 2,674,792 psi.						

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

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



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

LOG OF: Boring B-27 | Location: Sta. 38+04.1, 39.7 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 01/29/07 to 01/30/07

WATER OBSERVATIONS: Water seepage at: 8.5'-11.6', 13.0'-15.5'
Water level at completion: 14.4' (prior to coring)
35.8' (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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	626.8	Core 120"	Rec 120"	RQD 100%	R2		Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 32.0'-32.5', qu = 12,949 psi. @ 33.7', calcareous.							
35														
36.5	620.3						Bottom of Boring - 36.5'							
40														
45														
50														
55														
60														

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

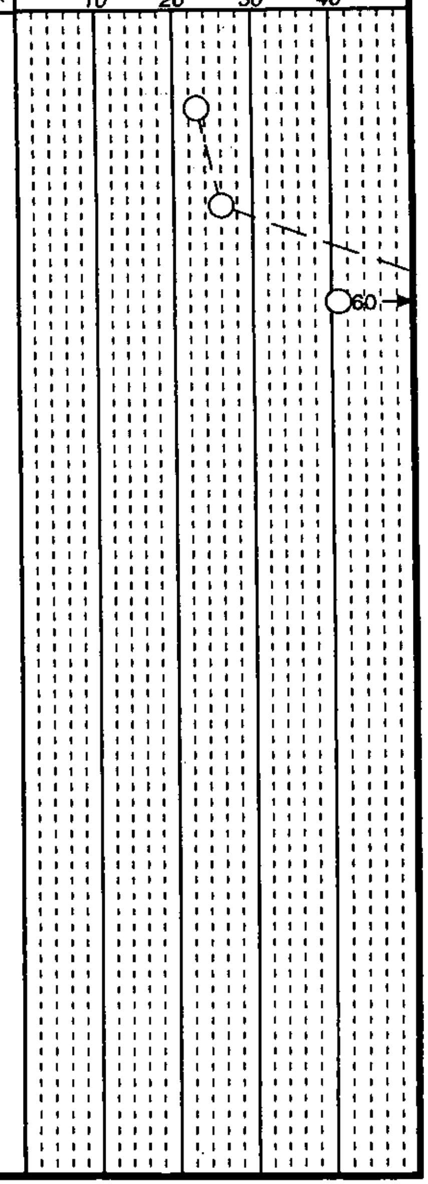
LOG OF: Boring TR-27 — Location: Sta. 35+91.3, 5.9 ft. LT of Rel. Shumway Hollow CL — Date Drilled: 8/25/04

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

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

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

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



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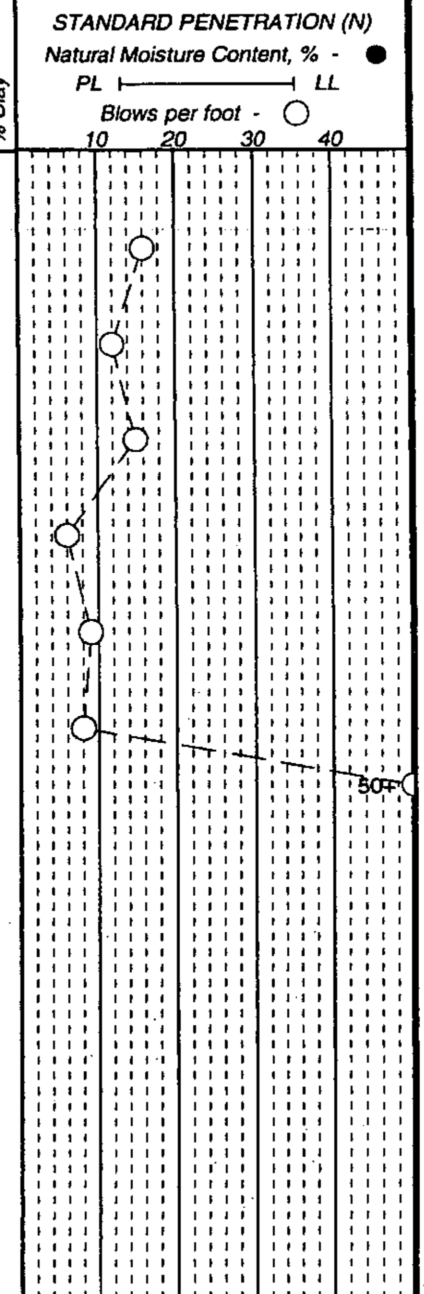
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

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

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

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



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Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring TR-28	Location: Sta. 38+20.7, 17.8 ft. RT of Rel. Shumway Hollow CL	Date Drilled: 02/02/05

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

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

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: Transystems Corp / ODOT D-9 | PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass | SHEET NO. 1 OF 1
SUBJECT: Shumway Hollow Rd over CSX RR | COMP. BY SJK | DATE 11-14-06
Log of Railroad Rock Cut | CHECKED BY ______ | DATE ______

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

Elev (ft)

660

Soil/Brush ~ Brush / Grass Semi-mature hardwood trees ~ Soil/Brush

645

Soft to medium Hard brown Sandstone highly weathered medium to thinly bedded, highly fractured.

638

Soft brown Sandstone interbedded with SHALE OR SILTSTONE highly weathered to decomposed, very thinly to thinly bedded, highly fractured.
Isolated areas of seepage noted in this layer

635

M. Hard brown SANDSTONE, moderately to highly weathered,

634

Residual Soil & Decomposed Rock Fragments.

627

Ditch Flow Line

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

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

* Elevation view of Eastern slope. View facing East.

Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring RR Cut	Location: Shumway over CSXT RR-Forward Abutment Location	Date Drilled: 9/15/06

WATER OBSERVATIONS: Water seepage at: Water level at completion:

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

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	660.0						Soil / Brush; contains semi-mature hardwood trees.						
15.0	645.0						Soft to medium hard brown SANDSTONE; highly weathered medium to thinly bedded, highly fractured. Vertical fractures were noted in the exposed rock. No significant lateral movement is evident from visual inspection of these fractures.						
22.0	638.0						Soft brown SANDSTONE; interbedded with shale or siltstone, highly weathered to decomposed, very thin to thinly bedded, highly fractured Isolated Seepage from elevation 635 to 638						
25.0	635.0						Medium hard brown SANDSTONE; moderately to highly weathered.						
26.0	634.0						Residual soil and highly weathered rock fragments.						

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

Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring RR Cut	Location: Shumway over CSXT RR-Forward Abutment Location	Date Drilled: 9/15/06

WATER OBSERVATIONS: Water seepage at: Water level at completion:

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	630.0													
33.0	627.0						@ 33.0' (elev 627) Ditch Flow Line							
							Bottom of Boring - 33.0'							
35														
40														
45														
50														
55														
60														

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

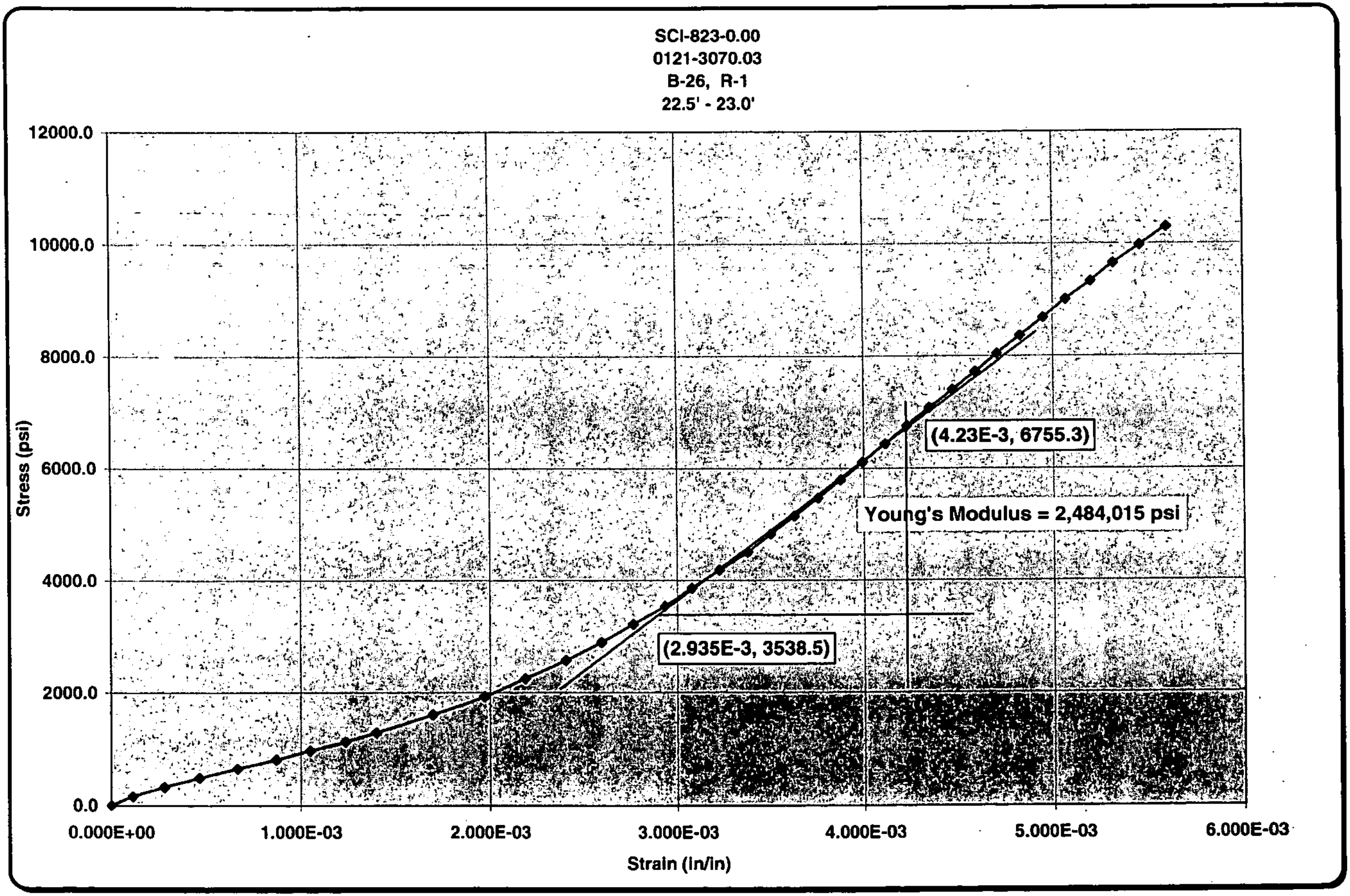
APPENDIX III
Laboratory Test Results

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

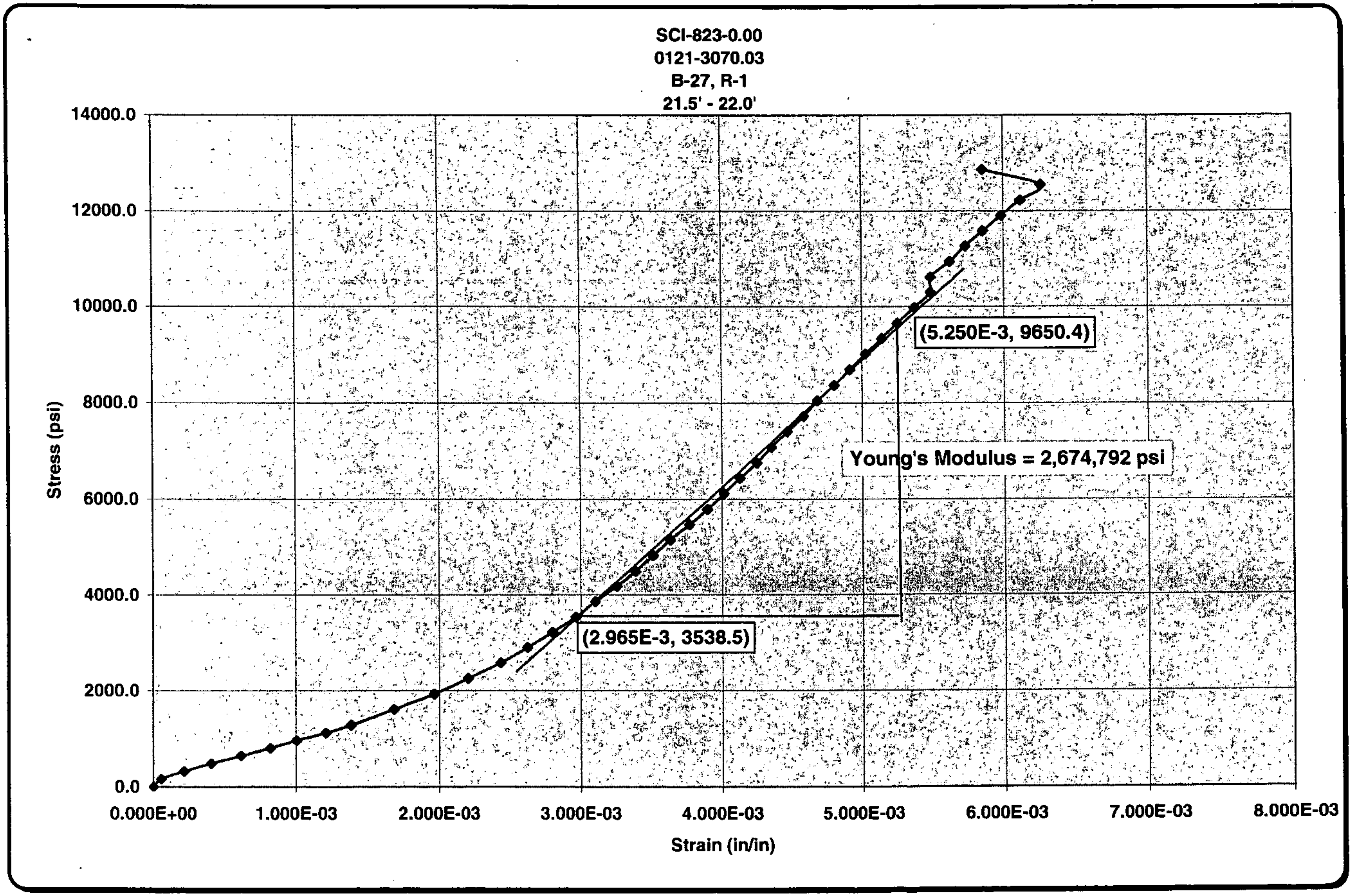
DLZ Project No.: 0121-3070.03 | **Client: TranSystems**

Project Name: SCI-823-0.00 | **Date: 2/2/07**

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-24	1	10.0'-10.5'	1.985	1.986	1.988	1.987	4.522	4.522	4.521	4.522	2.276	0.0081103	580.30	157.7	30,860	**9,952**
			1.987	1.989	1.987											
B-25	1	10.5'-11.0'	1.981	1.980	1.981	1.980	4.469	4.471	4.472	4.471	2.258	0.0079624	562.65	155.8	31,700	**10,295**
			1.981	1.980	1.977											

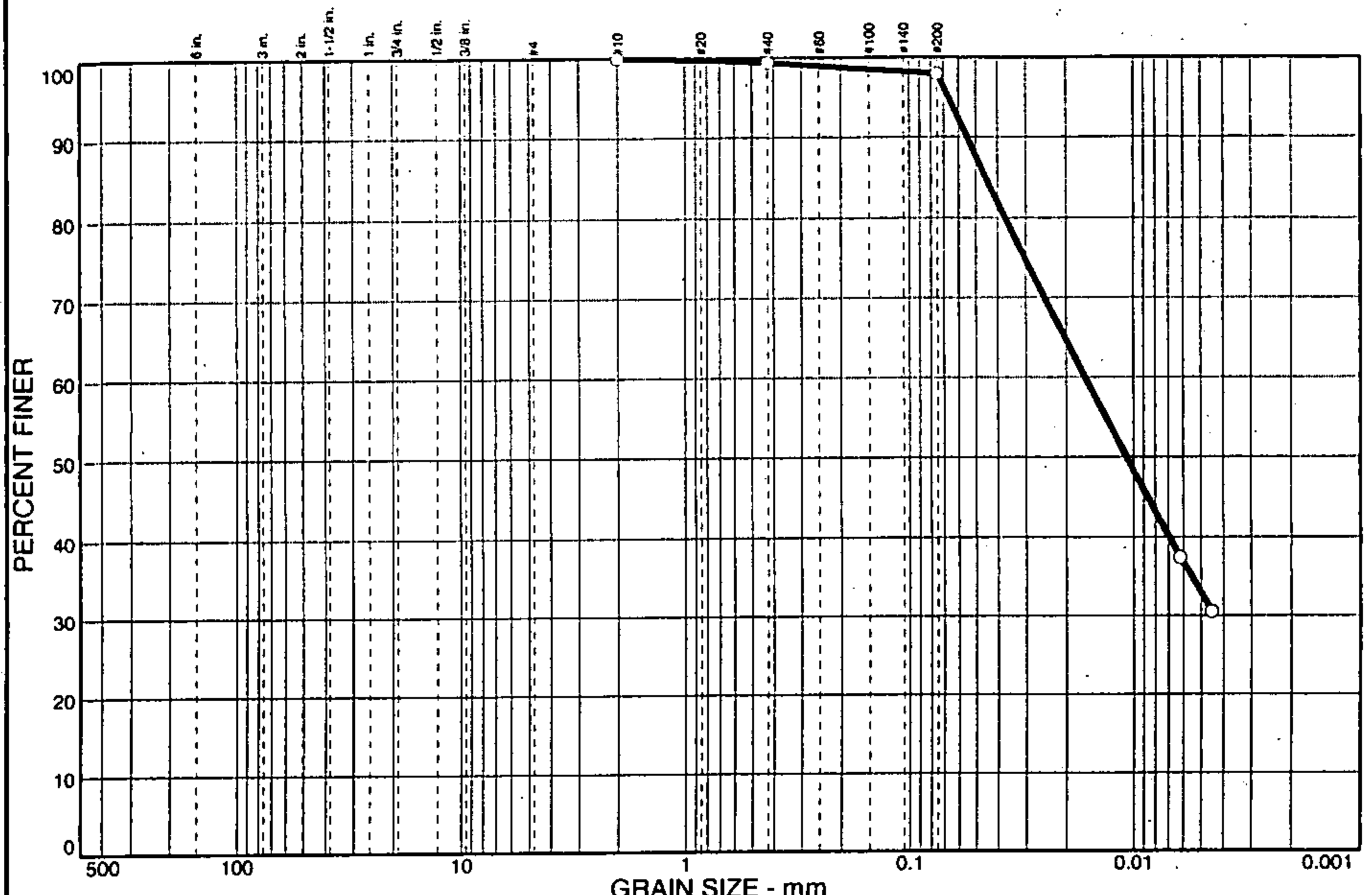


01-307003 B-26 R1 Chart 1



01-307003 B27 R-1 Chart 1

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.6	1.5	65.1	32.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.4		
#200	97.9		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{85}= 0.0337 D_{60}= 0.0140 D_{50}= 0.0100
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

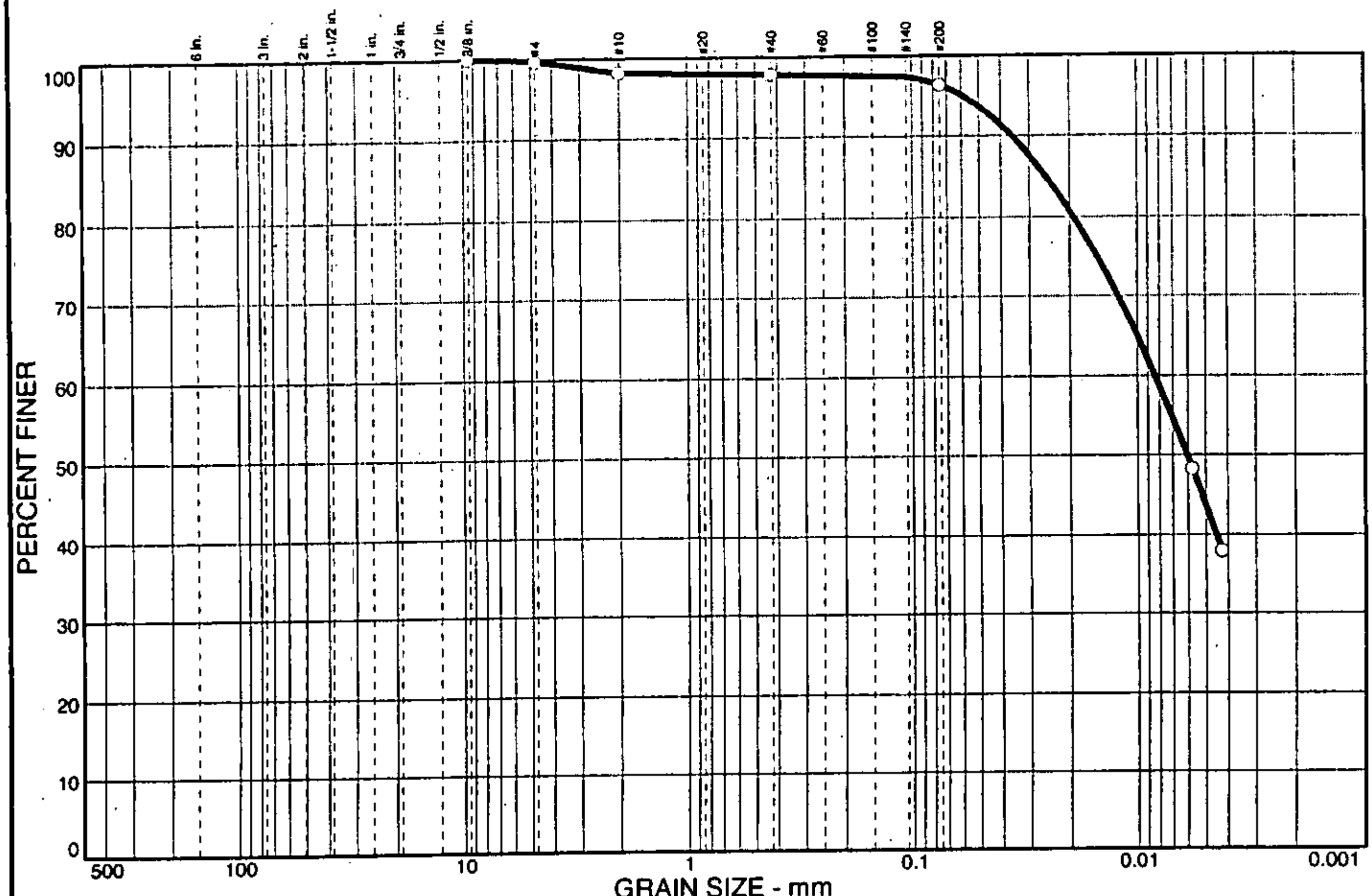
Moisture Content= 30.6%

Sample No.: 1 **Source of Sample:** B-24 **Date:** 2/12/07
Location: **Elev./Depth:** 1.0

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.3	1.5	0.4	1.5	52.8	43.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.7		
#10	98.2		
#40	97.8		
#200	96.3		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 22 LL= 35 PI= 13

Coefficients

D_{85}= 0.0255 D_{60}= 0.0084 D_{50}= 0.0061
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(13)

Remarks

Moisture Content= 31.2%

Sample No.: 1 **Source of Sample:** B-25 **Date:** 2/12/07
Location: **Elev./Depth:** 1.0

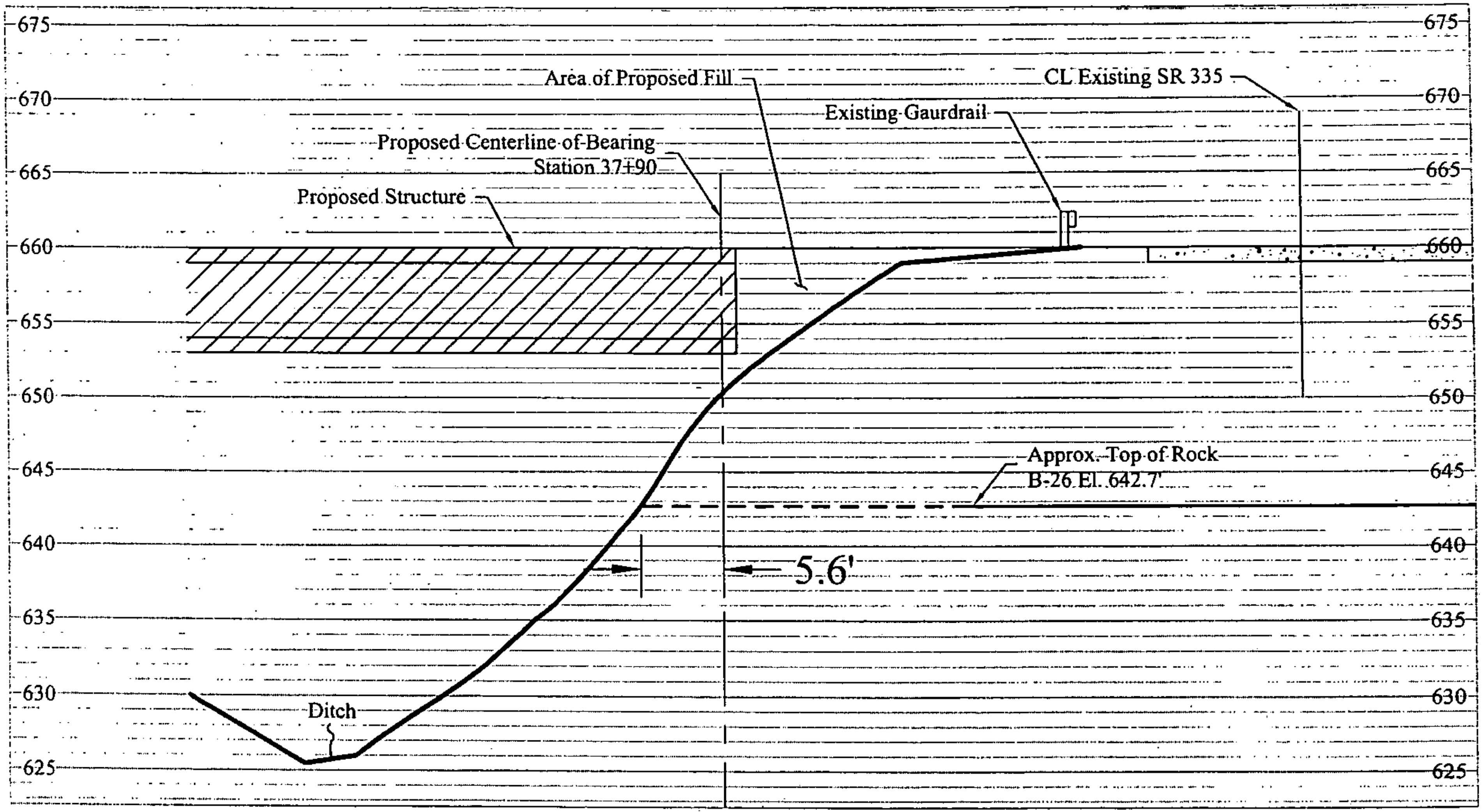
DLZ

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

APPENDIX IV

Forward Abutment Profile Drawings
MSE Wall Calculations
Forward Abutment Calculations
Sample LPILE Output File

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



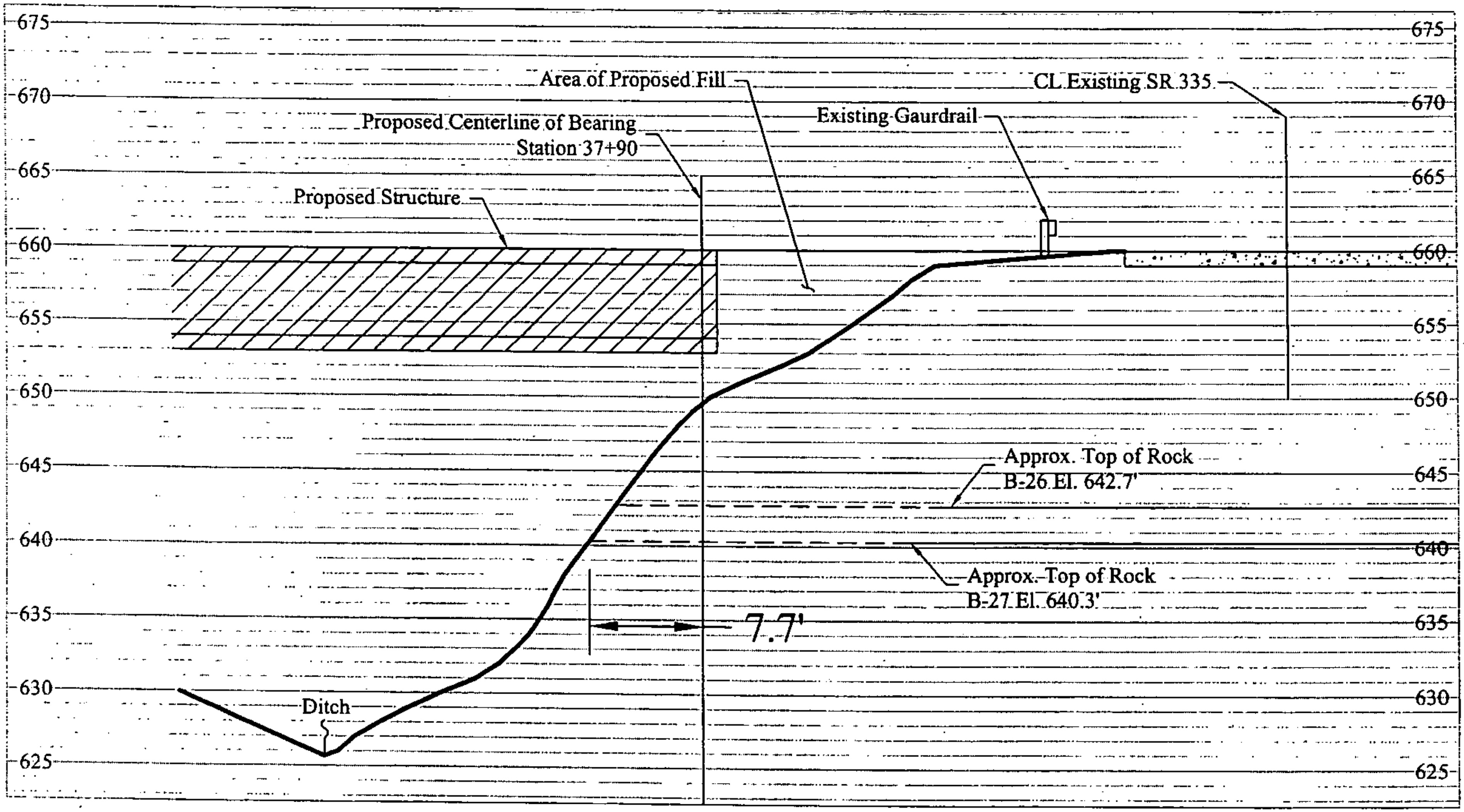
RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION 25' LEFT OFFSET		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 04/04/07

Sheet 1 of 11

SCALE: 1"=10'

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:07 AM, \\Dtztree\Q_geotechhp5100tn

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

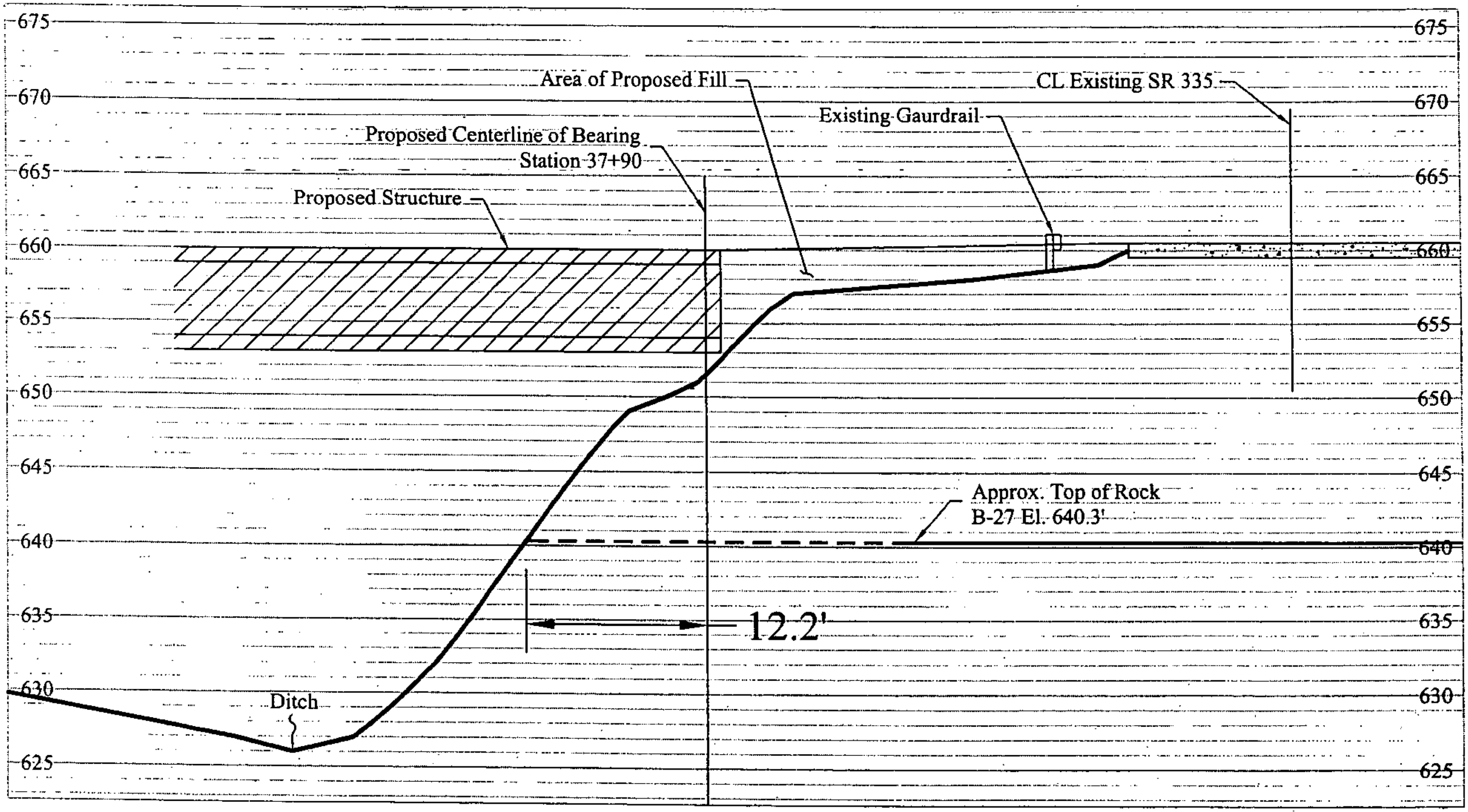


Sheet 2 of 11

RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION ON BASELINE		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 04/04/07

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:21 AM, \\Dtztree\Q_geotechhp\5100tn SCALE: 1"=10'

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



sheet 3 of 11

M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:33 AM SCALE: 1"=10'

RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR
FORWARD ABUTMENT LOCATION
25' RIGHT OFFSET

PROFILE (VIEW LOOKING NORTH)
SCI-823-0.00 PORTSMOUTH BYPASS

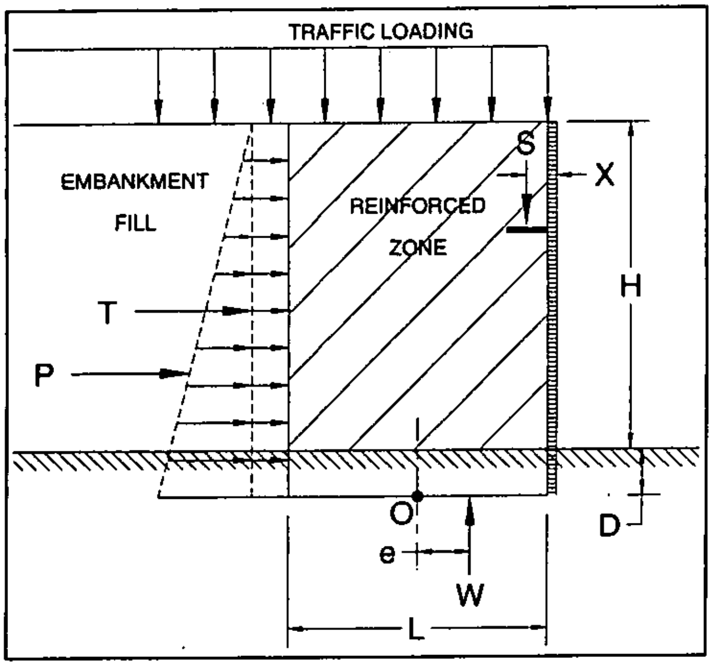
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 04/04/07

DLZ	SUBJECT			
	Client	TranSystems Corp	JOB NUMBER	0121-3070.03
	Project	SCI-823 Portsmouth Bypass	SHEET NO.	4 OF 11
	Item	MSE Wall Bearing Capacity	COMP. BY	SJR DATE 5-10-07
	Rear Abutment using spread footings		CHECKED BY	DAA DATE 9-12-07

Assumes 9' Wide Footing at qa=4 ksf

BEARING CAPACITY OF A MSE WALL *(Using Spread footings to Support Abutments)*

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	30.1	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	=	43	ft	
H	=	40	ft	Height of wall
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	21.5	ft	Γ Pa	=	14.3	ft
				Γ S	=	7.6	ft

B'	=	21.34	ft	
γ'	=	57.6	pcf	
W_t		7,224	lb/ft of wall	Weight from traffic
W_{mse}	=	155,316	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

3' Setback from MSE wall; 9' wide footing
X = 3' + (9'/2) = 7.5'

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Effective Bearing Pressure

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

σ_v = 9,304 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 30,322 psf

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

q_{ALL} = 12,129 psf

Factor of Safety = 3.26 **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 30,322 psf

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

q_{ALL} = 12.129 psf

Factor of Safety = 3.26 **OK**

Eccentricity of Resultant Force

e = 4.38 ft

Kern

e < L/6 = 5.02 ft

DLZ

SUBJECT		JOB NUMBER	0121-3070.03
Client	TranSystems Corp	SHEET NO.	5 OF 11
Project	SCI-823 Portsmouth Bypass	COMP. BY	SJR DATE 5-10-07
Item	MSE Wall Stability	CHECKED BY	DAA DATE 9-12-07
Rear Abutment, Based upon boring B-33			
Based on Compacted Granular Fill foundation			

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=40'
2 Sssume bridge is supported on spread footings
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 Neglect spread footing load (acts as resisting force)

Wall Properties

H+D = 43 feet
γ_{mse} = 120 pcf
L = 30.1 feet
L factor = 0.70
ϕ = 30 deg

Foundational Soil Properties

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

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 40,016 lbs per foot of wall

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

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

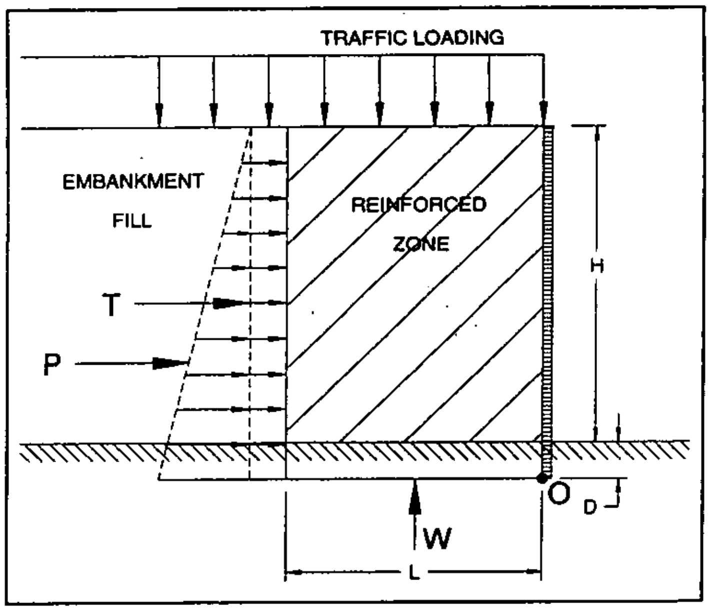
P_r = 69,892 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value



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

	Calculated	Required
	FS = **1.75**	FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 2,337,506 lb-ft $$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 597,967 lb-ft $$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

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

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

Resistance Against Overturning is **OK**

* From Tested Rock Core Samples $q_u = 9952$ psi (lower bound) $= 68.6$ MPa

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

$q_{max} = 4.83 [q_u (MPa)]^{0.51}$ { FHWA - IF - 99-025 Eq. 11.6
{ Drilled Shaft Const Procedures and Design Methods.

$q_{max} = 4.83 [68.6 \text{ MPa}]^{0.51} = 41.7$ MPa $= 6054$ psi $= 871$ ksf

$q_a = \frac{q_{max}}{F.S.} = \frac{871 \text{ ksf}}{3.0} = 290$ ksf

* Use $q_a = 80$ ksf for this type of rock at both rear and forward abutments.

2) Side Friction Assumes Smooth Rock Socket

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

$f_{max} = 0.65 (14.7 \text{ psi}) \left[\frac{9952}{14.7} \right]^{0.5} \leq 0.65 (14.7 \text{ psi}) \left[\frac{4500}{14.7} \right]^{0.5}$

$f_{max} = 249$ psi ≤ 167 psi

$f_{allow} = \frac{167 \text{ psi}}{3.0} = 55.7$ psi $= 8016$ psf

* Use $f_{allow} = 7500$ psf

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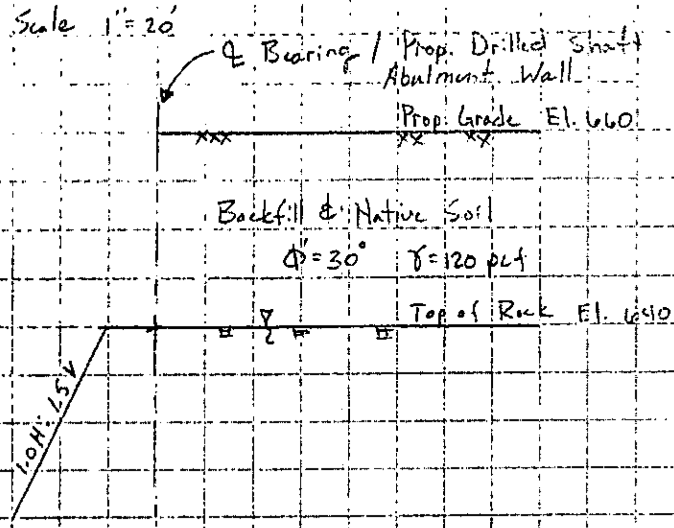
CLIENT: Transystems Corp / ODOT D-9
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Laterally Loaded Drilled Shafts
*Shumway over CSX, Forward Abutment

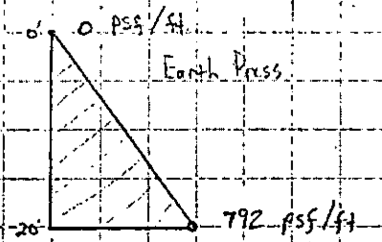
PROJECT NO. 0121-3070.03
SHEET NO. 7 OF 11
COMP. BY SJK DATE 5-9-07
CHECKED BY TAHA DATE 20-Aug-20

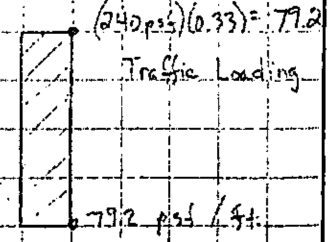
Scale 1" = 20'



Assuming K_a

Lateral Earth Pressure





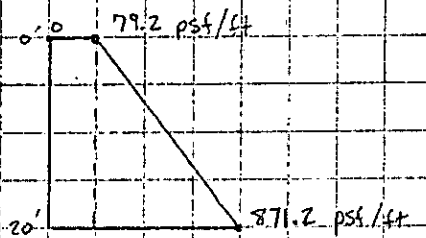
$P_{20} = \frac{1}{2} \cdot K_a \cdot \gamma \cdot H^2$ where $K_a = \tan^2(45 - \phi/2)$

$K_a = \tan^2(45 - 30/2) = 0.33$

$P_{20} = \frac{1}{2}(0.33)(120\ \text{pcf})(20')^2 + 0.33(240\ \text{psf})(20') = 9{,}504\ \text{lb/ft}$

Lateral Loading from Earth Pressure and Traffic

Combined



* Neglect passive resistance from soil.

* Assume groundwater table at the top of rock.

* Assume 3' Diameter drilled shafts on 2.5 D or 7.5'.

Total Lateral Force (7.5' Spacing) = 71,280 lb.

Assumptions for LPile analysis:

1) Proposed Grade = El. 660
2) Approx. top of rock = El. 640 (Boring B-27)
3) Assume Passive wedge cannot resist lateral pressures from El. 640 to 630
4) Assume rock socket development from El. 630 to 615.

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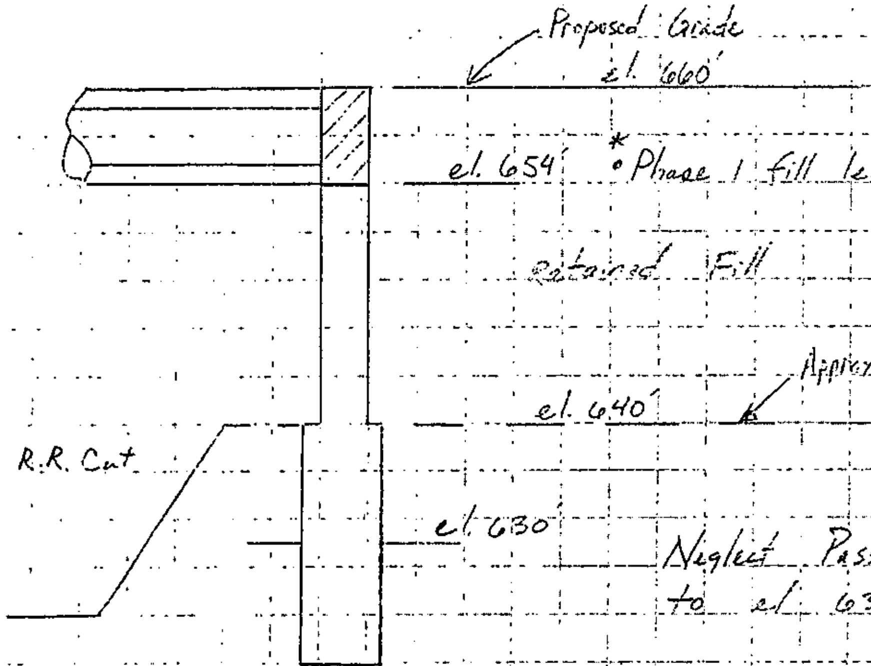
CLIENT: Transystem Corp / ODOT D-9 | PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass | SHEET NO. 8 OF 11
SUBJECT: Laterally Loaded Drilled Shafts | COMP. BY SAR | DATE 8-14-07
* Crossway over CSX, Forward Abutment | CHECKED BY TAA | DATE 20-Aug-20

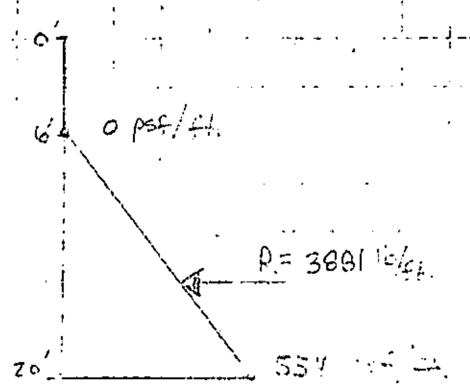


Neglect Passive Resistance from el. 640 to el. 630 in rock socket.

* Phase 1 fill level: Assume fill to elevation 654' prior to constructing superstructure.

Check deflection filled to el. 654'. Assuming K_a

• Neglect traffic loading for this case



$P = \frac{1}{2} K_a \gamma H^2$ where $K_a = \tan^2(45 - \frac{\phi}{2})$

$K_a = \tan^2(45 - \frac{30}{2}) = 0.33$

$P = \frac{1}{2}(0.33)(120 \text{ pcf})(20-6)^2 = 3881 \text{ lb.}$

$P_{20'} = H \cdot \gamma \cdot K_a = (20-6)(120 \text{ pcf})(0.33) = 554 \text{ psf/ft.}$

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CLIENT Tran Systems Corp / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Laterally Loaded Drilled Shafts
* Shumway over CSX Forward Abutment

PROJECT NO. 0121-3070.03
SHEET NO. 9 OF 11
COMP. BY SMK DATE 8-14-07
CHECKED BY TAH DATE

* Assume At-Rest Earth Pressures

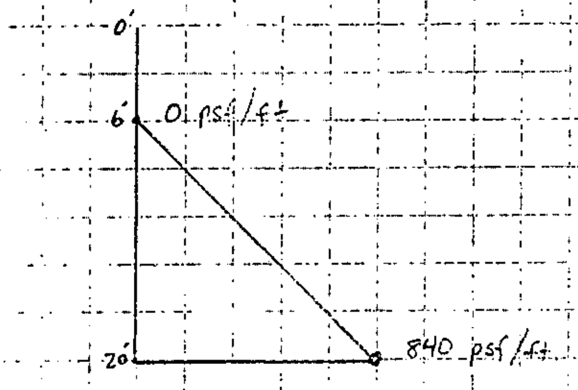
$K_o = (1 - \sin\phi)(1 + \sin\beta)$

1) Assume cohesionless backfill
2) Assume $\phi = \phi' = 30°$
3) $\beta = 0$ horizontal backfill

$K_o = (1 - \sin(30°))(1 + \sin 0) = 0.50$

* Phase 1 H = 14'

Lateral Pressure Distribution   Neglect traffic loading for this case

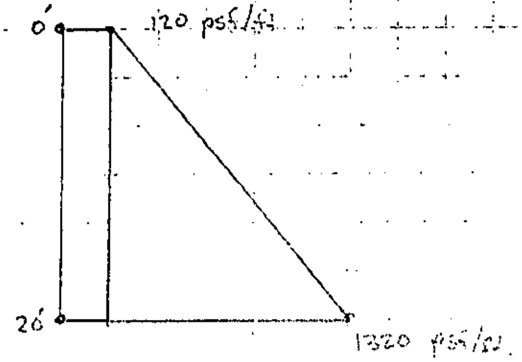


$p = H \cdot \gamma \cdot K_o$

$p_1 = 14'(120\ pcf)(0.50) = 840\ psf/ft$

* Phase 2 H = 20'

Lateral Pressure Distribution   * Include traffic loading $\omega_T = 240\ psf$



$p = H \cdot \gamma \cdot K_o + \omega_T\ K_o$

$p_1 = 0(120)(0.50) + 240(0.50) = 120\ psf/ft$

$p_2 = 20'(120\ pcf)(0.50) + 240(0.50) = 1320\ psf/ft$

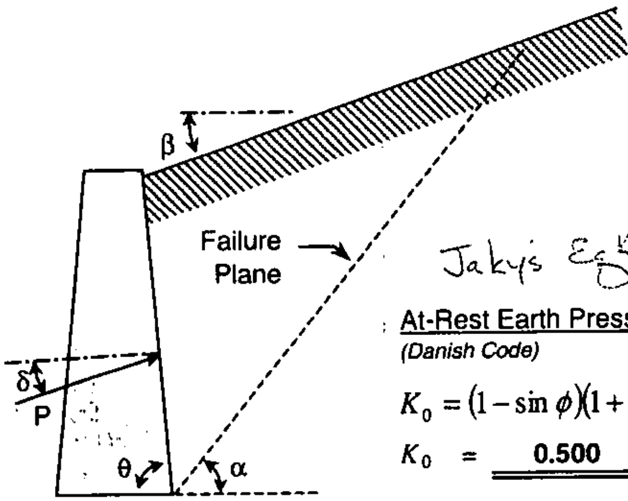
DLZ

CLIENT	Transystems Corp / ODOT D-9	JOB NUMBER	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	10 OF 11
SUBJECT	Laterally Loaded Drilled Shafts	COMP. BY	SAK DATE 8-14-07
	+ Shumway over CSX, Forward Abutment	CHECKED BY	[illegible] DATE 20-Aug-2007

EARTH PRESSURE COEFFICIENTS

Ref: EM 1110-2-2502 (1989) Retaining and Floodwalls

with corrections based on Bowles, J.E. (1988) Foundation Analysis and Design, 4th ed.



Parameters

ϕ	=	30	deg.	internal friction angle of soil
δ	=	0	deg.	angle of wall friction
θ	=	90	deg.	angle of wall face from horizontal
β	=	0	deg.	angle of backfill slope from horizontal

Jakys Eqn

At-Rest Earth Pressure

(Danish Code)

$$K_0 = (1-\sin\phi)(1+\sin\beta)$$

K_0 = **0.500**

Passive Earth Pressure

(Coulomb's Theory, wall friction must be less than $\phi/3$)

$$K_p = \frac{\sin^2(\theta-\phi)}{\sin^2\theta\cdot\sin(\theta+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\theta+\delta)\sin(\theta+\beta)}}\right]^2}$$

K_p = **3.000**

Active Earth Pressure

(Coulomb's Theory)

$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta\cdot\sin(\theta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$$

K_a = **0.333**

Angle between active failure plane and horizontal, α

$$\tan\alpha = \tan\phi + \sqrt{1+\tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$$

$\tan\alpha$ = 1.7321

α = **60.0** º

Recommended values for angle of wall friction, δ

- from U.S. Army Corps of Engineers, EM 1110-2-2502 (1989), page 3-37
  Active side, $\delta \le \phi/2$ Resisting side, $\delta = 0$ to $\phi/3$
- from NAVFAC 7.2 (1986) Foundations & Earth Structures, page 7.2-63

Mass concrete on the following foundation materials:	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, coarse sand	29 - 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 - 29
Clean fine sand, silty or clayey fine to medium sand	19 - 24
Fine sandy silt, nonplastic silt	17 - 19
Very stiff and hard residual or preconsolidated clay	22 - 26
Medium stiff and stiff clay and silty clay	17 - 19
(Masonry on foundation materials has same friction factors)	
Steel sheet piles against the following soils:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheet piling against the following soils:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 - 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	17 - 22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14

SCI-823 Porsmouth Bypass
Relocated Shumway Hollow Road over CSXT Railroad Bridge Structure

Forward Abutment Location
Results from LPile analyses Using non-linear EI Type III analysis

Assuming Active Condition, K_a=0.33

Diameter of Drilled Shafts (in)	Spacing c-c (in)	Retained Fill (ft)	Reinforcemetn Ratio, ρ (%)	*M_{max} (k-ft)	*V_{max} (k)	Deflection at pile head, δ (in)
36	36	14	5	178	60	0.149
36	36	20	5	502	187	0.809
48	48	14	5	237	63	0.065
48	48	20	5	669	177	0.221
48	72	14	5	355	94	0.098
48	72	20	5	1003	269	0.452
48	96	14	5	473	125	0.131
48	96	20	5	1338	377	0.663

Assuming At-Rest Condition, K_0=0.50

Diameter of Drilled Shafts (in)	Spacing c-c (in)	Retained Fill (ft)	Reinforcemetn Ratio, ρ (%)	*M_{max} (k-ft)	*V_{max} (k)	Deflection at pile head, δ (in)
36	36	14	5	269	91	0.312
36	36	20	5	760	305	1.277
48	48	14	5	359	95	0.099
48	48	20	5	1013	272	0.465
48	60	14	5	449	119	0.124
48	60	20	5	1267	357	0.617
48	72	14	5	538	142	0.149
48	72	20	5	1520	429	0.766
48	96	14	5	718	190	0.230
48	96	20	5	2027	584	1.050

*Maximum moment and shear are unfactored values taken directly from the results of LPile analyses

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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

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This program is licensed to:

S Riedy
DLZ, Ohio Inc.

Path to file locations: M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Final\LPile Preliminary\Report\
Name of input data file: 48 in shaft on 60 in centers 20 ft stage.lpd
Name of output file: 48 in shaft on 60 in centers 20 ft stage.lpo
Name of plot output file: 48 in shaft on 60 in centers 20 ft stage.lpp
Name of runtime file: 48 in shaft on 60 in centers 20 ft stage.lpr

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Time and Date of Analysis

Date: September 5, 2007 Time: 16:29: 8

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 includes automatic computation of pile-top deflection vs. pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

48 in shaft on 60 in centers 20 ft stage.lpo

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 200
- Deflection tolerance for convergence = 1.0002E-04 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 = 1000.00 in
Depth of ground surface below top of pile = 360.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	48.00000000	260576.0000	1810.0000	5000000.
2	1000.0000	48.00000000	260576.0000	1810.0000	5000000.

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 1 layers

Layer 1 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 1000.000 in

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	360.00	.08100
2	1000.00	.08100

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	360.000	10000.00000	.00	------	------
2	1000.000	10000.00000	.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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Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	50.00000
2	240.000	550.00000

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 = .000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = .000 lbs

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

Output of p-y Curves at Specified Depths

p-y curves are generated and printed for verification at 2 depths.

Depth No.	Depth Below Pile Head in	Depth Below Ground Surface in
1	380.000	20.000
2	420.000	60.000

Depth of ground surface below top of pile = 360.00 in

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

Number of pile sections = 1

Pile Section No. 1

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

Outside Diameter = 48.0000 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	=	22
Area of Single Bar	=	4.00000 In**2
Number of Rows of Reinforcing Bars	=	11
Cover Thickness (edge to bar center)	=	2.500 In

Unfactored Axial Squash Load Capacity = 11864.96 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	8.000000	21.2812
2	8.000000	19.5571
3	8.000000	16.2486
4	8.000000	11.6238
5	8.000000	6.0572
6	8.000000	.0000
7	8.000000	-6.0572
8	8.000000	-11.6238
9	8.000000	-16.2486
10	8.000000	-19.5571
11	8.000000	-21.2812

48 in shaft on 60 in centers 20 ft stage.lpo

Axial Thrust Force = .00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
1593392.	1.593392E+12	.00000100	.00002406	24.06134033
7884053.	1.576811E+12	.00000500	.00012030	24.06060791
7884053.	8.760058E+11	.00000900	.00015868	17.63104248
11249830.	8.653715E+11	.00001300	.00022967	17.66693115
14673859.	8.631682E+11	.00001700	.00030095	17.70318604
18079816.	8.609436E+11	.00002100	.00037254	17.74017334
21467147.	8.586859E+11	.00002500	.00044444	17.77752686
24835428.	8.563941E+11	.00002900	.00051664	17.81524658
28184871.	8.540870E+11	.00003300	.00058918	17.85406494
31514135.	8.517334E+11	.00003700	.00066204	17.89288330
34823813.	8.493613E+11	.00004100	.00073524	17.93280029
38112746.	8.469499E+11	.00004500	.00080879	17.97308350
41380855.	8.445073E+11	.00004900	.00088269	18.01409912
44627675.	8.420316E+11	.00005300	.00095696	18.05584717
47852710.	8.395212E+11	.00005700	.00103160	18.09832764
51055436.	8.369744E+11	.00006100	.00110663	18.14154053
54235301.	8.343892E+11	.00006500	.00118206	18.18548584
57391719.	8.317641E+11	.00006900	.00125788	18.23016357
60524629.	8.291045E+11	.00007300	.00133414	18.27593994
63588474.	8.258243E+11	.00007700	.00141041	18.31695557
66007072.	8.149021E+11	.00008100	.00148130	18.28765869
67892865.	7.987396E+11	.00008500	.00154732	18.20379639
69626546.	7.823207E+11	.00008900	.00161225	18.11517334
71346862.	7.671706E+11	.00009300	.00167742	18.03680420
72509588.	7.475215E+11	.00009700	.00173646	17.90167236
73643705.	7.291456E+11	.00010100	.00179546	17.77679443
80056894.	6.111213E+11	.00013100	.00221564	16.91326904
83902810.	5.211355E+11	.00016100	.00261756	16.25811768
85502495.	4.476570E+11	.00019100	.00302486	15.83697510
86590492.	3.918122E+11	.00022100	.00345708	15.64288330
87450049.	3.484066E+11	.00025100	.00388711	15.48651123

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 85404.84888 In-Kip

**** WARNING ****

An unreasonable input value for uniaxial compressive strength has been specified for a soil defined using the vuggy limestone criteria. The input value is greater than 2000 psi. You should check your input data for correctness.

p-y Curve for Vuggy Limestone (Strong Rock) Criteria

```
Soil Layer Number               =        1
Depth below pile head           =     380.000 in
Depth below Ground Surface      =      20.000 in
Shaft Diameter                  =      48.000 in
Uniaxial Compressive Strength   =   10000.000 lbs/in**2
p-multiplier                    =     1.00000
y-multiplier                    =     1.00000
```

y, in	p, lbs/in
.00000	.00
.01920	192000.00
.02560	195200.00
.03200	198400.00
.03840	201600.00
.04480	204800.00
.05120	208000.00
.05760	211200.00
.06400	214400.00
.07040	217600.00
.07680	220800.00
.08320	224000.00
.08960	227200.00
.09600	230400.00
.10240	233600.00
.10880	236800.00
.11520	240000.00

p-y Curve for Vuggy Limestone (Strong Rock) Criteria

Soil Layer Number = 1
Depth below pile head = 420.000 in
Depth below Ground Surface = 60.000 in
Shaft Diameter = 48.000 in
Uniaxial Compressive Strength = 10000.000 lbs/in**2
p-multiplier = 1.00000
y-multiplier = 1.00000

y, in	p, lbs/in
.00000	.00
.01920	192000.00
.02560	195200.00
.03200	198400.00
.03840	201600.00
.04480	204800.00
.05120	208000.00
.05760	211200.00
.06400	214400.00
.07040	217600.00
.07680	220800.00
.08320	224000.00
.08960	227200.00
.09600	230400.00
.10240	233600.00
.10880	236800.00
.11520	240000.00

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 = .000 lbs
Specified moment at pile head = .000 in-lbs

48 in shaft on 60 in centers 20 ft stage.lpo

Specified axial load at pile head = .000 lbs

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

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	.617530	-7.08E-06	-8.85E-08	-.002011	6.52E-10	1.59E+12	0.000
10.000	.597424	2500.000	604.167	-.002011	.230259	1.59E+12	0.000
20.000	.577319	12083.	1416.667	-.002010	1.113	1.59E+12	0.000
30.000	.557214	30833.	2437.500	-.002010	2.840	1.59E+12	0.000
40.000	.537112	60833.	3666.667	-.002010	5.603	1.59E+12	0.000
50.000	.517013	1.04E+05	5104.167	-.002010	9.594	1.59E+12	0.000
60.000	.496921	1.63E+05	6750.000	-.002009	15.005	1.59E+12	0.000
70.000	.476839	2.39E+05	8604.167	-.002007	22.028	1.59E+12	0.000
80.000	.456772	3.35E+05	10667.	-.002006	30.855	1.59E+12	0.000
90.000	.436726	4.53E+05	12938.	-.002003	41.677	1.59E+12	0.000
100.000	.416709	5.94E+05	15417.	-.002000	54.687	1.59E+12	0.000
110.000	.396728	7.61E+05	18104.	-.001996	70.076	1.59E+12	0.000
120.000	.376796	9.56E+05	21000.	-.001990	88.036	1.59E+12	0.000
130.000	.356923	1.18E+06	24104.	-.001984	108.759	1.59E+12	0.000
140.000	.337125	1.44E+06	27417.	-.001975	132.437	1.59E+12	0.000
150.000	.317417	1.73E+06	30938.	-.001965	159.263	1.59E+12	0.000
160.000	.297817	2.06E+06	34667.	-.001953	189.427	1.59E+12	0.000
170.000	.278347	2.42E+06	38604.	-.001939	223.121	1.59E+12	0.000
180.000	.259030	2.83E+06	42750.	-.001923	260.538	1.58E+12	0.000
190.000	.239891	3.28E+06	47104.	-.001904	301.870	1.58E+12	0.000
200.000	.220959	3.77E+06	51667.	-.001881	347.308	1.58E+12	0.000
210.000	.202266	4.31E+06	56437.	-.001856	397.043	1.58E+12	0.000
220.000	.183845	4.90E+06	61417.	-.001827	451.269	1.58E+12	0.000
230.000	.165735	5.54E+06	66604.	-.001793	510.177	1.58E+12	0.000
240.000	.147976	6.23E+06	72000.	-.001756	573.959	1.58E+12	0.000
250.000	.130611	6.98E+06	74750.	-.001714	642.807	1.58E+12	0.000
260.000	.113689	7.73E+06	74750.	-.001668	711.654	1.58E+12	0.000
270.000	.097257	8.47E+06	74750.	-.001595	780.502	8.74E+11	0.000
280.000	.081795	9.22E+06	74750.	-.001493	849.349	8.71E+11	0.000
290.000	.067393	9.97E+06	74750.	-.001383	918.197	8.69E+11	0.000
300.000	.054137	1.07E+07	74750.	-.001264	987.044	8.67E+11	0.000
310.000	.042119	1.15E+07	74750.	-.001136	1055.892	8.65E+11	0.000
320.000	.031426	1.22E+07	74750.	-.000999	1124.739	8.65E+11	0.000
330.000	.022145	1.30E+07	74750.	-.000853	1193.587	8.64E+11	0.000
340.000	.014363	1.37E+07	74750.	-.000699	1262.434	8.64E+11	0.000
350.000	.008169	1.45E+07	74750.	-.000536	1331.281	8.63E+11	0.000
360.000	.003649	1.52E+07	-1.08E+05	-.000364	1400.129	8.63E+11	-36489.
370.000	.000891	1.23E+07	-3.35E+05	-.000205	1132.899	8.65E+11	-8908.836
380.000	-.000444	8.51E+06	-3.57E+05	-8.48E-05	783.616	8.73E+11	4444.234
390.000	-.000806	5.16E+06	-2.95E+05	-1.98E-05	475.265	1.58E+12	8055.813
400.000	-.000840	2.62E+06	-2.12E+05	4.82E-06	241.112	1.59E+12	8399.446
410.000	-.000709	9.15E+05	-1.35E+05	1.60E-05	84.320	1.59E+12	7091.677
420.000	-.000521	-77678.	-73270.	1.86E-05	7.154	1.59E+12	5209.353
430.000	-.000338	-5.50E+05	-30345.	1.66E-05	50.649	1.59E+12	3375.780
440.000	-.000189	-6.85E+05	-4029.044	1.27E-05	63.051	1.59E+12	1887.327
450.000	-8.29E-05	-6.30E+05	9550.113	8.61E-06	58.071	1.59E+12	828.505
460.000	-1.65E-05	-4.94E+05	14520.	5.08E-06	45.459	1.59E+12	165.375
470.000	1.88E-05	-3.40E+05	14406.	2.47E-06	31.325	1.59E+12	-187.995
480.000	3.28E-05	-2.05E+05	11827.	7.55E-07	18.922	1.59E+12	-327.919
490.000	3.39E-05	-1.04E+05	8492.685	-2.15E-07	9.539	1.59E+12	-338.911
500.000	2.85E-05	-35586.	5373.601	-6.52E-07	3.278	1.59E+12	-284.905
510.000	2.09E-05	3905.012	2906.242	-7.51E-07	.359666	1.59E+12	-208.566
520.000	1.35E-05	22539.	1190.021	-6.68E-07	2.076	1.59E+12	-134.678
530.000	7.49E-06	27705.	141.958	-5.10E-07	2.552	1.59E+12	-74.935
540.000	3.26E-06	25378.	-395.614	-3.44E-07	2.337	1.59E+12	-32.580

550.000	6.15E-07	19793.	-589.269	-2.02E-07	1.823	1.59E+12	-6.151
560.000	-7.85E-07	13593.	-580.752	-9.74E-08	1.252	1.59E+12	7.855
570.000	-1.33E-06	8178.110	-474.828	-2.91E-08	.753234	1.59E+12	13.330
580.000	-1.37E-06	4096.328	-339.814	9.43E-09	.377287	1.59E+12	13.673
590.000	-1.14E-06	1381.830	-214.226	2.66E-08	.127272	1.59E+12	11.445
600.000	-8.35E-07	-188.186	-115.254	3.04E-08	.017333	1.59E+12	8.350
610.000	-5.37E-07	-923.243	-46.643	2.69E-08	.085034	1.59E+12	5.372
620.000	-2.97E-07	-1121.055	-4.908	2.05E-08	.103253	1.59E+12	2.975
630.000	-1.28E-07	-1021.394	16.369	1.37E-08	.094074	1.59E+12	1.281
640.000	-2.27E-08	-793.676	23.909	8.04E-09	.073100	1.59E+12	.227439
650.000	3.28E-08	-543.214	23.408	3.85E-09	.050032	1.59E+12	-.327591
660.000	5.42E-08	-325.511	19.062	1.12E-09	.029981	1.59E+12	-.541705
670.000	5.52E-08	-161.979	13.596	-4.10E-10	.014919	1.59E+12	-.551531
680.000	4.60E-08	-53.600	8.539	-1.09E-09	.004937	1.59E+12	-.459700
690.000	3.34E-08	8.810	4.570	-1.23E-09	.000811	1.59E+12	-.334230
700.000	2.14E-08	37.796	1.827	-1.08E-09	.003481	1.59E+12	-.214290
710.000	1.18E-08	45.353	.165374	-8.20E-10	.004177	1.59E+12	-.118069
720.000	5.03E-09	41.103	-.676529	-5.49E-10	.003786	1.59E+12	-.050312
730.000	8.35E-10	31.822	-.969837	-3.20E-10	.002931	1.59E+12	-.008350
740.000	-1.36E-09	21.706	-.943388	-1.52E-10	.001999	1.59E+12	.013640
750.000	-2.20E-09	12.955	-.765152	-4.30E-11	.001193	1.59E+12	.022007
760.000	-2.22E-09	6.403	-.543894	1.77E-11	.000590	1.59E+12	.022244
770.000	-1.85E-09	2.077	-.340358	4.43E-11	.000191	1.59E+12	.018463
780.000	-1.34E-09	-.403766	-.181154	4.96E-11	3.72E-05	1.59E+12	.013378
790.000	-8.55E-10	-1.546	-.071533	4.35E-11	.000142	1.59E+12	.008546
800.000	-4.69E-10	-1.834	-.005374	3.29E-11	.000169	1.59E+12	.004685
810.000	-1.98E-10	-1.654	.027932	2.19E-11	.000152	1.59E+12	.001976
820.000	-3.04E-11	-1.276	.039330	1.27E-11	.000118	1.59E+12	.000304
830.000	5.67E-11	-.867287	.038015	5.99E-12	7.99E-05	1.59E+12	-.000567
840.000	8.94E-11	-.515493	.030710	1.65E-12	4.75E-05	1.59E+12	-.000894
850.000	8.97E-11	-.253083	.021756	-7.62E-13	2.33E-05	1.59E+12	-.000897
860.000	7.41E-11	-.080381	.013563	-1.81E-12	7.40E-06	1.59E+12	-.000741
870.000	5.35E-11	.018172	.007178	-2.00E-12	1.67E-06	1.59E+12	-.000535
880.000	3.41E-11	.063180	.002797	-1.75E-12	5.82E-06	1.59E+12	-.000341
890.000	1.86E-11	.074107	.000164	-1.32E-12	6.83E-06	1.59E+12	-.000186
900.000	7.74E-12	.066451	-.001152	-8.76E-13	6.12E-06	1.59E+12	-7.74E-05
910.000	1.06E-12	.051060	-.001592	-5.07E-13	4.70E-06	1.59E+12	-1.06E-05
920.000	-2.41E-12	.034610	-.001524	-2.39E-13	3.19E-06	1.59E+12	2.41E-05
930.000	-3.71E-12	.020573	-.001218	-6.55E-14	1.89E-06	1.59E+12	3.71E-05
940.000	-3.72E-12	.010251	-.000846	3.12E-14	9.44E-07	1.59E+12	3.72E-05
950.000	-3.09E-12	.003653	-.000505	7.49E-14	3.36E-07	1.59E+12	3.09E-05
960.000	-2.23E-12	.000144	-.000240	8.68E-14	1.32E-08	1.59E+12	2.23E-05
970.000	-1.35E-12	-.001139	-6.06E-05	8.37E-14	1.05E-07	1.59E+12	1.35E-05
980.000	-5.53E-13	-.001068	3.48E-05	7.67E-14	9.83E-08	1.59E+12	5.53E-06
990.000	1.81E-13	-.000443	5.34E-05	7.20E-14	4.08E-08	1.59E+12	-1.81E-06
1000.	8.87E-13	0.000	0.000	7.06E-14	0.000	1.59E+12	-8.87E-06

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 = .61752962 in
Computed slope at pile head = -.00201054
Maximum bending moment = 15201667. lbs-in

```
Maximum shear force              =  -357007.55169 lbs
Depth of maximum bending moment  =      360.00000 in
Depth of maximum shear force     =      380.00000 in
Number of iterations             =             12
Number of zero deflection points =              8
```

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

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

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 0.000	M= 0.000	0.0000	.6175296	1.5202E+07	-357008.

Pile-head Deflection vs. Pile Length

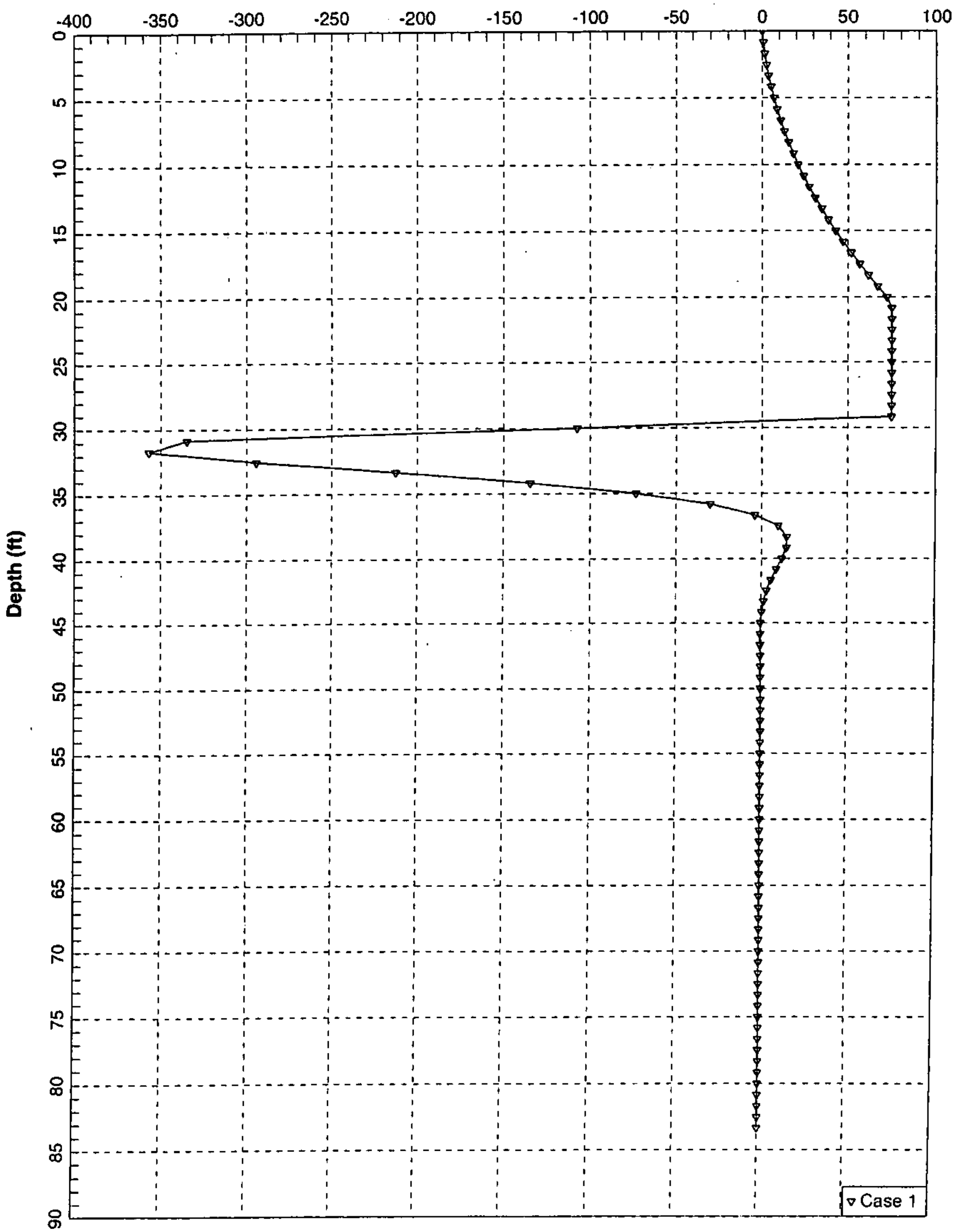
Boundary Condition Type 1, Shear and Moment

```
Shear      =            0. lbs
Moment     =            0. in-lbs
Axial Load =            0. lbs
```

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1000.000	.61752962	15201667.	-357007.55169
950.000	.62353982	15096851.	-356094.98536
900.000	.67037387	15417337.	-363864.33029
850.000	.65346184	15427181.	-362004.51737
800.000	.67988158	15732267.	-378337.37727
750.000	.67847649	15678281.	-378684.44173
700.000	.64872405	15267685.	-361007.69175
650.000	.63826064	14977361.	-355191.33770
600.000	.67200128	15516900.	-378042.03786
550.000	.63787495	15043766.	-362617.81619

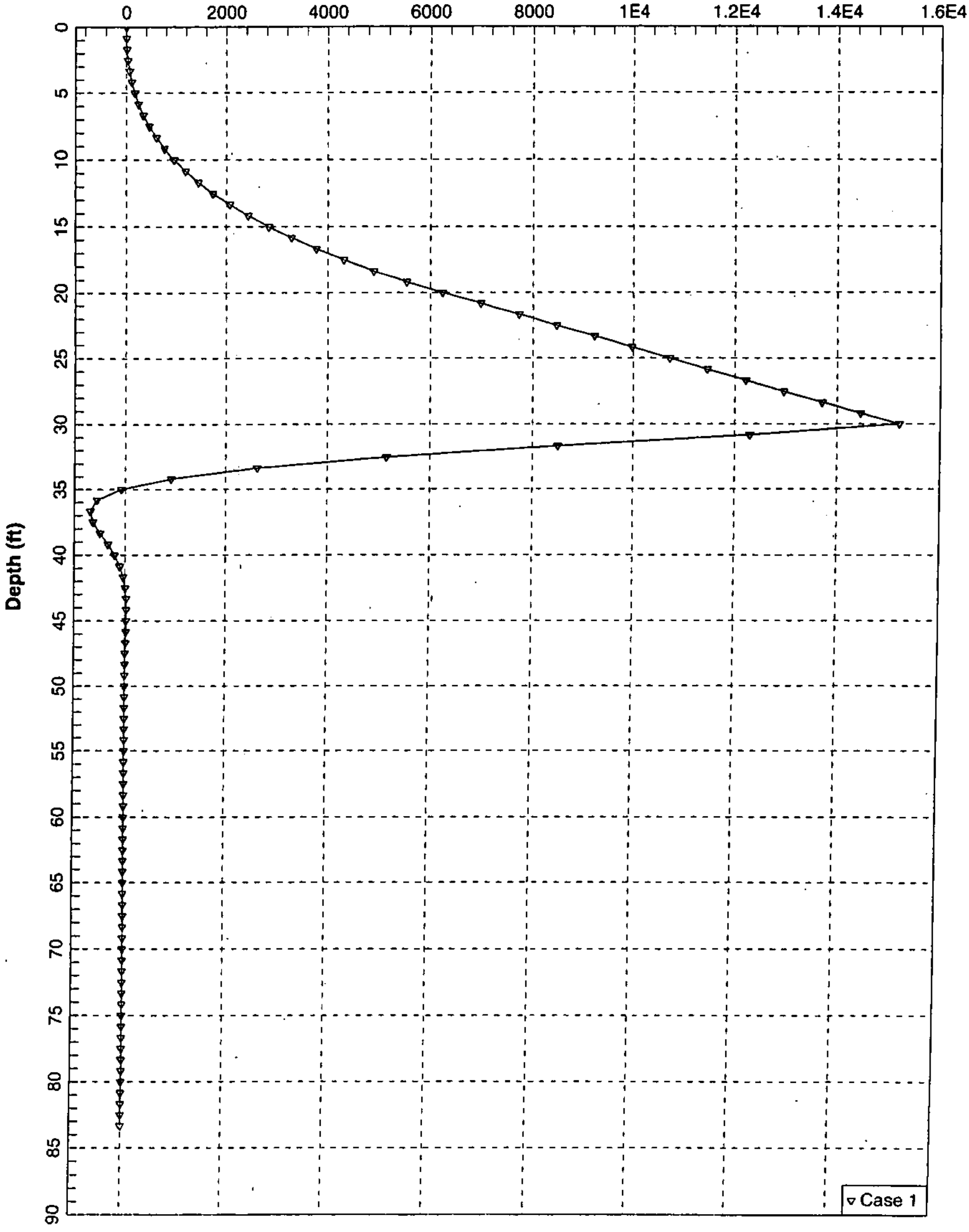
48 Inch Shafts on 60 inch spaicng, 20' Stage

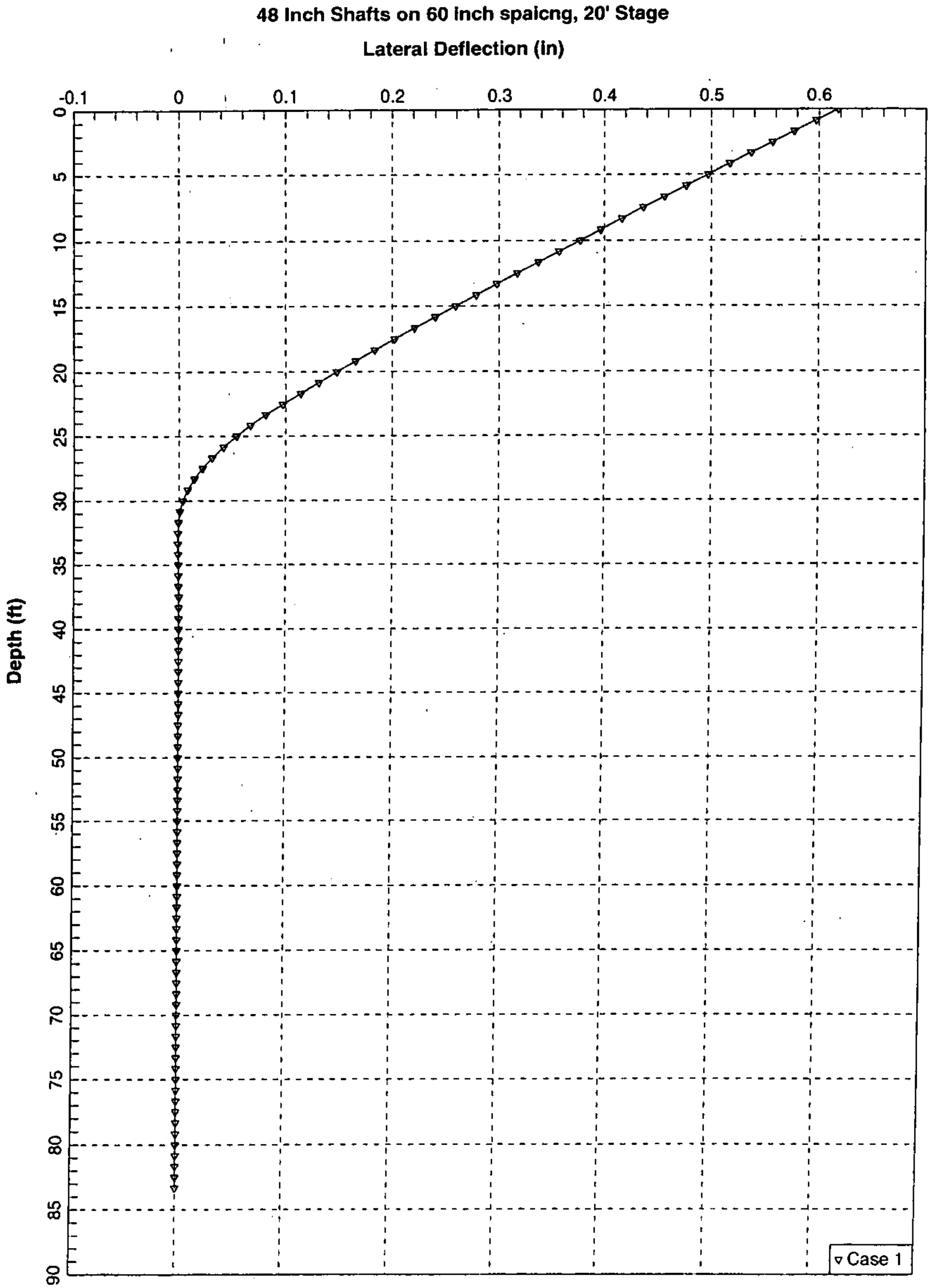
Shear Force (kips)



48 Inch Shafts on 60 inch spaicng, 20' Stage

Unfactored Bending Moment (in-kips)




48 Inch Shafts on 60 inch spaicng, 20' Stage
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
Case 1